Assessment of Pavement Acceptance Criteria and Quantifying its As-Constructed Material and Structural Properties

Karim Chatti, Ph.D.
Anshu Manik, Ph.D.
Imen Zaabar, Ph.D.

Michigan State University

Department of Civil and Environmental Engineering
Pavement Research Center of Excellence

Final Report
Project RC-1540
Contract/Authorization # 2006-0411 Z4
March 2011

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. MDOT Project Manager
RC-1540		Dave Smiley and Robert Pena

4. Title and Subtitle	5. Report Date
Assessment of Pavement Acceptance Criteria and Quantifying its As-constructed Material and Structural Properties	March 2011
	6. Performing Organization Code
7. Author(s) Karim Chatti, Ph.D. Anshu Manik, Ph.D. Imen Zaabar. Ph.D.	**8. Performing Org. Report No.**
9. Performing Organization Name and Address Michigan State University Department of Civil and Environmental Engineering, 3546 Engineering Building, Michigan State University East Lansing, MI 48824-1226	**10. Work Unit No. (TRAIS)** **11. Contract No.** 2006-0411 Z4 **11(a). Authorization No.**
12. Sponsoring Agency Name and Address Michigan Department of Transportation Construction and Technology Division P.O. Box 30049 Lansing, MI 48909	**13. Type of Report & Period Covered** Final Report **14. Sponsoring Agency Code**

15. Supplementary Notes

16. Abstract

State Highway Agencies (SHAs) have long realized the importance of quality assurance (QA) to ensure longer pavement service life. Construction QA programs are intended to ensure that the quality of the materials and construction in highway projects is satisfactory. Failure to conform to either material or construction specifications can result in the premature failure of pavements. The intent of this research project is to assess existing pavement acceptance criteria, conduct a feasibility study to consider the need of using new criteria that relate to pavement performance including the use of non-destructive testing, and develop a process to quantify as-constructed material and structural pavement properties and relate them to pavement service life. This was achieved by reviewing MDOT construction data, construction and performance data from the national LTPP database, reviewing QA programs of other state highway agencies, conducting a literature review of previous research studies on the subject, and finally running simulations using the new MEPDG software to relate quality characteristics to pavement performance. Finally, Monte-Carlo based simulations were developed to understand how these quality characteristics should be used in the specifications to encourage better quality and reduce the risk of overpayment or underpayment. A plan of action was also developed for MDOT to implement the recommendations in its quality assurance and design process. An important part of this is a feedback process for design.

17. Key Words	18. Distribution Statement
Quality assurance, acceptance criteria, as-constructed material and structural properties, MEPDG, risk analysis, end-result specifications, feedback to design.	No restrictions. This document is available to the public through the Michigan Department of Transportation.

19. Security Classification - report	20. Security Classification - page	21. No. of Pages	22. Price
Unclassified	Unclassified	411	

Table of contents

List of Figures

List of Tables

Introduction

CHAPTER 1: Introduction and Background

1.1 INTRODUCTION

State Highway Agencies (SHAs) have long realized the importance of quality assurance (QA) to ensure longer pavement service life. Construction QA programs are intended to ensure that the quality of the materials and construction in highway projects is satisfactory. Failure to conform to either material or construction specifications can result in the premature failure of pavements.

Currently, the strategies and practices used by state highway agencies to ensure quality employ a wide variety of QA approaches to meet the FHWA's Quality Assurance Procedures for Construction regulation (*1,2*). In many SHA specifications, quality assurance procedures require contractors to perform quality control (QC) tests and the state to perform acceptance tests. Frequently, these tests measure the same engineering properties. However, it is known that these results vary, even when taken from the same population. The effect of this variability in terms of the difference between as-constructed and design values on pavement performance needs to be assessed.

In general, materials/construction-related distresses refer to pavement failures that are a direct result of the properties of the materials, construction quality and their interactions with the environment to which they are exposed. In this sense, these failures are differentiated from others that might be associated with inadequate design for the traffic and environmental loading or the use of improper practices during pavement construction (*3*).

For flexible pavements, test methods used for in-place quality control and acceptance of individual pavement layers and of new and rehabilitated pavement systems have not changed much in past decades. Such quality control and acceptance operations typically rely on nuclear density measurements or the results of testing conducted on pavement cores. Roughness measurements are often used to confirm that the newly constructed pavement has an adequate initial smoothness. More recently, nondestructive testing (NDT) methods, including lasers, ground-penetrating radar, falling weight deflectometers, cone penetrometers, and infrared and seismic technologies, have been significantly improved and have shown potential for use in the quality control and acceptance of flexible pavement construction. Furthermore, the new Mechanistic-Empirical Pavement Design Guide (ME-PDG) will use pavement layer stiffness as a key material property. This will lead to increased measurement of layer moduli by owner agencies, an activity that is not at present a typical component in the acceptance of a completed project (*4*).

For rigid pavements, many methods exist for the nondestructive testing of concrete materials (*5-8*). Most of these testing techniques are highly refined and many have been standardized. However, these techniques are directed at strength determination rather than the identification of materials-related distress (MRD is normally a durability issue rather than a strength issue); the test methods that are directed at condition assessment as well as MRD will be considered.

This research project will assess pavement acceptance criteria, investigate the application of new non-destructive tests for measuring the quality of flexible and rigid pavements, and develop a process of quantifying as-constructed material and structural properties and relating their variability to pavement performance for different conditions including new, rehabilitation and preservation projects.

1.2 PROBLEM STATEMENT

The acceptance criteria for concrete (strength, air content, slump) and HMA (density, air voids, and asphalt content) have been used for decades and, therefore, need to be updated. The current acceptance criteria for pavements need to take into account other factors causing pavements to deteriorate. To be reliable, any acceptance criteria should relate to the performance experienced by the pavement structure (performance related specifications). Over time, numerous department investigations and research studies regarding distress initiation have found other factors that have contributed to the pavement's performance, both good and poor. These factors include adequate pavement support, varying material properties/characteristics, air-void distribution and sizes in concrete, excessive rates of concrete and HMA permeability, post-material (concrete) interaction chemistry, and asphalt film thickness.

Furthermore, when a pavement type and its cross section are determined during design, pre-established input values are used for material properties to counter the effects from truck loading. These same values are assumed to exist after construction occurs, if specifications are met. However, historical field sampling and testing for investigation projects after construction have found a wide variance in pavement material properties and the pavement's base/subbase support characteristics. Only a small portion of this variance can be attributed to natural aging and environmental effects. The remaining variance in pavement material properties occurs during construction that can cause pavement service life to vary within construction project limits.

The intent of this research project is to assess existing pavement acceptance criteria, conduct a feasibility study to consider the need of using new criteria that relate to pavement performance including the use of non-destructive testing, and develop a process to quantify as-constructed material and structural pavement properties and relate them to pavement service life.

1.3 OBJECTIVES

The objectives of this study are:

1. To conduct a feasibility study to consider the need of using new acceptance criteria/testing for pavements
2. To develop a feedback process for implementation during pavement design
3. To identify the pros/cons together with the impacts of using new criteria
4. To develop a plan of action for implementation

1.4 RESEARCH APPROACH

To achieve the objectives identified above, the following steps were taken.

(1) Collection and analysis of MDOT construction and performance data: Since the objective of this project is to assess the quality assurance program being used by the Michigan Department of Transportation, the most direct approach would be to gather construction data and specific details about projects, to assess if the QA program helped in ensuring that pavement construction quality was maintained up to the desired level and to verify if that did actually translate into a better performing pavement.

a. Determine if QA led to identification of projects on which quality control was good, fair or poor.

b. Assess if projects with poor quality control were corrected through suitable means and that the contractor was sufficiently penalized to recover the loss that MDOT would incur when improving the quality of the pavement.

c. Determine if those projects which were assessed to be of good quality have performed as well as or better than they were designed for.

This approach was meant to build on MDOT's on-going data gathering efforts to include collecting QC/QA data, design and construction data (including physical inventory data, material properties from in-situ and laboratory tests) from past projects as well as performance data (DI, RQI or IRI, rut depths and faulting etc.). This information was extracted from MDOT PMS database, records, and data files for pavement projects (both new construction and rehabilitation/preservation) that are longer than 1-mile. The information was extracted for the period between 1992 and 2001 (pavement performance data are not available for projects completed prior to 1992 and it is too early to analyze the pavement distress data for projects completed after the year 2001). The information was expected to include, if available, project identification (control section, project number, and BMP and EMP), project completion and so forth. It was decided that an effort will be made to collect data from a range of projects with varying levels of compliance in the respective specifications. However, an intensive search for MDOT's quality assurance and quality control data led to the finding that most of such data remain unaccounted for. Therefore, alternative approaches to relate the quality characteristics used in a QA program to actual pavement performance were indentified and are described in the following points.

(2) Analysis of LTPP data: It was decided that analysis of MDOT data as described in item (1) above would be supplemented with similar analysis on data extracted from the LTPP database. Chapter 6 and chapter 15 provide details of this effort for flexible and rigid pavements, respectively. Analysis performed on such data revealed that it is very difficult to reach firm conclusions using this data. This happens primarily because LTPP data was collected from real projects which have been constructed for a wide range of traffic using varied construction materials and in very different climates by different state agencies that differ in their design procedures and implementation details. This lead to too many factors varying among the projects and therefore it is almost impossible to isolate the effect of specific variables on pavement performance, although in some cases,

some general understandings can be derived regarding how different quality characteristics may affect pavement performance.

(3) Mechanistic-empirical analysis: Since empirical data analysis has shortcomings, mechanistic-empirical analysis is required to establish relationships between different candidate quality characteristics and pavement performance. The Mechanistic-Empirical Pavement Design Guide (MEPDG) is the most appropriate tool for such analysis. However, MEPDG does not include all the construction related inputs, particularly for rigid pavements (e.g., the time of the day when pouring of concrete is done). These variables are important for the prediction of early age cracking and/or built-in curling in concrete pavements. Therefore, an alternate software such as HIPERPAV can be used to study such construction related issues.

(4) Synthesis of empirical and mechanistic empirical analyses: This is the most important step in this project. Firstly, it requires preparing an exhaustive list of quality characteristics which should be considered for inclusion in the QA program. The different sources for preparing this list have been enumerated below.

a) Quality characteristics being used in other states' QA programs: Different states use varying combinations of quality characteristics. Some of these quality characteristics are used in determining payment to be made to the contractor for any project, while others are used merely to provide feedback for proper construction.
b) Quality characteristics being used in other states' QC program: Any of the quality characteristics that are used in a QC program, i.e., the variables that are monitored by the contractor and not used in the QA program should also be considered.
c) Quality characteristics used in the Mechanistic-Empirical Design Guide Software: The MEPDG software predicts pavement performance from the material, pavement structure, construction, traffic and environmental variables. The models used in the software are the result of studies carried out by many research teams after extensive testing and analysis to relate those variables to performance. Therefore, those variables or quality characteristics that are within the control of the contractor and can be tested at the time of construction should also be included in the list.
d) Quality characteristics studied in other research projects which have been shown to have impact on performance.

The second step is to shortlist those candidate QA variables that are known to affect pavement performance. These relationships can be established through steps described in items (1) through (3) above.

The third and last step is to identify those variables that should be included into the Michigan QA program. The criteria for including those variables are that:

a. They affect pavement performance either directly or in conjunction with other variables.
b. They need to be tested individually and cannot be estimated or calculated from other significant QA variables already being used in the QA program. For example there is a strong correlation between compressive strength and flexural strength of concrete.
c. It is feasible to test for them within a reasonable amount of time during the construction.
d. The testing for these candidate QA variables does not require very specialized or costly equipment.
e. It is possible for the contractor to control those variables through sound construction practices and tight quality control.

(5) Analyzing MDOT End-Result Specifications: Apart from how the individual quality characteristics affect pavement performance, it is important to understand how these quality characteristics should be used in the specifications to encourage better quality and reduce the risk of overpayment or underpayment. Monte-Carlo based simulations were developed to achieve this goal.

(6) Plan of Action: Using the conclusions and recommendations reached through steps (1) to (5) above, a plan of action was developed so that MDOT can implement the recommendations in its quality assurance and design process. An important part of this plan is a feedback process for design.

1.5 ORGANIZATION OF THE REPORT

This project addresses the quality assurance program for flexible and rigid pavements. Chapter 1 provides introduction, background, objectives and overall organization of the entire report. The rest of the report has been divided into two parts: Part I for flexible pavements and Part II for rigid pavements.

Part I - Flexible Pavements: This part begins with chapter 2.

Chapter 2 reports the effort that was made to collect actual quality assurance program data from Michigan pavement projects. An effort was made to collect data from MDOT projects with reasonably long performance history to be able to relate QA quality characteristics to pavement performance.

Chapter 3 contains the summary of MDOT's current QA program and the design process. Chapter 4 presents a broad picture of the types of specifications being used in QA programs across the United States with specific details from several states to be able to understand the current state of practice.

Chapter 5 documents the conclusions derived in other research studies which are of direct relevance to the objectives in this project. This knowledge base has been used in deriving conclusions regarding the importance of various quality characteristics from the point of view of pavement performance and in making recommendations.

Chapter 6 documents the details of analysis that was performed using data extracted from the Long Term Pavement Performance (LTPP) program. This analysis was intended to empirically explore relationships between different quality characteristics and pavement performance. Since it was found that firm conclusions are difficult to be drawn from empirical analysis, mechanistic-empirical analysis using MEPDG was conducted, and the details of this analysis have been presented in chapter 7.

Chapter 8 presents the details of a simulation which was developed to analyze the current MDOT QA program for flexible pavements. This analysis shows how to identify the strengths and weaknesses of a QA program and thereby improve it by suitable changes in the specifications. This was followed by MDOT construction data analysis and another simulation program to design an optimal feedback process for design presented in chapter 9. Chapter 10 presents the conclusions and recommendations for MDOT QA program for flexible pavements.

Part II presents the analysis performed and conclusions and recommendations for the rigid pavement QA program. The overall organization of this part is identical to that presented for flexible pavements above. Therefore, a similar outline is used for Part II.

Chapter 11: MDOT QA Data Collection Effort

Chapter 12: Review of MDOT's Current PCC QA Program and Design Process

Chapter 13: Survey of QA Programs in USA

Chapter 14: QA Variables and Rigid Pavement Performance-A Review

Chapter 15: Empirical Analysis

Chapter 16: Mechanistic-Empirical Analysis

Chapter 17: ERS Risk Analysis Using Simulation

Part I: Flexible Pavements

CHAPTER 2: MDOT Quality Assurance Data for Flexible Pavements

Certain projects were identified to get a sample of construction and materials test data collected on highway construction projects. Although the objective of this project is to evaluate quality *assurance* procedures, data from quality *control* procedures were also collected as part of the data collection effort. Different quality assurance procedures were followed at different times in the history of highway construction in Michigan. However, if suitable data were collected under the quality control procedures, they could be used with the current quality assurance procedure (or a slightly modified version of it like a shadow specification) to assess how the construction would be rated according to those specifications. This could then be related to the actual performance of those pavements. In other words, any material test data, whether under QC or QA, can be evaluated for a possible relationship with pavement performance. If any of such test results show strong correlation to performance, then they can be considered for quality assurance.

2.1 SELECTION OF SAMPLE PROJECTS

Two sources of data were identified:

(1) A list of projects for which performance data had already been processed by MDOT was used to identify the first set of sample projects for data collection.

(2) Actual construction documents collected from the record center of Michigan Department of Transportation and microfilms stored in the MDOT Construction & Technology (C&T) office.

Table 2.1 lists the projects which were selected as sample projects. All the data available at MDOT related to these projects were collected. This was done to understand the types of data that will be available for analysis at the time of the project.

Table 2.1 Sample flexible pavement projects selected for data mining.

REGION	ROUTE	CS	JN	BMP	EMP	LET	LOCATION
University	M-99 NB	33011	00434	4.260	5.178	12/21/1976	Victor Avenue to Moores River Drive
University	M-99 SB	33011	00434	4.310	5.210	12/21/1976	Victor Avenue to Moores River Drive
Grand	M-44 NB	41051	5745/2574	4.287	5.155	6/6/1990	N of I-96 to Windcrest Court
Grand	M-44 SB	41051	5745/2574	4.241	5.155	6/6/1990	N of I-96 to Windcrest Court
University	I-96 EB	33083	29581	0.000	2.348	2/4/1994	Ingham/Eaton Co. Line to Richard Road
University	I-96 WB	33083	29581	0.041	2.371	2/4/1994	Ingham/Eaton Co. Line to Richard Road
University	M-99 NB	23092	10729	2.085	9.229	1/18/1978	N of Petrieville Highway to N of Holt Road
University	US-27 NB	19033	20046	8.526	12.775	2/10/1993	Price Road to S of Wildcat Road
Superior	US-2	36022	37563	0.203	0.717	3/6/1998	River Avenue to N of Cayuga Street
Bay	M-53	44031	36021	1.592	2.835	3/3/1999	S of Water Street to N of Kingsbrook Dr.

Key: CS: Control Section
JN: Job Number
BMP: Beginning Mile Point
EMP: Ending Mile Point

2.2 SAMPLE DATA GATHERING

All the boxes and microfilms (if available) were searched for data related to the sample projects listed in Table 2.1. Table 2.2 summarizes the test data available for asphalt pavements. Appendix A gives a more detailed list of the types of data available.

For asphalt pavements, we have observed that very few tests were performed during construction for older projects; i.e., prior to 1990. Two of the sampled projects constructed in 1993 and 1994 have limited amount of core density and VMA data in the case of HMA pavements. Other quality characteristics used in contemporary QA programs are scarce. From all the projects selected as sample projects no quality characteristic data was obtained.

Considering the fact that there is scarcity of data required for determining the influence of various quality characteristics on pavement performance, it was deemed necessary to consider other alternatives rather than just relying on Michigan data. The alternatives are: (1) to collect relevant data from the Long Term Pavement Performance (LTPP) database and (2) to conduct mechanistic-empirical analysis using MEPDG and other performance models.

Table 2.2 Summary of test results collected from sampled HMA pavement projects.

Material Property	Sample			Material		Source					
	Truck	Mat	Virgin Material	Aggregate	Binder Also Mod. Polymer	Plant Mix	Top Course	Leveling Course	Base Course	Shoulder	Sub-Base
Actual Depth Measurement									2		2
Air Voids		1		2		2					
Asphalt Content						1					
Chert			1	1							
Crushed Material	1	3	4	4		1			2		
Density (Marshal)						2	1	1	1		
Density (Theoretical Maximum)						2	1	1	1		
Density (Average Core)		1				1					
Ductility		1	4		5	5					
Ductility of Residue (TFO)			4		5	5					
Elastic Recovery					1	1					
Fineness Modulus			1	1							
Flash Point		1	2		5	4					
Gradation	5	5	5	5	4	5	5	4	5	1	
Hard Absorbent Particles			1	1							
Incrusted Particles Less than 1/3 Area	1	1	2	2							
Incrusted Particles More than 1/3 Area	1	1	2	2	2	2					
Loss on Heating (TFO)		1	3		2	2					
Penetration (Original & Recovered)	4	1	4	1	5	5	4	5	3	1	
Penetration of Residue (TFO)		1	4		5	5					
Softened Particle	1	1	2	2							
Softening Point		1	1		3	2					
Solubility in Trichloroethylene		1	4		5	5					
Specific Gravity		1	2		3	3					
Spot Test			4		5	5					
Temperature of Mix	2		1	1		3	4	3	1	1	
Thin or Elongated Piece	1	1	1	1							
V. M. A.		1		2		2					
Viscosity (Also TFO)			4		5	5					
Wear Index			1	2		2	2				

CHAPTER 3: Review of MDOT QA Program and Design Process for HMA Pavements

3.1 Michigan DOT Quality Assurance Program

The Michigan Department of Transportation currently uses the following quality characteristics in their Quality Assurance (QA) program for Hot Mix Asphalt mixtures when calculating the payment to be made to the Contractor:

1. Air Voids (AV)
2. Voids in Mineral Aggregates (VMA)
3. Asphalt Binder Content (AC)
4. In-Place Density

The testing for these parameters, evaluated by the Engineer under the department's Quality Assurance Acceptance program, for Hot Mix Asphalt materials is performed on a Lot-by-Lot basis. Prior to beginning the work, individual Lot size is agreed upon by the Engineer and each Lot is divided into Sublots of approximately equal size and smaller than 1,000 tons.

If the total tonnage of a specific mixture does not exceed 5,000 tons, the total quantity of that mixture will be considered as a Lot and will be divided into a minimum of three and up to a maximum of seven approximately equal Sublots for testing and acceptance.

Quality Assurance & dispute resolution samples are taken within each Sublot through a random process managed by the Engineer. Each sample, weighing approximately 20,000 grams, is assigned an identifier by the Engineer and delivered to the testing facility, as specified in the HMA quality assurance plan, where one is tested and one is retained for possible appeal testing.

Within four calendar days after sampling, the following tests are conducted by the Engineer:

- Maximum Specific Gravity, G_{mm} , (MTM 314)
- Bulk Compacted Density, N_{Max} , (AASHTO TP 4-97)
- Air Voids, N_{ini} , N_{des} , N_{max} , (AASHTO PP28-97) (*For information only*)
- Voids in Mineral Aggregate, VMA , (AASHTO PP28-97)
- Voids Filled with Asphalt, VFA , (AASHTO PP28-97) (*For information only*)
- Ratio of Fines to Effective Asphalt Binder, (Passing #200 / P_{be})

- Composition of the Mixture:
  - **Method 1**: Asphalt binder content based on calculated value using sublot maximum specific gravity (Gmm) and current job mix formula (JMF) effective specific gravity (Gse); Gradation (ASTM C 136, and ASTM C 117) and crushed particle content (MTM 117) from extracted (AASHTO T 164) or incinerated aggregate (MTM 319).
  - **Method 2**: Asphalt binder content based on vacuum extraction (MTM 325 and the checklist for HMA mixture analysis vacuum extraction of the HMA Production Manual); Gradation (ASTM C 136, C117) and crushed particle content (MTM 117) based on extracted aggregate (AASHTO T 164).

After completion and final rolling of each Sublot four core samples are taken by the Contractor from locations specified and marked by the Engineer. It is cautioned that these random cores should be taken at a time which is independent of paving operations. Each core, approximately 6 inches in diameter, needs to be measured for thickness at the time of extraction. Any disqualified core based on minimum thickness criteria is discarded and the Engineer selects a new core location.

A Pavement In-Place Density acceptance test is completed by the Engineer in accordance with MTM 315, (Michigan Test Method for Bulk Specific Gravity and Density of Compacted HMA Mixtures using Saturated Surface-Dry Specimens) within four calendar days of core extraction. If more than fifty percent of cores in a Lot are disqualified, production has to stop and will not continue until the Engineer approves the conformation of contract application and paving operation.

The Engineer’s test results for the compacted HMA will be used as a basis of acceptance and payment. HMA pay items will be paid for according to contract prices for completed items on a Lot-by-Lot basis.

The Engineer will calculate percent within limits, pay factor, and payment in accordance with the procedures described in the following sub-sections.

3.1.1 Percent Within Limits

The percentage of each Lot within the specification limits established for each Quality Assurance parameter is determined as follows:

a) ***Arithmetic mean*** and ***standard deviation*** of the test results are computed:

$$\overline{X} = \frac{\sum X_i}{n}$$

$$S = \sqrt{\frac{\sum(X_i - \overline{X}^{2}}{n-1}}$$

where:

$\overline{X}$ = Arithmetic mean of test results

X_i = Test results

n = Number of test results

S = Standard deviation of the test results

b) Follow the flowchart below

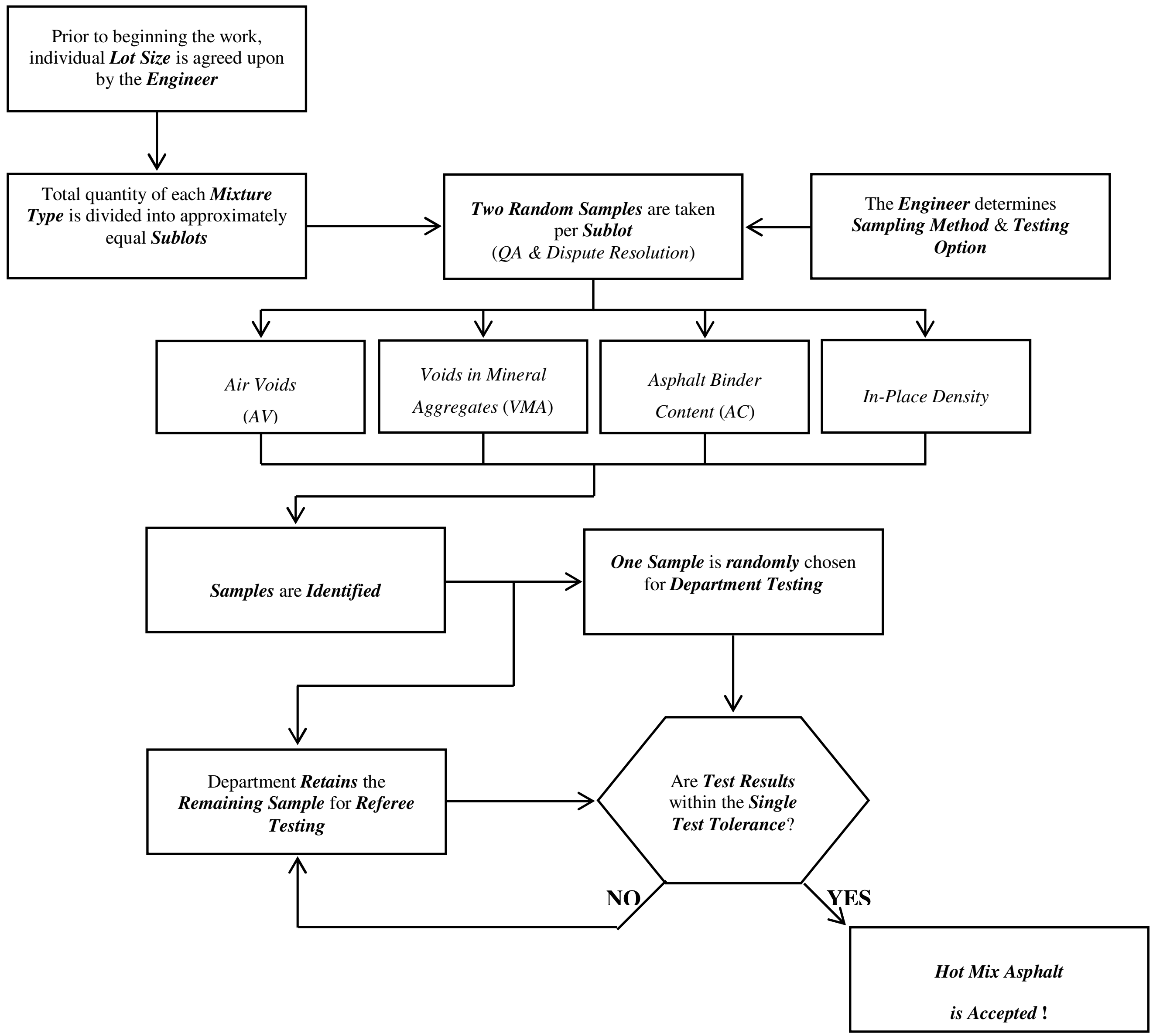


Figure 3.1 Flowchart for Acceptance Testing Procedure for HMA Pavements

c) ***Upper Quality Index***, QU, and ***Lower Quality Index***, QL, are determined using Upper Specifications Limit and Lower Specifications Limit. The results should be rounded to the nearest 0.05 according to ASTM E 29, Section 6.6. If any of Upper Specifications Limit or Lower Specifications Limit is not specified, the Upper Percent within Limits or Lower Specifications Limits will be 100:

$$Q_U = \frac{USL - \overline{X}}{S}$$

$$Q_L = \frac{\overline{X} - LSL}{S}$$

where:

$\overline{X}$ = Arithmetic mean of test results
USL = Upper Specifications Limit
LSL = Lower Specifications Limit
S = Standard deviation of the test results

d) ***Percentage of materials*** within the Upper Specification Limit, PU, and Lower Specification Limits, PL, is estimated according to Tables 3.1 and 3.2 (Table 106-1 of the Standard Specifications). For this purpose one should enter the table with QU or OL and follow the column which is appropriate to the total number of tests, n.

e) ***Quality Level*** stated as Percent within Limits, PWL, is calculated using the following formula. Note that all values of Percent within Limits, PWL, are Percents.

$$PWL = (P_U + P_L) - 100$$

If only an Upper Specification Limit or Lower Specification Limit applies, then PU or PL, respectively, is the Percent within Limits, PWL.

f) ***Percent Defective***, PD, if required to calculate pay adjustments, can be determined using

$$PD = 100 - PWL$$

Table 3.1 Estimated Percent Within Limits (Table 106-1 of Standard Specifications)

Q_U or Q_L	Percent Within Limits for Given Sample Sizes (a)									
	n=1 (b)	n=2 (b)	n=3	n=4	n=5	n=6	n=7	n=8	n=9	n=10
0.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00
0.05	55.10	51.68	51.38	51.67	51.78	51.84	51.87	51.89	51.91	51.92
0.10	60.20	53.36	52.76	53.33	53.56	53.67	53.74	53.78	53.82	53.84
0.15	65.31	55.03	54.15	55.00	55.33	55.50	55.60	55.67	55.71	55.75
0.20	70.41	56.71	56.54	56.67	57.10	57.32	57.46	57.54	57.60	57.65
0.25	75.51	58.39	56.95	58.33	58.87	59.14	59.30	59.41	59.48	59.53
0.30	80.61	60.07	58.37	60.00	60.63	60.94	61.13	61.25	61.34	61.40
0.35	85.71	61.74	59.80	61.67	62.38	62.73	62.94	63.08	63.18	63.25
0.40	90.82	63.42	61.26	63.33	64.12	64.51	64.74	64.89	65.00	65.07
0.45	95.92	65.10	62.74	65.00	65.84	66.27	66.51	66.67	66.79	66.87
0.50	100.00	66.78	64.25	66.67	67.56	68.00	68.26	68.43	68.55	68.63
0.55	100.00	68.46	65.80	68.33	69.26	69.72	69.99	70.16	70.28	70.36
0.60	100.00	70.13	67.39	70.00	70.95	71.41	71.68	71.85	71.97	72.06
0.65	100.00	71.81	69.03	71.67	72.61	73.08	73.34	73.51	73.63	73.72
0.70	100.00	72.49	70.73	73.33	74.26	74.71	74.97	75.14	75.25	75.33
0.75	100.00	75.17	72.50	75.00	75.89	76.32	76.56	76.72	76.83	76.90
0.80	100.00	76.85	74.36	76.67	77.49	77.89	78.12	78.26	78.36	78.43
0.85	100.00	78.52	76.33	78.33	79.07	79.43	79.63	79.76	79.84	79.90
0.90	100.00	80.20	78.45	80.00	80.62	80.93	81.10	81.21	81.28	81.33
0.95	100.00	81.88	80.75	81.67	82.14	82.39	82.52	82.61	82.67	82.71
1.00	100.00	83.56	83.33	83.33	83.64	83.80	83.90	83.96	84.00	84.03
1.05	100.00	85.23	86.37	85.00	85.09	85.18	85.23	85.26	85.28	85.29
1.10	100.00	86.91	90.16	86.67	86.52	86.50	86.51	86.51	86.5	86.50
1.15	100.00	88.59	97.13	88.33	87.90	87.78	87.73	87.70	87.68	87.66
1.20	100.00	90.27	100.00	90.00	89.24	89.01	88.90	88.83	88.79	88.76

Table 3.2 (Continued) Estimated Percent Within Limits (Table 106-1 of Standard Specifications)

Q_U or Q_L	Percent Within Limits for Given Sample Sizes (AASHTO R 9 Appendix C) (a)								
	n=1 (b)	n=2 (b)	n=3	n=4	n=5	n=6	n=7	n=8	n=9	n=10
1.25	100.00	91.95	100.00	91.67	90.54	90.19	90.02	89.91	89.85	89.79
1.30	100.00	93.62	100.00	93.33	91.79	91.31	91.07	90.94	90.84	90.78
1.35	100.00	95.30	100.00	95.00	92.98	92.37	92.08	91.90	91.78	91.70
1.40	100.00	96.98	100.00	96.67	94.12	93.37	93.02	92.81	92.67	92.56
1.45	100.00	98.66	100.00	98.33	95.19	94.32	93.90	93.65	93.49	93.37
1.50	100.00	100.00	100.00	100.00	96.20	95.19	94.72	94.44	94.26	94.13
1.55	100.00	100.00	100.00	100.00	97.13	96.00	95.48	95.17	94.97	94.82
1.60	100.00	100.00	100.00	100.00	97.97	96.75	96.17	95.84	95.62	95.46
1.65	100.00	100.00	100.00	100.00	98.72	97.42	96.81	96.45	96.22	96.05
1.70	100.00	100.00	100.00	100.00	99.34	98.02	97.38	97.01	96.76	96.59
1.75	100.00	100.00	100.00	100.00	99.81	98.55	97.89	97.51	97.25	97.07
1.80	100.00	100.00	100.00	100.00	100.00	98.99	98.35	97.96	97.70	97.51
1.85	100.00	100.00	100.00	100.00	100.00	99.36	98.74	98.35	98.09	97.91
1.90	100.00	100.00	100.00	100.00	100.00	99.65	99.07	98.69	98.44	98.25
1.95	100.00	100.00	100.00	100.00	100.00	99.85	99.35	98.99	98.74	98.56
2.00	100.00	100.00	100.00	100.00	100.00	99.97	99.57	99.24	99.00	98.83
2.05	100.00	100.00	100.00	100.00	100.00	100.00	99.74	99.45	99.23	99.06
2.10	100.00	100.00	100.00	100.00	100.00	100.00	99.86	99.61	99.41	99.26
2.15	100.00	100.00	100.00	100.00	100.00	100.00	99.94	9.74	99.57	99.42
2.20	100.00	100.00	100.00	100.00	100.00	100.00	99.99	99.84	99.69	99.56
2.25	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.91	99.79	99.68
2.30	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.96	99.86	99.77
2.35	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.98	99.92	99.84
2.40	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.95	99.89
2.45	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.98	99.93
2.50	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.99	99.96
2.55	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.98
2.60	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	99.99
2.65	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

3.1.2 Pay Factors for Quality Acceptance Items

3.1.2.1 Pay Factor for Air Voids – PF_{AV}:

A. If PWL for Air Voids (PWL_{AV}) is between 100 and 70, the following formula is used to determine PF_{AV}. The value of PF_{AV} is rounded off to two decimal places.

$$PF_{AV} = 55 + (0.5 \times PWL)$$

B. If PWL for Air Voids is between 70 and 50 inclusive, the following formula is used to determine PF_{AV}. The value of PF_{AV} is rounded off to two decimal places.

$$PF_{AV} = 37.5 + (0.75 \times PWL)$$

C. If PWL for Air Voids is less than 50, the Engineer may elect to do one of the following:

(1) Require removal and replacement of the entire Lot with new QA sampling and testing and repeat the evaluation procedure.
(2) Allow the Lot to remain in place and apply an Overall Lot Pay Factor of 50.00.
(3) Allow submittal of a corrective action plan for the Engineer's approval. The corrective action plan may include removal and replacement of one or more sublots. If one or more sublots are replaced, the Sublot(s) will be retested and the Overall Lot Pay Factor will be recalculated according to this special provision. If the Engineer does not approve the plan for corrective action, subsections (1) or (2) above will be applied.

3.1.2.2 Pay Factor for Binder Content (PF_{BINDER})

A. If PWL for Binder Content (PWL_{BINDER}) is between 100 and 70, the following formula is used to determine PF_{BINDER}. The value of PF_{BINDER} is rounded off to two decimal places.

$$PF_{BINDER} = 55 + (0.5 \times PWL)$$

B. If PWL for Binder Content is between 70 and 50 inclusive, the following formula is used to determine PF_{BINDER}. The value of PF_{BINDER} is rounded off to two decimal places.

$$PF_{BINDER} = 37.5 + (0.75 \times PWL)$$

C. If PWL for Binder Content is less than 50, the Engineer may elect to take one of the actions specified in description for PF_{BINDER} above.

3.1.2.3 Pay Factor for Voids in Mineral Aggregates – PF_{VMA}:

A. If PWL for VMA (PWL_{VMA}) is between 100 and 70, the following formula is used to determine PF_{VMA}. The value of PF_{VMA} is rounded off to two decimal places.

$$PF_{VMA} = 55 + (0.5 \times PWL)$$

B. If PWL for VMA is between 70 and 50 inclusive, the following formula is used to determine PF_{VMA}. The value of PF_{VMA} is rounded off to two decimal places.

$$PF_{VMA} = 37.5 + (0.75 \times PWL)$$

C. If PWL for VMA is less than 50, the Engineer may elect to take one of the actions specified in description for PF_{AV} above.

3.1.2.4 Pay Factor for In-Place Density – PF_D:

A. If PWL for In-Place Density (PWL_D) is between 100 and 70, the following formula is used to determine PF_D. The value of PF_D is rounded off to two decimal places.

$$PF_D = 55 + (0.5 \times PWL)$$

B. If PWL for In-Place Density is between 70 and 50 inclusive, the following formula is used to determine PF_D. The value of PF_D is rounded off to two decimal places.

$$PF_D = 37.5 + (0.75 \times PWL)$$

C. If PWL for In-Place Density is less than 50; the Engineer may elect to take one of the actions specified in description for PF_D above.

3.1.2.5 Overall Lot Pay Factor

$$OLPF = (0.20 \times PF_{BINDER}) + (0.20 \times PF_{VMA}) + (0.60 \times PF_D)$$

3.1.3 Payment

Payment for HMA Pay Items are based on the Contract Prices for the completed items of work as adjusted according to Special Provisions for Hot Mix Asphalt Percent within Limits (PWL). Adjusted Payment for HMA Type is calculated on a Lot-By-Lot basis. The Overall Lot Pay Factor, OLPF, is used to determine the Lot Pay Adjustment as follows:

$$\text{Lot Payment Adjustment} = \frac{(OLPF - 100)\times(\text{Contract Unit Price})\times(\text{Lot Quantity})}{100}$$

Table 3.3 Quality Assurance Testing Tolerance

Quality Characteristic	Initial Production Lot Single Test Tolerance
G_{ms}	± 0.019
G_{mb}	± 0.020
Air Voids	± 1.00 %
VMA	± 1.20 %

3.2 Michigan DOT Pavement Design Process

An effective pavement design is highly dependent upon performing an adequate investigation of the existing pavement structure. Therefore, prior to construction/reconstruction of a pavement some investigations such as:

- Reviewing As-Built Plans
- Reviewing and Analyzing Existing Pavement Distress Condition
- Determining Causes of Pavement Surface Distresses
- Evaluating Pavement Ride Quality
- Reviewing Pavement Remaining Service Life
- Evaluating Drainage System
- Evaluating Subgrade

should be conducted. With no doubt, a comprehensive investigation of the pavement structure not only ensures the Engineer is employing the proper reconstruction or rehabilitation strategies, but also aids the Designer in selection of appropriate input values for pavement design.

M-DOT uses the pavement design methodology recommended by the American Association of State Highway and Transportation Officials, AASHTO:

- 1993 AASHTO Guide for Design of Pavement Structures
- AASHTO Pavement Design Software “DARWin”

The followings summarize “Typical Values” recommended for “Design Inputs”:

- New / Reconstruction Life Design Life ………………… 20
- Accumulated ESAL’s Years …………………………… 20
- Initial Serviceability …………………………………….. 4.5
- Terminal Serviceability …………………………………. 2.5
- Reliability Level ……………………………………….. 95%
- Overall Standard Deviation …………………………….. 0.49

- Structural Coefficient:
  - HMA Top & Leveling Course ………………….... 0.42
  - HMA Base Course ……………………………….. 0.36
  - Rubblized Concrete ………………………………. 0.18
  - Crush & Shaped HMA …………………………… 0.20
  - Aggregate Base …………………………………... 0.14
  - Sand Subbase …………………………………….. 0.10
  - ASCRL & Stabilized Base ……………………….. 0.30

- Elastic Modulus:
  - HMA Top & Leveling Course …………………... 390,000 – 410,000 psi
  - HMA Base Course ………………………………. 275,000 – 320 000 psi
  - Rubblized Concrete …………………………….. 45,000 – 55,000 psi
  - Crush & Shaped HMA ………………………….. 100,000 – 150,000 psi
  - Aggregate Base ………………………………….. 30,000 psi
  - Sand Subbase …………………….......................... 13,500 psi
  - ASCRL & HMA Stabilized Base ……………….. 160,000 psi

- Roadbed Soil Resilient Modulus: Use “Falling Weight Deflectometer” (FWD) data when possible, otherwise a value is chosen based on the predominant subgrade soil type. A correlation can be made between “Soil Type” and “Resilient Modulus”.

- Drainage Coefficient: (Refer to Table 2.4, Page II-25, AASHTO Guide for Design of Pavement Structures)

    - HMA Top & Leveling Course …………………… 1
    - HMA Base Course ……………………………….. 1
    - Rubblized Concrete ………………………………. 1
    - Crush & Shaped HMA …………………………… 1
    - Aggregate Base …………………………………... 1
    - Sand Subbase …………………………………….. 1

- Stage Construction …………………………………...... .1

Chapter 4: HMA Quality Assurance Programs in the United States

A detailed study of the current quality assurance practices of state and federal departments of transportation with regard to highway materials and construction was carried out. The motivation behind this exercise was to assess how the MDOT QA program compares to those being used by other states and agencies and to identify the items that can be considered for possible inclusion into the MDOT QA program. The motivation was not necessarily to exhaustively gather information on all the QA programs.

Different agencies view QA differently. In a nutshell, it can be stated that different states follow different combinations of end-result and materials and methods requirements with varying emphasis on different requirements. More recently there has been a trend to orient QA programs towards ensuring performance rather than checking if the construction methods were followed properly. Therefore, quality characteristics, like volumetric properties for HMA, which affect performance, are tested. It was also observed that most of the states use the same quality characteristics and test methods for quality assurance as for quality control. Only 10 states in the US specify test methods only for QA.

The most common quality characteristics used in QA programs in the US along with the number of agencies using them (NCHRP Synthesis 346) are shown below.

Table 4.1 Most commonly used quality characteristics for QC and QA on HMA pavements

Quality Characteristic	**No. of Agencies**	
	QC	QA
Asphalt content	40	40
Gradation	43	33
Compaction	28	44
Ride quality	16	39
Voids in total mix	20	26
Voids in mineral aggregate	26	23
Aggregate fractured faces	25	23
Thickness	13	22
Voids filled with asphalt	19	13

Table 4.1 and figure 4.1 show that the attributes most often used for the acceptance of HMA pavements are asphalt content, used by 40 agencies; compaction by 44; and ride quality by 39.

Thirty three agencies accept HMA pavements based on gradation and 26 accept voids in total mix. The lesser-used acceptance attributes are aggregate fractured faces, thickness and voids filled with asphalt.

Table 4.2 presents other quality characteristics which are used by some of the agencies although far less often than those mentioned in table 4.1 above.

Table 4.2 Less often used QC/QA quality characteristics for HMA pavements

Other Quality Characteristics	
Retained tensile strength	Joint density
Fine aggregate angularity	Aggregate moisture
Stability	Percent lime
Water sensitivity	Sand equivalence
Wheel tracking test	Flat and elongated particles
Plant mix temperature	Indirect tensile strength
Asphalt temperature	Percent lightweight particles

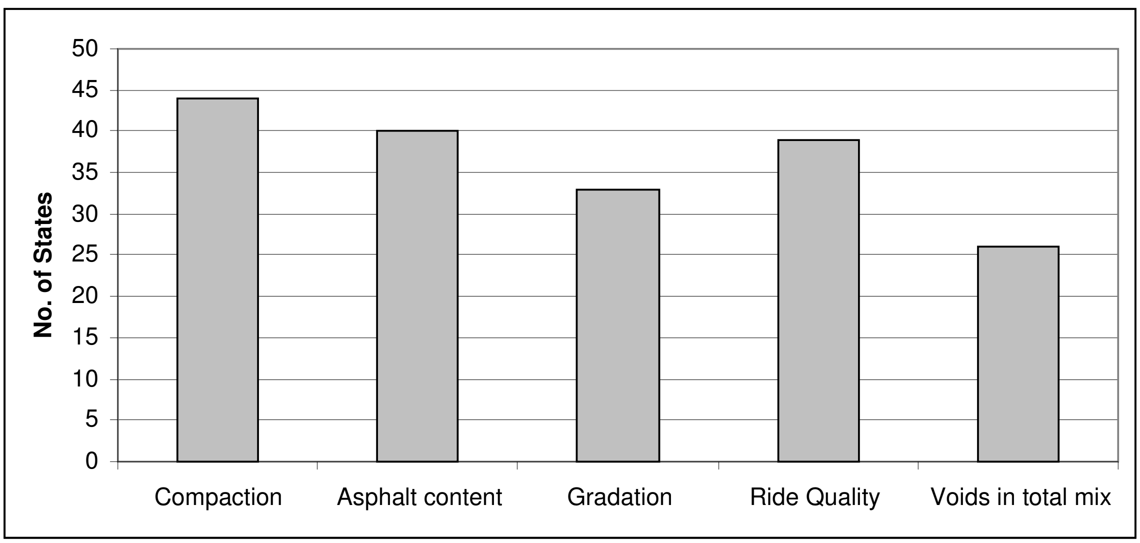


Figure 4.1 Attributes most often used for QA of HMA (40 responses) (adapted from NCHRP Synthesis 346)

4.1 Regional Level Analysis of the QA Practices

Another approach to analyze the similarities and differences in agencies' QA norms is to categorize them on the regional level as has been attempted below.

4.1.1 North- East Region

New Hampshire and Maine use similar testing and sampling methods for gradation-AASHTO T30, while New York employs AASTO T27. For testing the asphalt binder content, numerous sampling and testing procedures are in use. They are AASHTO T 164, NHDOT B-2, NHDOT B-6 (New Hampshire) & NYSDOT MM 5.0 (New York). Michigan uses AASHTO T 164 for the sampling and testing purpose.

The tolerance level for the air void at N_d is 2.5 to 5.5% in Maine while it is 3 to 5% in Michigan. 3 to 5% is more commonly used all across United States for air voids.

We have summarized the QA variables for the north-east states and compared them with Michigan in table 4.3.

4.1.2 Southern Region

The flexible pavement's QA testing and sampling method for aggregate gradation in the state of Arkansas is AASHTO T 30 & T 308, while AASHTO T 168 is used in Alabama and GDT-38 in Georgia. In Arkansas, the frequency at which samples are tested for gradation purpose is 1 per 750 metric tons. In Georgia, two samples are collected after 5 days of production. Table 4.4 presents information on the QA requirements for the southern states.

4.1.3 Mid-West Region

In Wisconsin, the sample for the Asphalt Material Content (AMC) is not collected if its quantity is less than 501 tons, while one sample is collected for quantities ranging between 510 tons - 30,000 tons. In South Dakota, SD 202 procedure is used in flexible pavements in order to carry out the sampling and testing of the mineral aggregate gradation, whereas in Michigan, the department of transportation employs AASHTO T 164. We have provided more information in table 4.5.

The overall lot pay factor for Michigan is = $(0.4 \times PF_D) + (0.3 \times PF_{AV}) + (0.15 \times PF_{BINDER}) + (0.15 \times PF_{VMA})$

Table 4.3 Quality Assurance Practices in North-East States

	Maine	New Hampshire	New York	Michigan
Sample & Test				
Aggregate Gradation	AASHTO T 30	AASHTO T 30	AASHTO T 27	AASHTO T2
Asphalt Binder Content(ABC)	AASHTO T 308	AASTO T 164 NHDOT B-2	NYSDOT MM 5	AASHTO T 164
Air Void	AASHTO T 312	Per QC Plan	AASHTO T 166 AASHTO T 209	AASHTO PP28-97
Sample Location				
Aggregate Gradation	Paver Hooper	Haul Unit		
Asphalt Binder Content	Paver Hooper	Haul Unit		
Air Void	Paver Hooper	Haul Unit		
Frequency				
Aggregate Gradation	1 per 500 Mg (500 ton)			
Air Void	1 per 500 Mg (500 ton)		1 per sub lot	1 per day
Asphalt Binder Content	1 per 500 Mg (500 ton)		4 per day per mix	1 per day
Tolerance				
Asphalt Binder Content	Target ± 0.4%			JMF ± 0.40%
Air Voids at N_d	4.0% ± 1.5%			3% - 5%
In Place Air Voids in total mix		± 2%		0%-8%

Table 4.4 Quality Assurance Practices in Southern States

	Alabama	Arkansas	Georgia	South Carolina	Michigan
Sample & Test					
ABC	AASHTO T 168	AASHTO T308	GDT-125	SC-T-64	AASHTO T 164
Air Void	AASHTO T 168	AASHTO T 269		SC-T-66	AASHTO PP28-97
Material Density	ALDOT 210	AASHTO T 209		SC-T-101	MTM 314
Voids in Mineral Aggregatge (VMA)	ALDOT 210	AHTD 464		SC-T-101	AASHTO PP28-97
Sample Location					
ABC	Loaded Truck		Truck/Road way	Plant	
Air Void	Loaded Truck			Plant	
Material density	Roadway			Road (in place)	
VMA	Loaded Truck			Plant	
Frequency					
ABC	1 per day per Lot		2 samples per 5 days production	1 per Sub lot	1 per day
Air Void	1 per day per Lot		2 samples per 5 days production	1 per Sub lot	1 per day
Material density	1 per 3,000 lane feet/lift			1 per1,500 ft. Sub lot	1 per day
VMA	as needed			1 per Sub lot	1 per day
Tolerance					
AC		± 0.3% for mix design value	± 0.4%	± 0.4%	JMF ± 0.40%
Air Void		3% - 5%			3% - 5%
Material density		Bases, Binder, Surfaces : 92% - 96%			Bases, Binder, Surfaces : 92% - 100%
VMA		ACHM Base, Binder, Surface Course : 11.0%-17%			±1%
Gradation		± 7.0%	± 4.0%		

Table 4.5 Quality Assurance practices in Mid-West States

	South Dakota	**Wisconsin**	**Michigan**
Sample & Test			AASHTO T2
Aggregate Gradation	SD 202		AASHTO T 164
ABC	SD 314		
Maximum Specific Gravity of Asphalt Concrete	SD 312	AASHTO T 209	MTM 314
Bulk Specific Gravity of Asphalt Concrete	SD 313	AASHTO T 166	AASHTO T 168
Air Voids		AASHTO T 269	AASHTO PP28-97
Frequency			
Aggregate Gradation	1 per1000 tons	No test : 0 to 501 tons 1 sample : 501 to 30,000 tons More than 30,000 tons: Add One Test for Each Additional 30,000-ton Increment	
ABC	1 per day		1 per day
Maximum Specific Gravity of Asphalt Concrete	1per 1000 tons		
Bulk Specific Gravity of Asphalt Concrete	1 per 1000 tons		
Air Voids			1 per day
Tolerance			
Aggregate Gradation	± 5.0%	37.5 mm ± 6.0	
ABC		± 0.4%	JMF ± 0.40%
Maximum Specific Gravity of Asphalt Concrete	± 0.020 %		
Bulk Specific Gravity of Asphalt Concrete	± 0.020 %		
Air Voids	± 1.2 %	± 1.3%	3% - 5%

4.1.4 Western Region

In accordance with procedure CP 41, samples are collected for testing the asphalt content in Colorado. One sample is collected for every 500 tons of asphalt. Procedure for asphalt content sampling and testing in Montana is MT-302. Again, 1 sample is collected for 500 tons of asphalt. Colorado and Montana have developed their own procedures for carrying out the QA exercise for the flexible pavements. We have summarized sampling and testing procedures, sample location, tolerance limits for western region states in table 4.6.

4.2 Conclusion

The MDOT HMA QA program is similar to that being used by the states who have adopted end result specifications for their QA program. This similarity exists in (1) the tests that are used for verifying the quality of the constructed pavement, (2) the specifications limits that are used and (3) the quality characteristics, the statistical method and the pay formula used for calculating the payment to be made to the contractor. In other words there is nothing alarmingly different in the QA program being used by MDOT as compared to other “ERS” states.

Table 4.6 Quality Assurance Practices in Western States

	Colorado	Montana	Utah	Michigan
Sample & Test				
Gradation	CP 30 CP 31A CP 31B	MT-202	AASHTO T 30	AASHTO T2
ABC	CP 41 CP 55	MT-302	AASHTO T 308	
VMA	CP 41		AASHTO T 269	AASHTO PP28-97
Maximum Specific Gravity	CP 41 CP 55	MT-212	AASHTO T 166	MTM 314
Sample location				
Gradation	Aggregate from the cold feed, pug mill discharge or extraction	Before Bitumen Is Added to Mix	Grade Behind the Paver	
ABC	Plant discharge, at/or behind paver. For Central Lab Correction Factor, sample from belt and binder from Contractors tank	At the Plant	Grade Behind the Paver	
VMA	Plant Discharge, windrow, at/or behind paver			
Maximum Specific Gravity	Plant Discharge, windrow, at/or behind paver	Randomly After Rolling and Before Opening to Traffic	Grade After Compaction Prior to Traffic	
Frequency				
Gradation	1 per 2000 tons	1 per 600 tons	4 per Lot	
ABC	1 per1000 tons	1 per 500 tons	4 per Lot	1 per day
Maximum Specific Gravity		1 per 600 tons	10:2 in each of 5 equal Sub lots	
Tolerance				
Gradation	3/8": ± 6%		3/4": ± 6% 1/2": ± 6%	
ABC	± 0.3%	5% below minimum requirement	± 0.35%	JMF ± 0.40%
Maximum Specific Gravity			Lower Limit: -2.0% Upper Limit: +3.0%	

Chapter 5: Literature Review - Flexible Pavements

Several research projects were reviewed to verify the significance of candidate QA variables identified in this project for pavement performance. This section presents a brief mention of some of these findings.

5.1 Asphalt Content

Asphalt content primarily controls the voids contents of a mixture. It has been predicted that each 0.4 percent of increase in asphalt content results in about 1.0 to 1.5 percent decrease in in-place air void (Mauplin et al., 2006), which in fact improves durability of the mixture.

The effect of asphalt content is found to be more important during the initial stages of compaction compared to the final stages. Also, it has been found that an increase in asphalt content results in a significant decrease in work or force load required for compaction to 8 % air voids content, (typical for the construction stage in the field). The increase in asphalt content also resulted in a significant decrease, on the order of 2% to 6%, in air voids at the initial number of gyrations, $N_{Initial}$ (Stakston and Bahia, 2003). This confirms that higher asphalt content will result in a mixture less resistance to compaction. Besides, it seems that higher asphalt content will lead to lower resilient modulus and dynamic modulus to all frequencies (Flintsch et al., 2007). This was experimentally proved by 33 samples of 3 different mixtures in Virginia. It was also observed in 25 samples of four different Performance-Grade mixtures that for a 0.5-percent increase in asphalt content, flexural stiffness of mixture peaked (Mauplin et al., 2006). Such an increase did also make an upward trend for fatigue life of the sample, however, an additional 1.0-percent of asphalt was needed for a major beneficial effect (Mauplin et al., 2006).

“Development of Simplified Asphalt Concrete Stiffness-Fatigue Testing Device” (Tran and Hall, 2004) study suggests similar conclusion. That is, mixtures with asphalt content of 0.5 percent below optimum are stiffer than those of 0.5 percent above the optimum.

A study of 84 asphalt mix samples of two aggregate sources in Nevada also experimentally proved that mixes which violated the binder content on the low side performed better than the optimum mix especially in the fatigue or thermal cracking resistance (Sebaaly and Bazi 2004).

The results of laboratory-controlled strain flexural beam testing, that is fatigue life and flexural stiffness, for one aggregate and asphalt cement combination, five asphalt contents, and three air void contents clearly indicate that increased asphalt content increases fatigue life and reduce stiffness (Harvey and Tsal, 1996). It has been also experimented that for relatively thicker pavements, fatigue life increases approximately 10 percent for each 0.5 percent increase in asphalt content. For the thin structure, on the other hand, thinner structures experience approximately 20 percent increase in fatigue life for each 0.5 percent increase in asphalt content (Harvey and Tsal, 1996).

5.2 Air Void Content

Air Voids Content (AVC), which is the amount of voids in a compacted HMA pavement, can have a detrimental and significant effect on performance of the pavement (Harrigan, 2002). However, it has been suggested that this effect is not as significant as conditioning method is on pavement fatigue life (Vivar and Haddock, 2006).

Many studies have shown that the initial in-place voids should be no more than approximately 8% and that the in-place voids should never fall below approximately 3% during the life of the pavement. There is evidence that when in-place air void content drops below 3.090 to 3.5%, the probability of rutting increases drastically (Brown and Cross, 1992). That is, in-place air void content above 3.0% is needed to decrease the probability of premature rutting throughout the life of the pavement (Brown et al., 1991).

Mixes meeting $N_{Initial}$ and N_{Max}, initial and maximum number of gyrations, criteria that have been specified in order to avoid tender mixes and mixes prone to rutting, respectively, do not necessarily show less rutting potential than mixes which do not meet these criteria (Kandhal and Mallick, 1999). An experimental study of mixture of different aggregates, gradation, Nominal Maximum Aggregate Size (NMAS), and asphalt content in Alabama proved that no correlation could be established between APA rut depths and the gyratory compaction slopes (between $N_{Initial}$ and N_{Design}) of all mixes (Kandhal and Mallick, 1999).

Since permeability is directly related to the amount of interconnected voids within the pavement, high air void content leads to permeability to water and air, resulting in water damage, oxidation, raveling, and cracking where as low air void content results in rutting and shoving of the HMA (Brown et al, 2004). In addition, due to consolidation under wheel loading, high AVC can also contribute to the development of rutting in the wheel paths and low AVC, however, increases the likelihood of bleeding, shear flow, and permanent deformation (i.e., rutting) in the wheel paths (Harrigan, 2002).

In general, it has been investigated and the findings suggest that for each percent drop in air void content, there is a corresponding 10 percent loss of pavement life (Linden et al., 1988). This statement was also proven by three other separate sources: existing literature on the effect of air void content on pavement performance, a questionnaire survey of 48 state highway agencies on compaction practice, and performance data from the Washington State Pavement Management System emerge that, as a rule-of-thumb, for each 1 percent increase in air voids (over a base air void level of 7 percent), there is about a 10 percent (approximately one year) loss in pavement life (Linden et al., 1988). As it appears, more air voids will lead to lower resilient modulus and lower dynamic modulus (Flintsch et al., 2007). Monitoring 33 samples of three different mixtures in Virginia proves the foregoing statement.

It has been observed from a study of 60 samples of 5 asphalt pavement sites for rutting, that low voids (in re-compacted samples and/or field samples) are the cause of most rutting in the pavements (Brown et al., 1989). It is possible the relationship between rutting and air voids is affected by an increase in air voids once the pavement begins to rut and shove. Also, there is a good possibility that the void level decreases under compaction to some point at which rutting begins to occur and at which time the void level begins to increase due to shoving of the mixture (Brown et al., 1989).

The in-place rutting study of asphalt pavements clearly shows that very little rutting occurs when the re-compacted air voids are 3.0 percent or higher for compactive efforts of 75 blow Marshall and Gyratory with 120 psi, 1 degree and 300 revolutions. Three of the four mixes with more than 3 percent air voids have no rutting while the other mix has only 10 percent rutting. Significant rutting occurs in those layers having less than 3.0 percent re-compacted air voids. In this case three of the eight pavement layers have more than 20 percent rutting which is significant (Brown et al., 1989).

Mallick et. al. (2003) determined that there is a significant effect of air void content (as measured by voids in total mix) of dense graded HMA on in-place permeability of pavements and of NMAS on permeability of coarse-graded Superpave designed mixes. They recommended that State DOTs consider designing mixes as less permeable than coarse graded mixes at similar void levels i.e. placed 100 mm below the pavement surface on the fine side of the maximum density line, and thereby less susceptible to allowing moisture or moisture vapor to propagate upward through the pavement structure. This in turn would reduce the possibility of moisture damage within such structures.

A Wisconsin DOT study (WisDOT, 2004) estimates and measures permeability during mixture design and defines a relationship between lift thickness and aggregate gradations that minimizes the densification problem and addresses the permeability concerns. They state that it is well recognized that field density is significantly affected by maximum aggregate size of aggregates, gradation, and lift thickness and that permeability of asphalt mixtures is a function of aggregate gradation, density achieved, and distribution of air voids. With a shift in mixture designs to Superpave methods, gradations, which are unique in their densification characteristics and claimed to be more permeable, on the coarse side of the maximum density line are being widely used as well as recommended. This trend, according to them, might be due to changes in the air voids distribution, the lower densities being achieved, or both. Permeability is also a directional property such that orientation of the aggregates, which is affected by lift thickness and level of compaction, has a significant effect on total permeability.

5.3 Mixture Density

One of the most important parameters in construction of asphalt mixtures is density. It must be closely controlled to insure that the voids stay within an acceptable range. However, effect of mixture density on performance of mixture is dependent upon the mixture gradation and aggregate size, and NMAS (Vivar and Haddock, 2006).

The Asphalt Institute recommends that the mix design density should closely approach the maximum density obtained in the pavement under traffic. A review of several state DOT specifications has shown that in-place density, measured as a percent of maximum theoretical density, ranges between 91% and 98% (with many falling between 92% and 97%), supporting the previous statement (Brown et al., 2004). HMA mixtures of lower density tend to have higher permeability, lower dynamic modulus, lower flexural stiffness, and shorter fatigue life (Vivar and Haddock, 2006).

Much of the loss in pavement life is a direct result of low density. Study of 36 mixture samples in lab and field surveys have both concluded that inadequate density is a significant problem on a high percentage of paving projects (Brown et al., 1991).

A mixture that is properly designed and compacted will contain enough air voids to prevent rutting due to plastic flow but low enough air voids to prevent permeability of air and water. Since density of an asphalt mixture varies throughout its life, the voids must be low enough initially to prevent permeability of air and water and high enough after a few years of traffic to prevent plastic flow.

In general, it has been shown that the rut potential is significantly reduced with increases in HMA density, which is the reduction in air voids content (Vivar and Haddock, 2006). Insufficient compaction during mix design and mixture testing results in a higher required asphalt content to obtain the specified void level. For that matter, compaction during construction and under traffic loads results in a density which is higher than laboratory density and consequently lower voids than measured during mix design. Thus, compaction during mix design and field quality control has to produce a density in the laboratory equal to what will be obtained in the field after a few years of traffic (Brown et al., 1989).

5.4 Aggregate Size and Gradation

It is well known that the density which could be achieved in the field and its effects on HMA mixture performance (Vivar and Haddock, 2006) are significantly affected by aggregate size and gradation. Also well recognized is that permeability of asphalt mixture is a function of aggregate gradation. It has been shown that for any given in-place air void content, permeability of HMA mixture increases by one order of magnitude as the maximum aggregate size increases (Mallick et al., 2003).

Aggregate size, shape, and gradation influence the size of voids within a pavement structure. Coarse graded mixes require more asphalt content (Haddock et al., 1991). Such mixtures contain a relatively higher percentage of coarse aggregate than fine graded mixes. This higher percentage of coarse aggregate leads to larger voids within the mix matrix. The combination of larger voids and fewer fine aggregates to fill the voids likely result in more interconnected voids which make the mix more permeable. In addition, coarse graded mixtures tend to have lower dynamic modulus and flexural stiffness in comparison with fine graded mixtures but higher permeability and longer fatigue life (Vivar and Haddock, 2006).

Increasing the size of the largest aggregate in a gradation will increase the mix quality with respect to creep performance, resilient modulus, and also tensile strength. However, there will not be any significant effect on Marshall Stability (Brown and Bassett, 1990). For instance, monitoring 84 asphalt mixture samples of two aggregate sources in Nevada proved that mixtures which were low on the # 4 sieve and high on the # 200 sieve never achieved a performance that is equivalent to or better than the optimum mixture. In fact, mixtures of high percent passing # 200 were always worse than optimum mixture unless higher binder content was introduced (Sebaaly and Bazi, 2004). It can be said, thus, that mixes with larger maximum aggregate size are stiffer and will reduce stresses in the underlying layers (Brown and Bassett, 1990). This conclusion has been experimentally proven in Arkansas by 36 mixture samples of two aggregate size and four different asphalt content that large aggregate size of 25 mm has higher dynamic modulus than small aggregate size of 12.5 mm (Tran and Hall, 2004).

In addition, a study of 4 HMA mixtures with different air void content, aggregate size and gradation showed that mixtures with a 19.0-mm NMAS tend to have higher dynamic modulus and flexural stiffness, higher permeability and moisture damage, and lower fatigue life than mixtures with a 9.5-mm NMAS (Vivar and Haddock, 2006).

It is also true in the case of coarse-graded against fine-graded mixes. Coarse-graded mixtures tend to have lower dynamic modulus and flexural stiffness, but higher permeability and fatigue life compared to fine-graded mixes (Vivar and Haddock, 2006). In the same study, at constant 96 percent density, a coarse-graded mixture with 9.5-mm NMAS showed the best fatigue life where fine-graded mixtures of higher dynamic modulus with 19.0-mm NMAS manifested less of rut performance (Vivar and Haddock, 2006).

Studies on effects of aggregate angularity (Stakston and Bahia, 2003) show that the influence of aggregate gradation in predicting mixture performance, such as control of voids, is highly specific to the angularity of aggregate and its source.

“Evaluation of Asphalt Pavement Analyzer for HMA Mix Design” (Kandhal and Mallick, 1999) states that the effect of gradation on granite and limestone wearing and binder courses with PG 64-22 asphalt is significant, with below restricted zone gradation showing higher rutting compared to above and through restricted zone. This experiment studied mixture samples of different aggregates type, gradation, nominal maximum size aggregates and binder in Alabama and proved that effect of gradation is similar and significant for granite PG 58-22 wearing courses but not significant for granite binder course. However, such an effect it is not significant for rutting of gravel wearing and binder course mixes with PG 64-22. The above and through restricted zone mixes showed slightly higher rutting compared to below zone mixes.

5.5 Pavement Thickness

In general, the thicker the asphalt layer, the stiffer it is. This fact has been studied in Arkansas where 25-mm thick HMA layer samples presented higher dynamic modulus than samples of 12.5-mm HMA thickness (Tran and Hall, 2004). However, roles of aggregate size and binder content on this effect should not be overlooked.

Investigations of 18 national projects along with Specific Pavement Studies in LTPP database (Von Quintus and Simpson, 2003) have clearly concluded that greater amounts of fatigue cracking occur on pavement structures of thinner HMA layers. Chatti et al. (2005) confirmed that pavements with “thin” (102 mm = 4 in.) HMA surface layer have shown more fatigue cracking, slightly more rutting, and higher changes in IRI than those with “thick” (178 mm = 7 in.) HMA surface layer.

Selezneva et. al. (2002) studied the quality and completeness of pavement layering information and layer thickness data for LTPP sites. They evaluated the consistency of material type and thickness data between different data sources and also layer thickness variability indicators, within-section material type consistency, and material type and thickness reasonableness. They also analyzed experiments to determine characteristics of within-section layer thickness variation, including layers with different material and functional types and computed descriptive statistics such as mean, standard deviation, skewness, and kurtosis for each section.

5.6 Binder Performance Grade

The asphalt binder affects various performance aspects of the asphalt mixtures such as permanent deformation, fatigue cracking, and low temperature cracking. The Performance Grade, PG, binder specification is intended to select the binder to optimize its effect on the performance of the pavement.

The increase in the performance grade resulted in a marginal increase in air voids, on the order of 2% to 6%, at the initial number of gyrations, Ninit, (Stakston and Bahia, 2003). This confirms that higher performance grade asphalt could result in a mixture which is more resistant to compaction (Stakston and Bahia, 2003). Also, in Alabama it was observed from asphalt mixtures with different aggregates, gradation, nominal maximum aggregate size, and binder, that rut depths of mixes with PG 58-22 asphalt binder (tested at 58 °C) were higher than those of mixes with PG 64-22 asphalt binder (tested at 64 °C).

5.7 Review of different Non-Destructive tests

A detailed review of non-destructive tests is provided in Appendix B. The following sections give a brief overview of some of the relevant tests for flexible pavements.

5.7.1 Thickness

Pavement layer thickness is an important factor in determining the quality of newly constructed pavements and overlays because deficiencies in thickness reduce the life of the pavement. In order to use pavement thickness as a measure of quality assurance, it is necessary to have an accurate and reliable method for making the thickness measurement. Cores are accurate, but they are time consuming, they damage the pavement, and they represent a very small sample of the actual pavement. Therefore, it is desirable to have a thickness measuring method which is quick, non-destructive, and which can generate an accurate and representative population of pavement thickness data points. Some of the non destructive test methods available for thickness measurements have been listed here with their key features from the point of view of their suitability of use in a quality assurance (QA) program.

5.7.1.1 Ground-penetrating radar (GPR)

Ground-penetrating radar (GPR) is a high resolution geophysical technique that utilizes electromagnetic radar waves to scan shallow subsurface objects. It can provide information on pavement layer thickness or locate targets (Daniels,1990; Hasted, 1973; Ulriksen, 1982, Harris, 1998). Frequency of the GPR antenna affects depth of penetration. Lower frequency antennas penetrate further, but higher frequency antennas yield higher resolution. To successfully provide pavement thickness information or scan an interface, the following conditions have to be present:

- Physical properties of the pavement layers must allow for penetration of the radar wave.
- Interface between pavement layers must reflect the radar wave with sufficient energy to be recorded.
- Difference in physical properties between layers separated by interfaces must be significant.

Physical (electrical) properties of pavement layers, thickness of pavement layers, and magnitude of difference between electrical properties of successive pavement layers impact the ability to detect thickness information using GPR. Conductive losses occur when electromagnetic energy is transformed into thermal energy to provide for transport of charge carriers through a specific medium. Presence of moisture or clay content in a pavement layer will cause significant conductive losses and hence will increase the dielectric permittivity and decrease the depth of penetration.

For asphalt pavements, GPR is by far the most established technology for measuring pavement thickness. Evaluation studies have been carried out by over ten state highway agencies, by SHRP, MnROAD, and by the FHWA, all of which have documented the accuracy of GPR asphalt thickness vs. core samples (Maser, 1999; Wenzlick and Maser, 1999). The studies have generally compared the GPR results to cores, and have shown differences that range from 2 to 10%. The lower differences (2-5%) are generally associated with newly constructed pavements, while the bigger differences are generally associated with older pavements (Infrasense, 2003). In general, where there are large deviations between GPR and core values, the GPR gave the larger values, and the difference appeared to be due to portions of the core that remained in the hole (Infrasense, 2006). Studies have also shown that with proper equipment and data processing, GPR can accurately determine thickness for overlays as thin as 25 mm (1 inch) (Maser and Scullion, 1992).

5.7.1.1.1 Horn Antenna GPR

The air-launched antenna is routinely used at highway speeds and is not physically affected by rough road conditions. Most importantly, it is not necessary to obtain cores to calibrate the air-launched horn antenna system. Another important advantage of the horn antenna is the ability to measure thin pavement layers.

Advantages:

- Only highway-speed subsurface pavement testing tool
- Excellent for flexible pavement rehabilitation projects
- Can be merged with surface video and other NDT data

Limitations (Wimsatt et al., 2008):

- Depth limited to top 20 – 24 inches
- Attenuation problems with concrete layers
- Initially limited software available for processing data

Barriers to Implementation:

- FCC restrictions on manufacturers in the US
- Oversold - initial results disappointing
- No certification of equipment and vendors

5.7.1.1.2 Ground-coupled GPR

As the name suggests, a ground-coupled antenna needs to remain in contact with the ground (or suspended very slightly above the ground) to properly couple the electromagnetic energy to and from the antenna. To calibrate the ground-coupled system it is necessary to obtain cores from the pavement and physically measure the actual pavement thickness.

Advantages:

- Fairly inexpensive
- Robust equipment – technology and software widely available
- Deep investigations possible with low frequency equipment

Limitations (Wimsatt et al., 2008):

- Speed typically less than 10 mph
- Limited near surface information
- Penetration limited in clay material
- Qualitative information requiring an expert for interpretation

Barriers to implementation:

- Technology not well understood by DOT's
- No other significant barriers

5.7.1.1.3 GPR applications

Air coupled GPR:

- Thickness of pavement layers
- Moisture or density related defects in HMA and base layers

- Density of new HMA layers
- Delaminations in bridge decks (with HMA surfaces)
- Section uniformity

Ground coupled GPR:

- Detecting buried objects
- Voids under thick concrete slabs
- Detecting steel presence and depth
- Locations where deep investigations are required

It is capable of detecting a number of parameters in reinforced concrete structures:

- location of reinforcement
- depth of cover
- location of voids
- location of cracks
- in situ density
- moisture content variations

User expertise

Users must have good knowledge of wave propagation behavior in materials in order to meaningfully collect and interpret results. Training and experience are required.

5.7.1.1.4 Advantages and Limitations of GPR

- It can be used to survey large areas rapidly for locating reinforcement, voids and cracks. GPR can be collected continuously at various speeds, and thus allowing for the availability of a large number of thickness data points to be collected economically.
- Results must be correlated to test results on samples obtained. Any features screened by steel reinforcement will not be recorded.
- GPR has also been effectively used to determine variations in asphalt density (Saarenketo and Roimela, 1998).
- With increasing depth, low level signals from small targets are harder to detect due to signal attenuation.
- It is expensive to use and uneconomical for surveying small areas.
- GPR technology lacks the ability to differentiate between the AC layers and layers of asphalt-treated materials in thickness estimation (Hanna, 2002).
- Most of these GPR layer thickness studies have been carried out with air-coupled horn antennas since these can be implemented at driving speed without lane closures. However, for the purposes of quality assurance, lower data collection speeds permit consideration of ground-coupled antennas as well.

5.7.1.1.5 Accuracy and Interpretation of GPR

There are a number of factors to be taken into account when interpreting radar data and signals:

- Hyperbolic shapes typically represent a point reflector
- The diameter of cylindrical objects ranging from rebars to metallic oil drums cannot be determined from radargrams
- Radar wave velocity reduces when travelling through wet concrete
- Radar waves are more rapidly attenuated when travelling through wet concrete
- Radar waves cannot penetrate conductors such as metals, clays, salt water
- Radar antennas cannot identify objects in the near field which are closer to the surface than $\lambda/3$, where λ is the wave length (IAEA, 2002)

5.7.1.2 Summary

The methods described in this section are summarized in Table 5.1 below.

Table 5. 1 NDT methods for measuring thickness

Method	Technology	Application	Measurement Type	Measurement Rate	Prior Experience
Horn antenna	Non-Contact GPR (electromagnetic)	asphalt	continuous	up to 9 m/sec (30 feet/sec)	extensive
Calibrated Single Antenna	Ground-Coupled GPR (electromagnetic)	asphalt or concrete	continuous	up to 1.5 m/sec (5 feet/sec)	none documented
Dual Antenna CMP	Ground-Coupled GPR (electromagnetic)	asphalt or concrete	Point	estimated 2 min./point	limited for pavement

The summary table distinguishes between continuous and “point” methods. The continuous methods can collect data while the equipment is moved continuously along the pavement. The "point" methods must be set up to make a measurement at a particular point. An estimated rate of data collection has been indicated. Note that some of the methods are well established, while others are relatively new for this application.

5.7.1.3 Conclusion

The GPR system is capable of estimating the layer thicknesses accurately, especially for HMA layers. It should be emphasized that the accuracy of the GPR system may be significantly affected when noise is present in the data due to external interferences. According to Holzschuhe et al., (2007), the repeatability of the GPR system is excellent for speeds ranging from less than 15 mph up to 70 mph. They also reported that the thickness predictions are very reliable at highway speeds. However, it is strongly recommended that when the data is collected at highway speeds, more markers be inserted in the GPR data in order to minimize the offset errors. These markers should be linked to physical objects with known mileposts. It is strongly recommended that the GPR system be used as a tool for assisting in pavement thickness determination. More accurate thickness information can be obtained when the core thicknesses are used as feedback into the GPR analysis for calibration of radar velocities (Holzschuhe et al., 2007).

5.7.2 Moisture and Density (Radioisotope Gauges)

Moisture gauges consist of a source of neutron radiation, which irradiates the material under test. As a result of radiation, gamma rays are created and detected. The result is a series of counts, which are a measure of the composition of the material. It can be used to measure moisture content of concrete, soil and bituminous materials and to map moisture migration patterns in masonry walls.

5.7.2.1 Advantages:

- Instrument is portable
- Moisture measurements can be made rapidly

5.7.2.2 Limitations:

- A minimum thickness of surface layer is required for backscatter to be measured,
- It measures only the moisture content of surface layer (50 mm),
- It emits radiation,
- Results are inaccurate because hydrogen atoms of building materials are measured in addition to those of water,
- Its use in concrete is limited and requires calibration in order to calculate density or moisture content

5.7.3 Modulus of Pavement Layers

Making accurate assessments of the structural condition of roads during construction helps tremendously in locating weak areas prone to localized failure and correcting them prior to completion of the pavement.

5.7.3.1 Humboldt Stiffness Gauge

The Humboldt Stiffness Gauge (HSG) provides a simple, quick and accurate means of directly measuring stiffness of the upper lift of unbound material. The HSG measures impedance at the soil surface by generating vibrations at 100 and 200 Hz that impart a very small change in the applied load (Humboldt, 1999). The stiffness of the pavement material in resisting this load is determined at each frequency and the average is displayed on the Stiffness Gauge display window. The entire process takes about one minute. The HGS weighs about 10 kg, is 28 cm in diameter, 25.4 cm tall and rests on the soil surface via a ring-shaped foot. The advantage of the HSG is that it is lightweight and can be used at many locations to assess variability in individual compacted lifts of unbound material. The disadvantage of the HSG is that its depth of penetration is limited to about six inches.

5.7.3.2 Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is a nondestructive testing device widely used for assessing the structural condition of a pavement. When complete deflection basins are available, deflection testing can provide key properties for the existing pavement structure through backcalculation of the measured pavement responses, Specifically, for hot-mix asphalt (HMA) pavements, the elastic modulus (E) of the individual paving layers can be determined, along with the resilient modulus (MR) of the subgrade. Deflection testing has also seen some limited use as a means of monitoring the quality of a pavement during construction.

Portable light weight FWD has been developed and used in Europe and has gained the interest of many DOTs. Its applications are:

- Rapid stiffness testing of bases and subgrades but discrete measurement of bearing capacity of granular layers.
- Alternative to Nuclear density gauges

5.7.3.2.1 Advantages and Disadvantages of FWD

Advantages:

Pavement deflection testing provides some distinct advantages over destructive testing, including the following (Hudson et al. 1987):

- More rapid testing operation.
- Relative ease of operation.
- Lower operating cost.

- Less manpower requirements.
- Less intrusive procedure.
- Increased number of test points.

Specific advantages of the FWD are (NHI, 1994):

- Realistic simulation of actual wheel loading.
- High productivity.
- Ability to measure deflection basin.
- Ability to measure joint/crack load transfer.

Disadvantages:

- High initial cost.
- The need for traffic control.
- Relatively complex electro-mechanical system.

Specific issues in backcalculation of layer moduli include:

- Depth of influence unknown
- Number of layers for backcalculation is typically limited to three
- Layer thicknesses must be known
- Difficult to distinguish between layers of similar stiffness

5.7.3.3 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) is a quick, simple, automated field test method for evaluating the in-situ stiffness of existing highway pavements. The greatest advantage associated with the DCP is its ability to penetrate into underlying layers and accurately locate zones of weakness within the pavement structure. It measures the strength and stiffness of unstabilized base and subgrade layers. The unit has software for storing DCP data. The DCP drives the penetrometer rod into the ground using constant energy for each blow, and the penetration index determined with the DCP is calculated as a running depth of penetration per blow.

5.7.3.3.1 Applications

- Quality assurance testing of subgrade and embankment materials
- Alternative to Nuclear density gauges

5.7.3.3.2 Advantages and limitations

Advantages (Wimsatt, 2008):

- Cheap/portable/simple
- Related to CBR and stiffness

Limitations:

- Slow, labor intensive
- Point specific
- Problems with granular materials
- Rod friction should be accounted for in clays

Barriers to implementations:

- No specifications
- Influence of layer moisture content

5.7.3.4 Seismic Pavement Analyzer

The Seismic Pavement Analyzer (SPA) lowers transducers and sources to the pavement and digitally records surface deformations induced by a large pneumatic hammer which generates low-frequency vibrations, and a small pneumatic hammer which generates high-frequency vibrations. The SPA differs from the FWD in that more and higher frequency transducers are used, and more sophisticated interpretation techniques are applied. All measurements are spot measurements; that is, the device has to be towed and situated at a specific point before measurements can be made. A complete testing cycle at one point takes less than one minute. A summary of pavement properties estimated by the SPA is provided in table 5.2.

Table 5. 2 Pavement properties estimated by the Seismic Pavement Analyzer

Pavement component	Parameter measured				
	Young's Modulus	Shear Modulus	Thickness	Damping	Other
Paving layer	yes	yes	yes	no	temperature
Base	yes	yes	yes*	no	
Subgrade	no	yes	no	yes	

*Thickness estimate of base depends on shear modulus contrast with subgrade.

5.7.3.4.1 Advantages and Limitations of SPA

Advantages (Wimsatt et al., 2008):

- Reduces number of destructive tests required for determining pavement layer properties
- Results can be obtained within two minutes, since the data is analyzed on site

Limitations (Wimsatt et al., 2008):

- The testing is discrete by nature (i.e. the testing measures properties at a single point per test, and it takes two minutes per test)
- Not suitable for rapid 100% coverage testing
- Unsuitable for testing composite pavements (Hanna, 2002)
- Unproven equipment reliability
- Need for high skills relevant to data reduction and analysis

5.7.3.5 Conclusion for Modulus of Pavement Layers

It is difficult to directly compare results of the various deflection testing equipment because they measure to different depths, they utilize different technologies to induce load and measure in-situ response, and different equations are used to convert surface deformation to layer modulus, particularly on two layered pavement structures. Data obtained in a study indicate strongly that the devices do give similar magnitudes of stiffness and modulus, and similar trends in the data with regard to relative stiffness of the in-situ layers (Hanna, 2002).

The Humboldt Stiffness Gauge is an effective tool for monitoring the integrity of individual material lifts as they are constructed, since the measurements are limited to that lift. Conversely, the FWD is effective in measuring the total composite stiffness of in-situ pavement structures. The FWD has a definite advantage over the Plate Load Test in being faster, less labor intensive and able to provide much better coverage within a given period of time. If specific areas of the pavement are identified with the FWD as having unusually low stiffness, the Dynamic Cone Penetrometer can be used to identify the cause(s) of low stiffness and locate specific layers within the structure which will likely cause premature distress. Engineers can then assess the cost and benefits of correcting the problem early to extend the service life of the pavement, and avoid higher maintenance costs and public inconvenience later.

5.7.4 HMA Temperature

Temperature measurement of the HMA mat during construction using infra-red cameras is very useful to investigate temperature uniformity of new HMA layers, detect thermal segregation, create a permanent log of paving operations, and locate and establish duration of paver stops.

Advantages

- Segregation of hot mix a continuing problem
- Newer lower cost camera systems widely available

- Automated system with 100% coverage
- Cameras and guns available

Limitations

- Equipment not widely available

Barriers to Implementation

- Unknown targets given the variability of PG gradations and mix types
- Not currently included in specifications

Chapter 6: Empirical Data Analysis – Flexible Pavements

As stated earlier, data from Michigan projects were rather scarce. Therefore, alternative sources of data needed to be explored to determine how quality characteristics used in QA programs affect pavement performance. A preliminary analysis was first performed to study the relationship of acceptance parameters like air voids (or density) and asphalt content to performance parameters like rutting and fatigue using data from Long Term Pavement Performance (LTPP) projects. In this analysis, data from several states was used. These states geographically lie in different climatic zones. The LTPP database contains performance data (rut depth, fatigue cracking, longitudinal cracking, transverse cracking, IRI etc.) and design and construction data (including physical inventory data, material properties from in-situ and laboratory tests). For the preliminary analysis all the data were derived from the Specific Pavement Studies – 1 (SPS -1) experiment. This analysis was followed by alternative analysis with data from General Pavement Studies (GPS) experiments.

Table 6.1 lists categories of data that were extracted from the LTPP database. The data comes from multiple states. There were very few data points available for the state of Michigan.

Table 6. 1 Categories of data extracted from LTPP database.

Construction number	Asphalt Content
Traffic opening date	Mean
Maximum specific gravity	Minimum
Voids in mineral aggregate	Maximum
Effective asphalt content	Standard deviation
	Number of samples
Bulk specific gravity	% Air voids (in-situ)
Mean	Mean
Minimum	Minimum
Maximum	Maximum
Standard deviation	Standard deviation
Number of samples	Number of samples

Different regions in the United States are broadly divided into four climatic zones: (a) Wet freeze (b) Wet non-freeze (c) Dry freeze and (d) Dry non-freeze. Michigan falls in the wet freeze zone. Therefore, data corresponding to all sections in the wet freeze zone were separated for analysis. Other states falling in the wet freeze zone, for which data was available and was extracted from the LTPP database, are Delaware, Iowa and Virginia. However, it was found that these four states have relatively little data available. Therefore, we decided to include states from other climatic zones in the analysis. It is important to mention that different projects from these states have partial results. Figure 6.1 and 6.2 show the number of projects for each of the states which have fatigue and rutting data respectively. The state code for Michigan is 26.

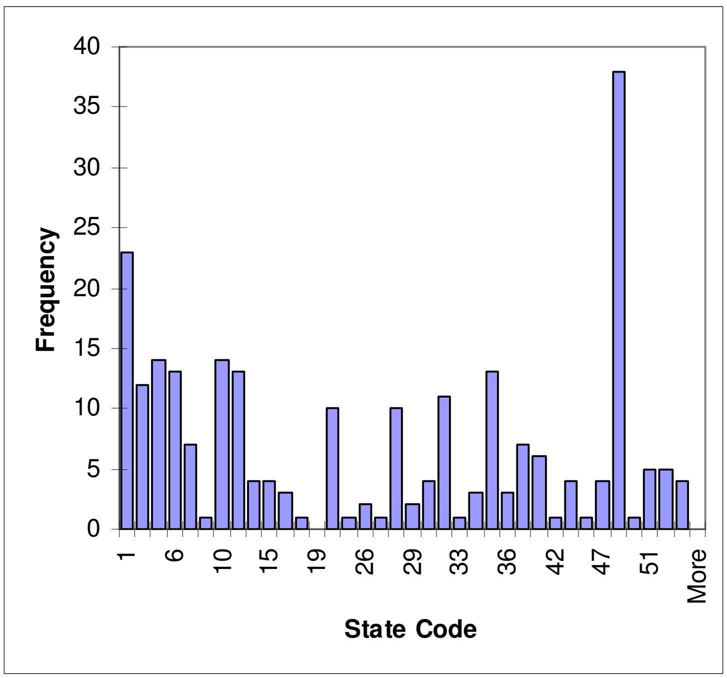


Figure 6.1 Number of projects from different states which have fatigue data.

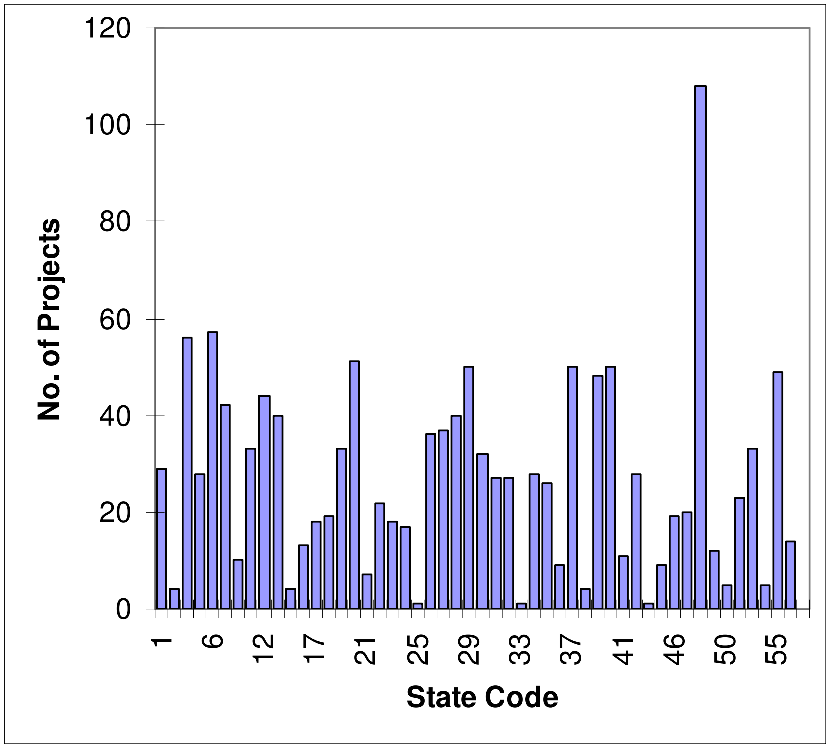


Figure 6.2 Number of projects from different states which have rutting data

Since different projects have varying age, the length of period for which performance data is available varies from project to project. Figure 6.3 and 6.4 show the distribution of all the projects with respect to number of years of performance data available. To be able to objectively compare their performances, the area under the performance curve was normalized by the length of period for which performance data is available.

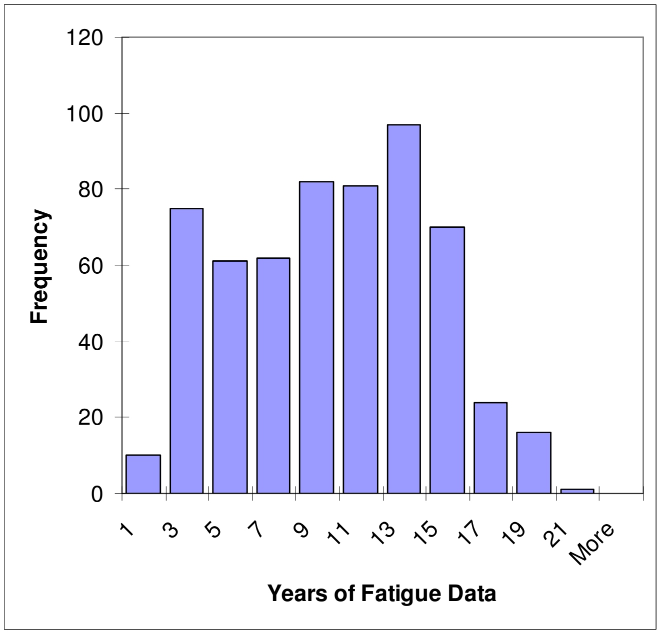


Figure 6.3 Distribution of projects based on years of fatigue data available

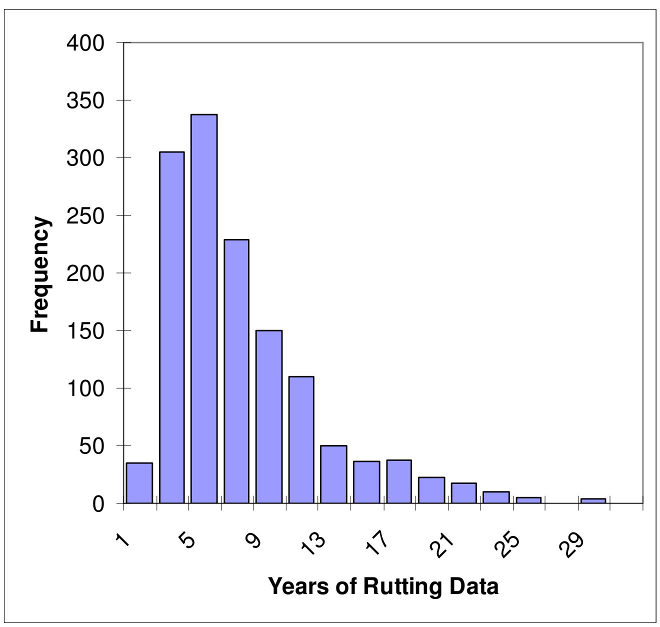


Figure 6.4 Distribution of projects based on years of rutting data available

6.1 Preliminary Analysis

6.1.1 Rutting

Figure 6.5 shows the plot of pavement rutting verses percent in-situ air voids immediately after construction. The purpose of this study was to see if certain levels of air voids affect rutting performance. It should be noted that a trend cannot be derived from such a plot because many other parameters were different between projects from which data has been used. However, it can be clearly seen that pavements with low (< 4%) or high (>8%) in-situ air voids have higher probability of rutting than those in which air voids fall in the range of 4 to 8%. This relationship is significant because it is a clear indication that in-situ air voids (or density) immediately after construction is an important parameter that needs to be controlled to ensure good rutting performance.

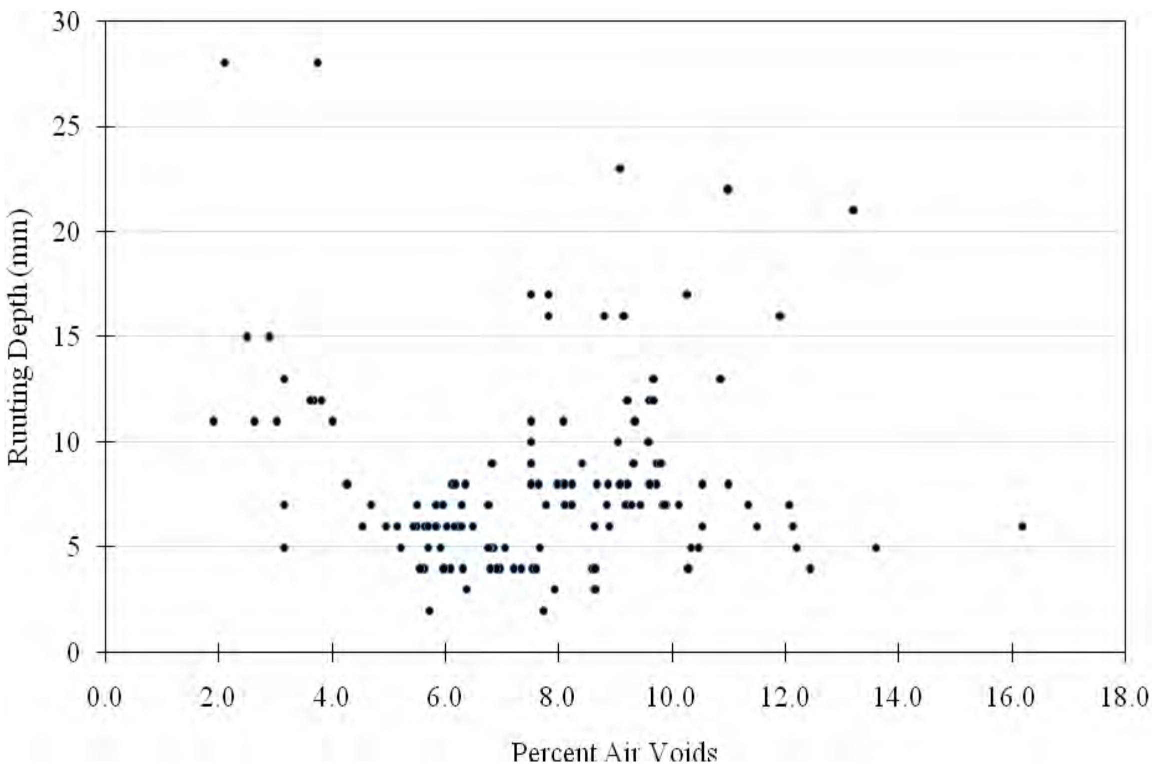


Figure 6.5 Rutting vs. percent air voids for all SPS-1 sections in LTPP database

Figure 6.6 shows a plot of rutting vs percent asphalt content. This scatter plot does not seem to show any particular relationship between them. However, intuitively it would be expected that very high values of asphalt content should lead to a richer mix which would rut more. A possible explanation for this apparent anomaly is presented below.

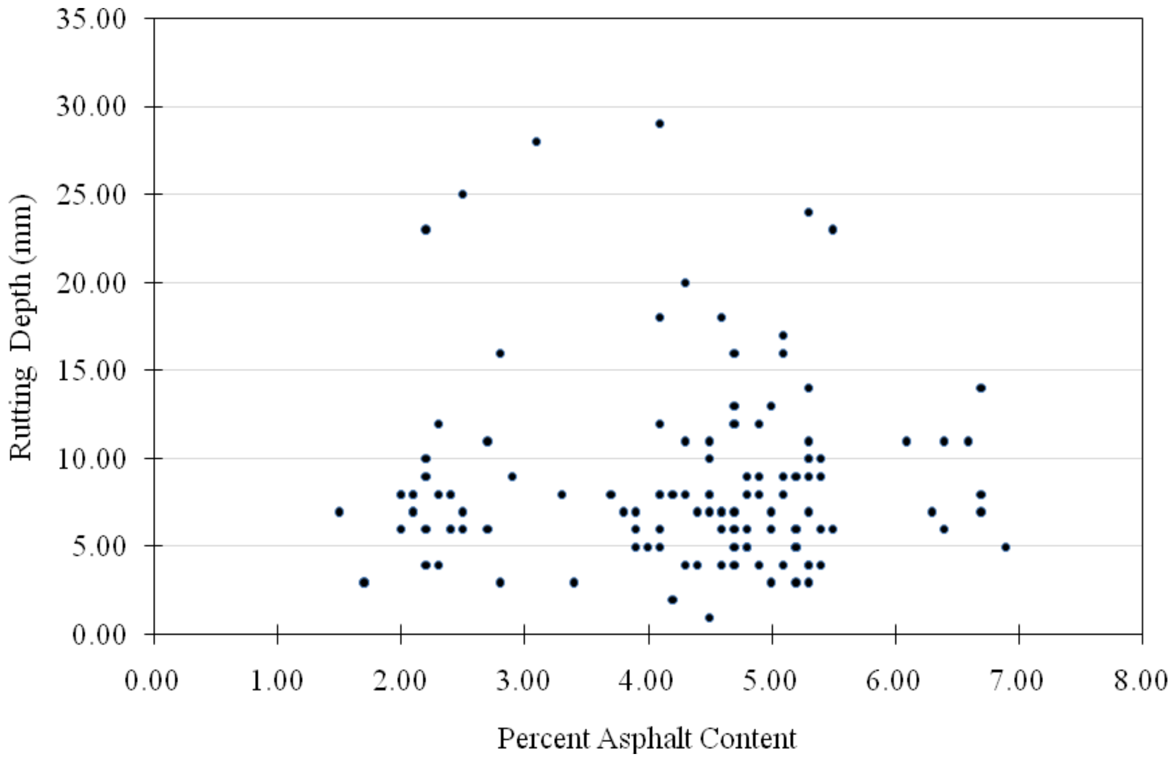


Figure 6.6 Rutting vs. asphalt content for all SPS-1 sections in LTPP database

From field experience, it is known that it is much easier to precisely control asphalt content in a mix than certain other parameters like air voids. SPS-1 was an experiment performed under strict quality controls. Therefore, most of the mixes have optimum quantity of asphalt as estimated during mix design. One may conclude that from this set of data asphalt content does not seem to be affecting rutting. However, a better conculsion would be that since there are very few projects with exceptionally high asphalt content, a relationship between asphalt content and rutting can not be established from such data.

To better understand the relationship between asphalt content and rutting projects with similar aggregate gradation and pavement structure but with varying asphalt content would be required.

6.1.2 Fatigue

It would be expected that generally higher air voids would lead to greater amount of fatigue on HMA pavements. Figure 6.7 shows this trend to some extent. Historically, it is also known that lean HMA mixes (i.e., HMA with low asphalt content) fatigue prematurely. This trend is not visible in Figure 6.8. However, there is some indication that higher asphalt content may contribute to poor fatigue performance. This needs to be verified with data from a more controlled experiment. The data plotted in either of the cases shown here come from the SPS-1 experiment of LTPP in which several other factors were also varied.

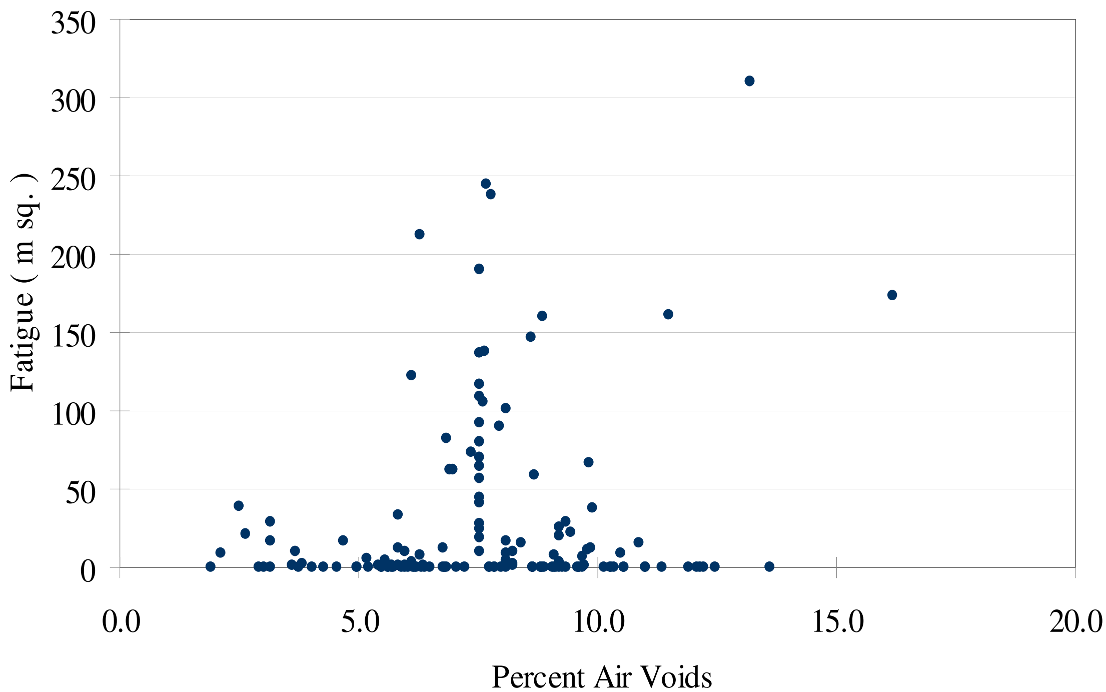


Figure 6.7 fatigue vs. in-situ air voids content for all SPS-1 sections in LTPP database

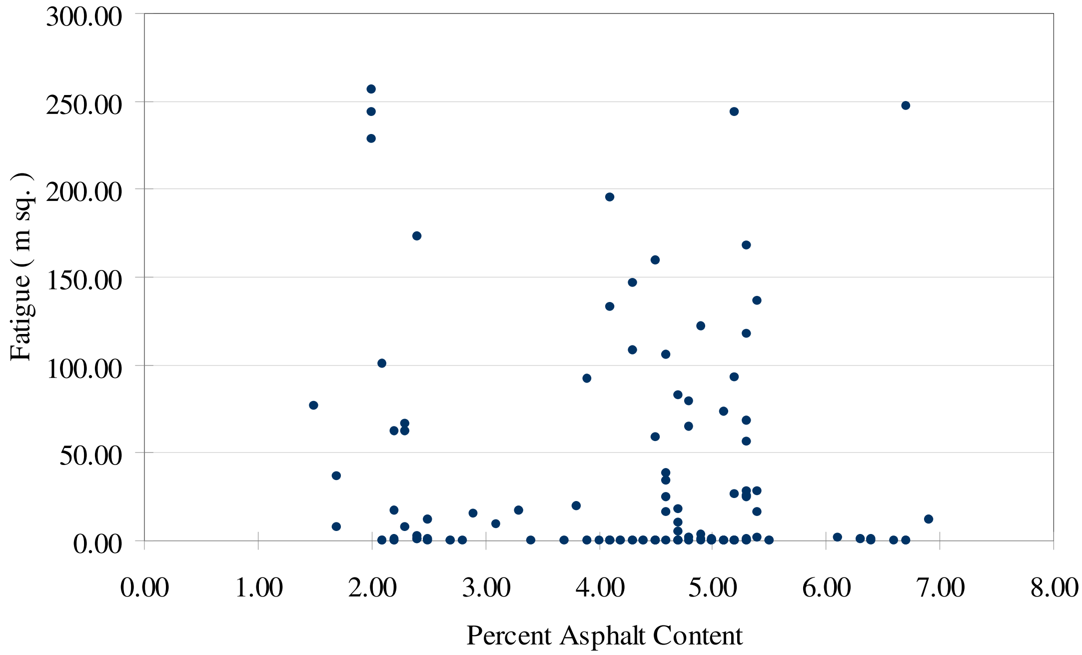


Figure 6.8 Fatigue vs. asphalt content for all SPS-1 sections in LTPP database

6.1.3 Longitudinal Cracking

Hot Mix Asphalt properties such as air voids content and amount of asphalt in the mixture, can have an effect on longitudinal cracking. However, Figure 6.9 does not show a particular trend. The cluster of high level cracking corresponding to low air voids (2 to 4%) is counterintuitive. However, at this point, no definite conclusions can be derived from this data. Further study of other possible factors would need to be undertaken to explain such poor performance.

These results highlight the need for more detailed analyses, with preferably the inclusion of forensic studies. LTPP database has data from several states. There were different approaches used in this analysis to be able to decipher credible relationships between QA tests and pavement performance.

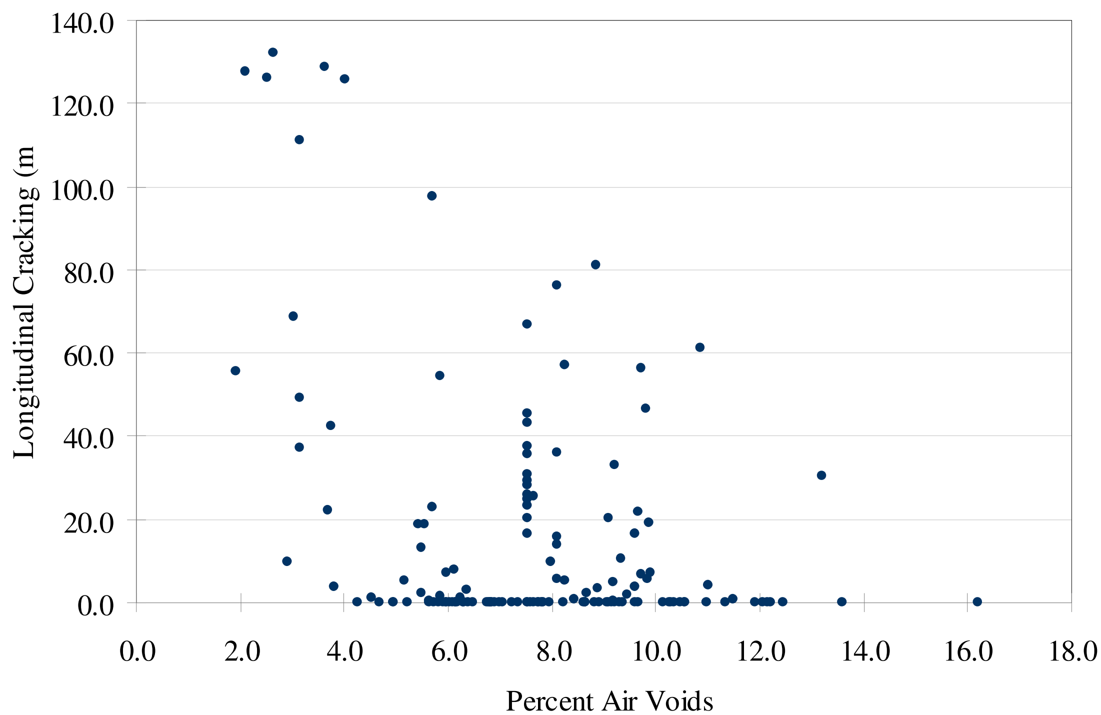


Figure 6.9 Longitudinal cracking vs. in-situ air voids content for all SPS-1 sections in LTPP database

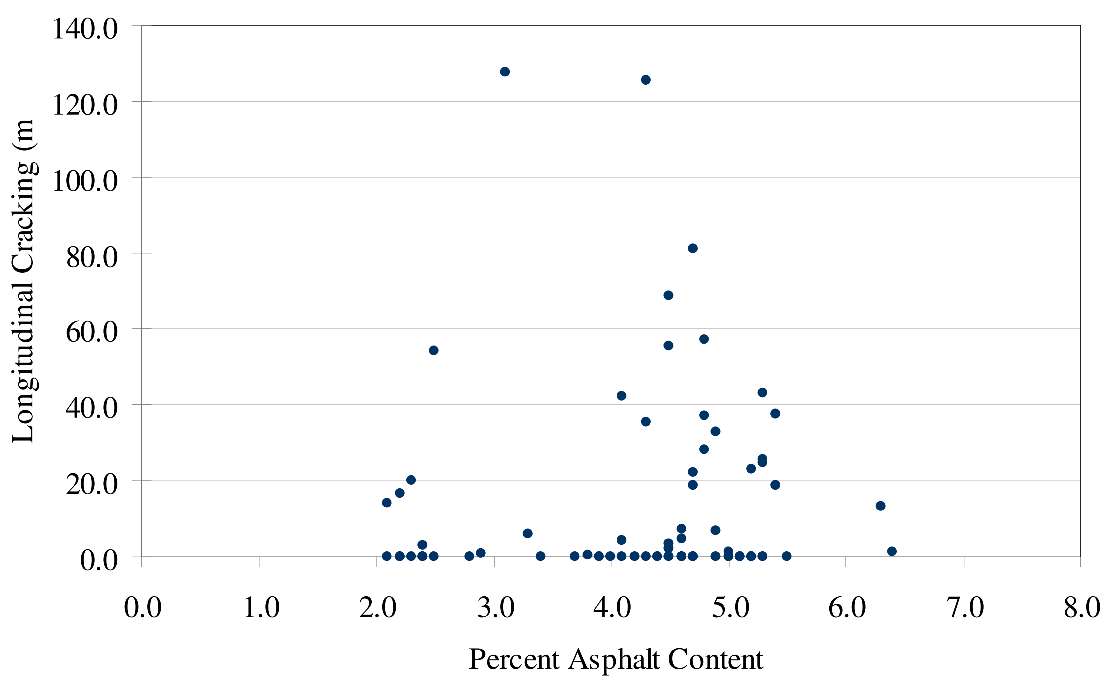


Figure 6.10 Longitudinal cracking vs. asphalt content for all SPS-1 sections in LTPP database

6.2 Analysis Using Percent-Within-Limits Concept

In a QA program, like the one that Michigan uses, a portion of payment completely depends on the percent within limits (PWL) achieved for one quality characteristic. For example, 40% of payment in Michigan flexible pavement construction depends on the in-situ density achieved immediately after final rolling. Therefore, the PWL for in-situ density is expected to be directly related to performance. Although PWL is calculated from quality achieved (in-situ density in this example), it takes into account mean as well as deviation from the mean. PWL of any project should be indicative of the expected performance irrespective of the site (state) and the HMA mix used. Therefore, relationships between PWL for various quality characteristics and performance would be of interest in this project.

A study of end-result specifications from many states showed that the majority of states follow statistical specifications using PWL. It was also observed that the specification limits used by the states are very similar to the ones being used in the state of Michigan. For example, most of the states use 91.5% or 92% as their lower specification limits for in-situ density, target ±0.4% or ±0.5% as limits for asphalt content and 4±1% or 4±1.2% for air voids at N_{design} for plant HMA samples. Also, the procedures followed by different states for calculating PWL are almost always identical. Therefore, it is possible to apply a common procedure to calculate PWL to data from other states, and then relate them to observed performance.

The inventory database in LTPP has mean bulk specific gravity for the as-placed mixture along with the maximum specific gravity. Therefore, mean in-situ density can be calculated for these projects. The same database also provides standard deviations of bulk specific gravity and number of samples used for calculating the standard deviations. However, standard deviations have not been provided for all the projects. Calculation of percent within limits requires mean and standard deviation for the projects. Therefore, those projects which had both types of data were filtered and extracted from the database. This filtering reduced the available data points from 2027 to 306.

Cracking performance for the projects is reported in the LTPP "monitoring" data table and is reported in the form of low, medium and high severity fatigue, longitudinal and transverse cracking. LTPP recommends that if distinction between severity levels does not have to be made in an analysis they can be added together. This was done to obtain the three categories of distress cracking. Then, the projects which had enough details for calculation of percent-within-limits in the inventory database were matched to those which had cracking performance reported.

Different projects have different ages, and distress surveys are collected at different times. To be able to compare cracking performance, it was first decided that a normalized cracking be calculated. Therefore, time histories of each of the projects reported were separately plotted for each project and the area under the fatigue cracking curve was calculated. Thereafter, the area was divided by the maximum age at the time of final survey. PWL values for those projects were then plotted against the normalized fatigue cracking performance. However, the plots did not show observable trends in terms of whether a change in PWL, which means change in quality of construction, reflects in cracking performance. Therefore, to avoid any distorting effect that the normalization process may have, three dimensional plots were generated with each data point having its own time in the third dimension.

It was also found that standard plotting options available in Microsoft Excel or Matlab do not represent such data very well. Therefore, a new plotting system was developed in-house using Matlab. This allowed plotting of each point as a column in the three dimensional space. Each column or point on the X-Y plane is plotted in color which is proportional to the magnitude of the distress being plotted. The color coding of the magnitude is shown in the color bar next to the plots. It should be noted that color coding is based on the range of distress values plotted in each plot, and therefore, the same color in two different plots may represent different magnitudes.

Figure 6.11 shows a three-dimensional plot of fatigue cracking at different times during the service life of projects versus percent-within-limits for in-situ density. This plot has 306 data points. However, any one project can have more than one point corresponding to different times at which the distress was measured. According to Figure 6.11, projects having PWL close to or equal to 100% show more distress than those with lower PWL values. However, this is not necessarily true. In QA programs generally a window around the target for each of the quality characteristic is allowed. As long as the quality characteristics remain within that window PWL values would be 100%. Therefore, a very large percentage of projects would have 100% PWL even if the mean and standard deviations of the quality characteristics may be varying. Also, many of such points with 100% or near-100% PWL plot right on top of each other and the points which have more fatigue cracking become more visible while hiding other points.

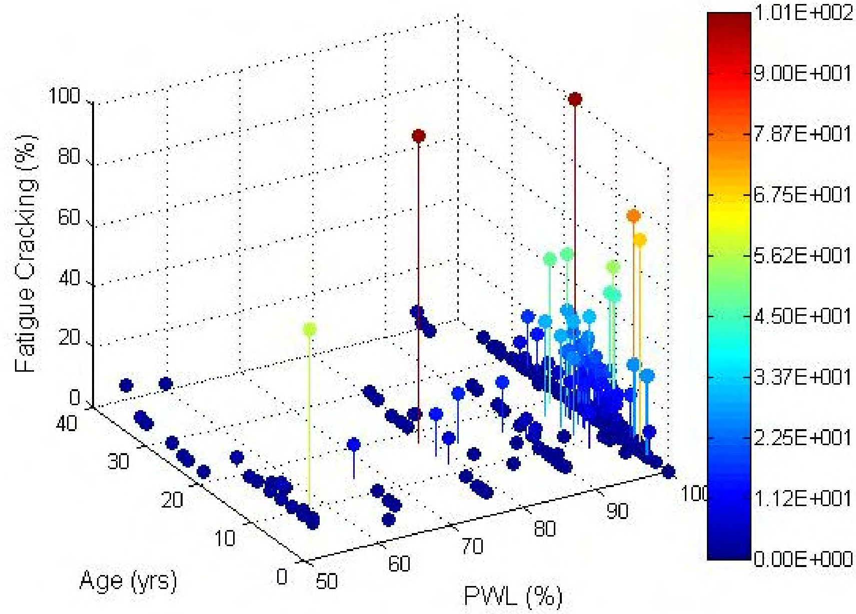


Figure 6.11 PWL (in-situ density) and fatigue cracking

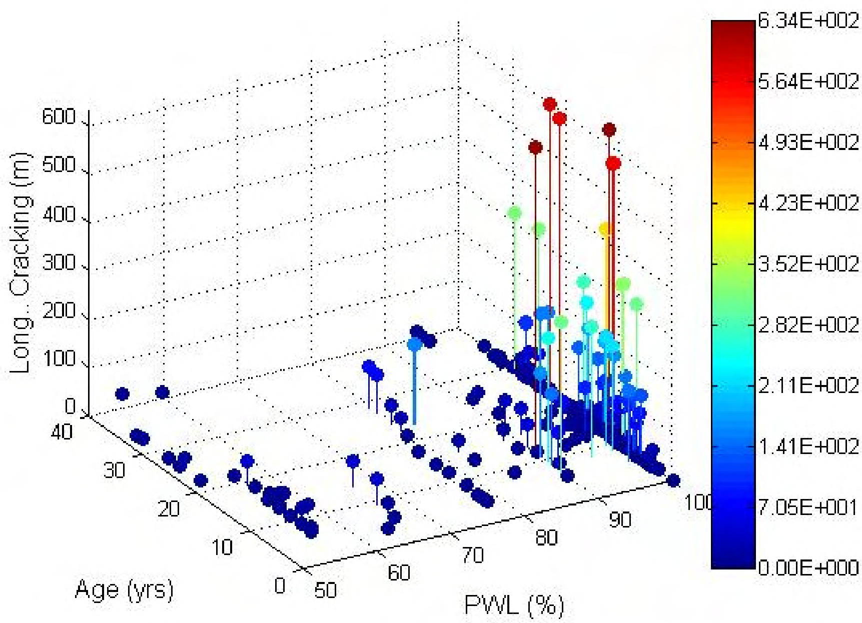


Figure 6.12 PWL (in-situ density) and longitudinal cracking

Figure 6.11 does show several projects for which PWL values are much lower than 100%, in some cases as low as 50%, which is the trigger point for "remove and replace" in QA programs, and yet they have no or almost no fatigue cracking. This is a very important observation from a QA point of view. Figures 6.12 and 6.13 also show many similar projects. This corroborates with some of the results that we have seen from MEPDG runs as well. Fatigue cracking occurs because of repeated loading and picks up towards the later phases of the pavement. Therefore, it is possible that two different projects may have been constructed to different quality levels, and still neither of them shows any cracking at the end of the 10th year. It is believed that if the pavements are monitored for 20 years or more, the differences would start becoming more visible in terms of magnitude of cracking at that age and the rate of deterioration after a certain level of initial deterioration.

Presence of data points with lower PWL with good cracking performance does raise issues about PWL being the criteria for payment. While PWL certainly represents quality achieved during construction, the design itself may be too good for the design life of the pavement. Our design methods are largely empirical in nature. The pavement constructed with such a design may perform well for much longer or shorter than the theoretical design life associated with the design. This issue has been further studied using simulation and discussed in greater detail in the following chapters of this report.

Figure 6.13 shows the results for transverse cracking. The dark brown colored columns represent cases where either there was no transverse cracking or the crack spacing would have been greater than 500 feet which is greater than the length of each LTPP section.

Figure 6.14 shows rutting performance for several projects against in-situ density. This plot also does not show any clear trend between PWL (in-situ density) and rutting. In this plot, there are a total of 433 points out of which 323 points have PWL higher than 95% and 281 of them have PWL higher than 99%.

The LTPP database also has plant air voids documented for several projects. Some of these projects have standard deviations also reported for air voids. All such project data points, numbering 452, were filtered and extracted from the database. These projects were then matched with their cracking and rutting performance from the monitoring data table and their age from the inventory data table. Figure 6.15 presents the three-dimensional plot of fatigue performance versus PWL (plant air voids) and age. This plot shows a greater number of projects which have poor PWL (plant air voids) having more fatigue cracking compared to those with PWL values closer to 100%. In most of the states, including Michigan, any projects having PWL between 90%

and 100% are not penalized; i.e., they are paid in full. Therefore, in this case, most of the projects being paid in full did not see much fatigue cracking. This tends to show that PWL (plant air voids) may be directly related to cracking performance. There are several projects, though, which have lower PWL, which means that they would have been penalized even though they do not show fatigue cracking.

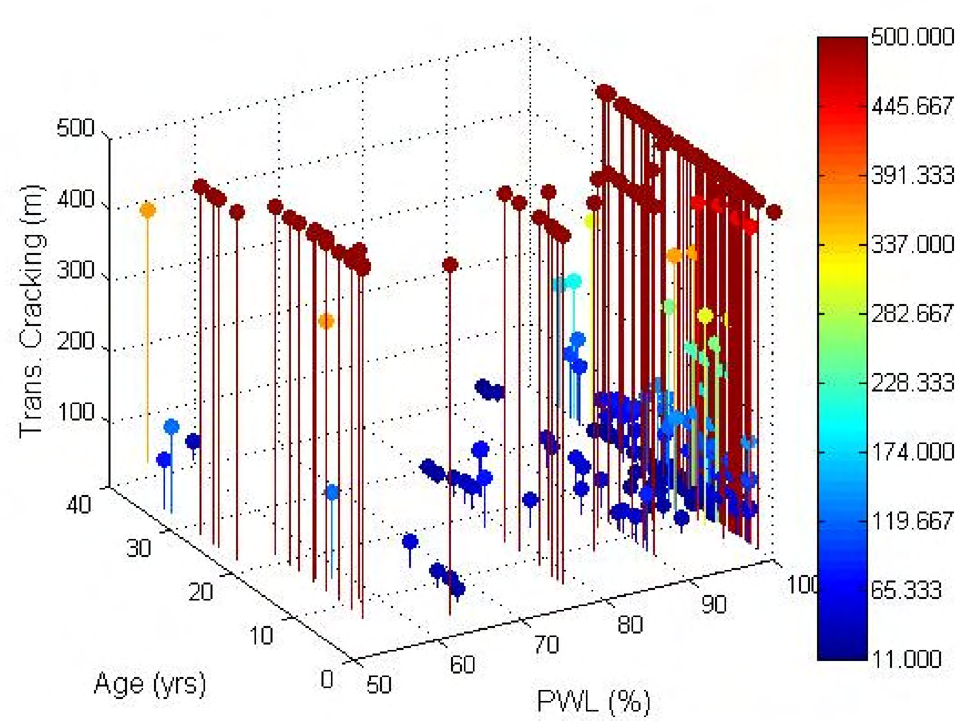


Figure 6.13 PWL (in-situ density) and transverse crack Spacing

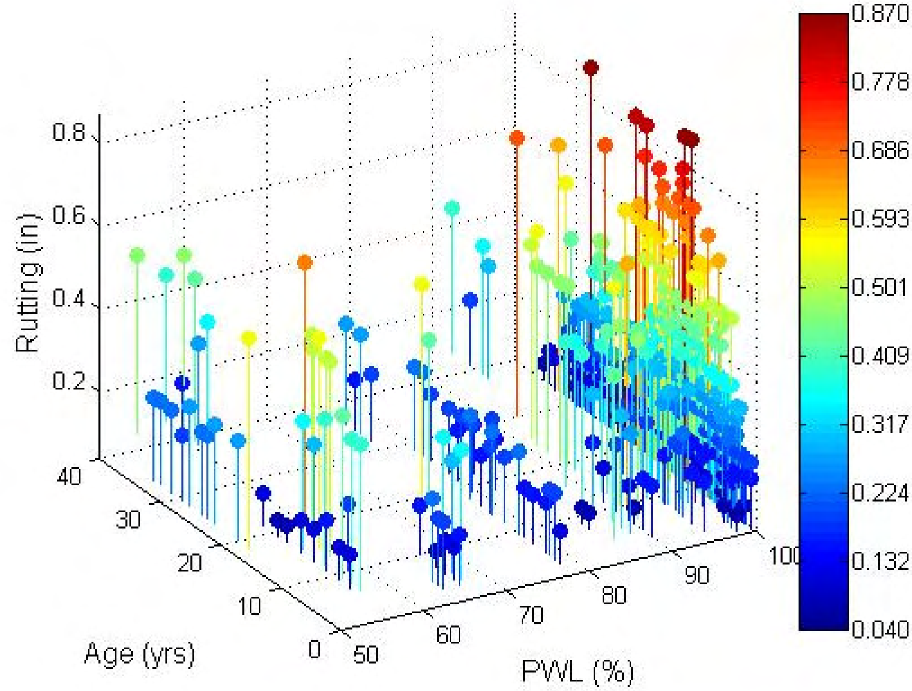


Figure 6.14 PWL (in-situ density) and rutting

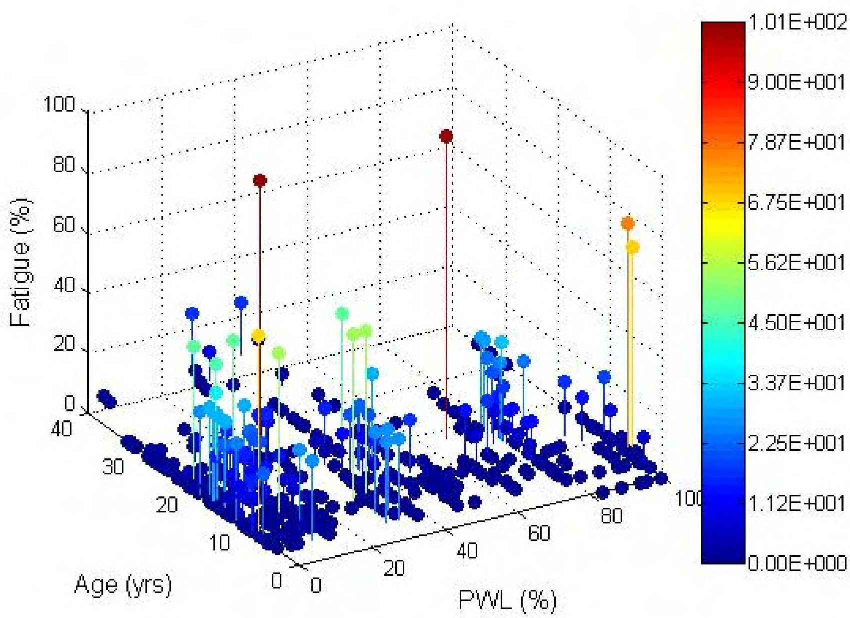


Figure 6.15 PWL (plant air voids) and fatigue cracking

Figures 6.16 and 6.17 present plots of longitudinal cracking and transverse cracking against PWL (plant air voids). Both plots show that as PWL (plant air voids) become higher, i.e. air voids are within the specification limits to a greater extent, there is better longitudinal and transverse cracking performance. Each plot has 453 points represented. Figure 6.18 shows rutting performance, which shows an increase in rutting with time; however, it does not show any observable relationship between PWL (plant air voids) and rutting.

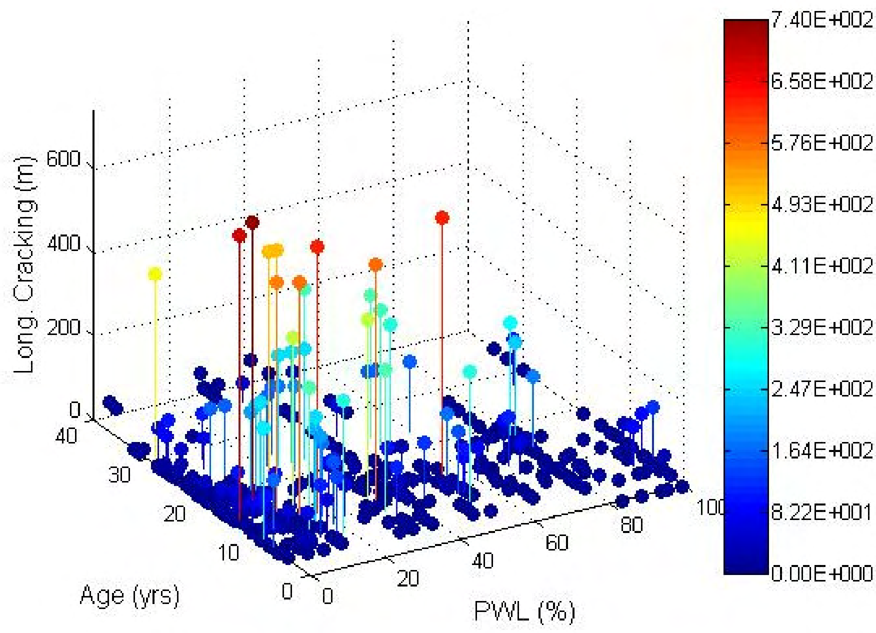


Figure 6.16 PWL (plant air voids) and longitudinal cracking

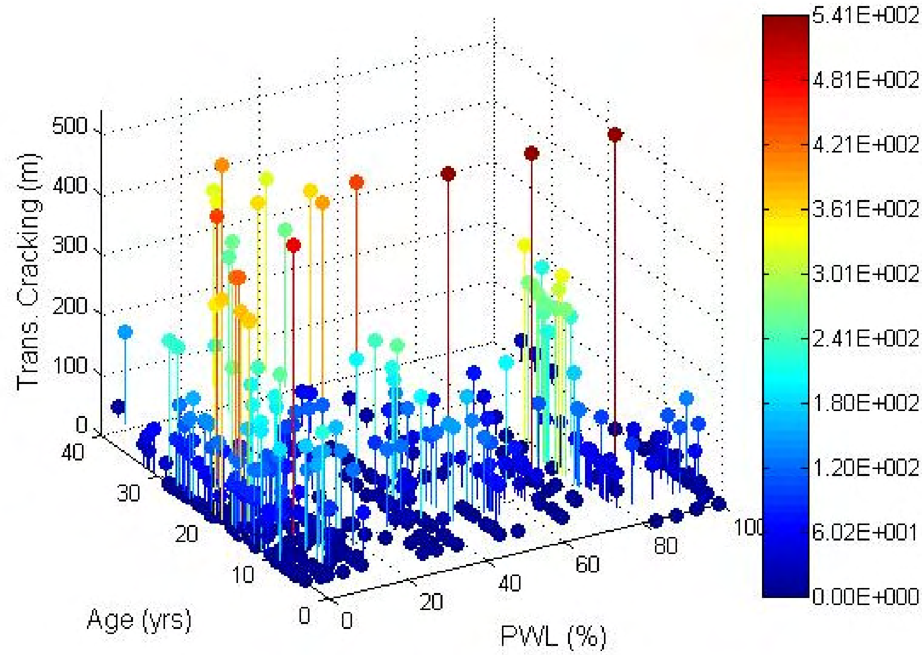


Figure 6.17 PWL (plant air voids) and transverse cracking

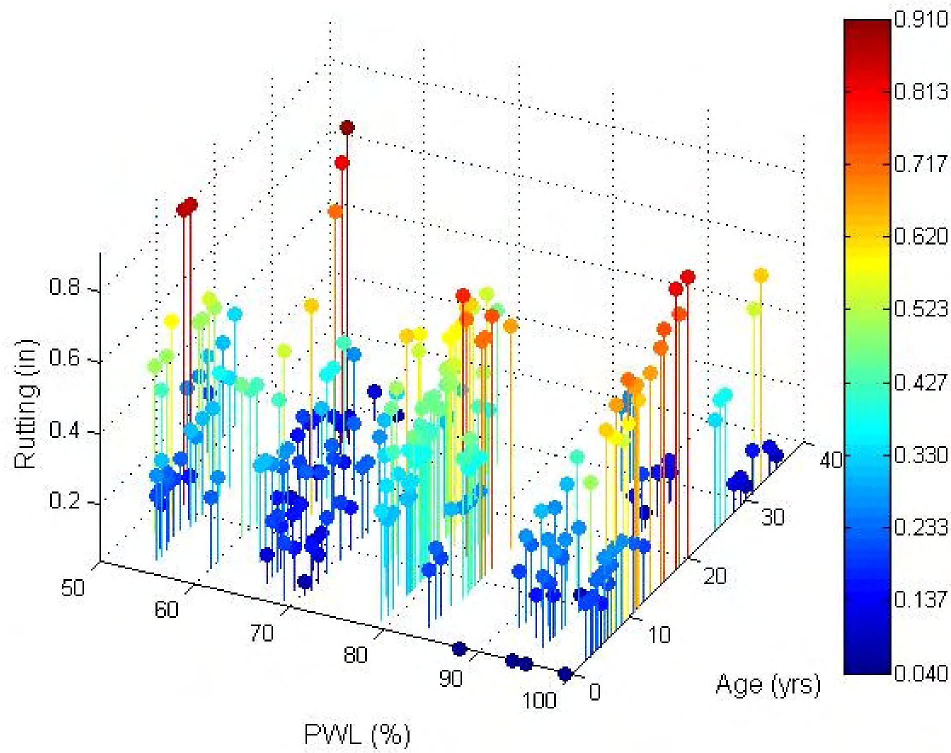


Figure 6.18 PWL (plant air voids) and rutting

6.3 Conclusions

This chapter presented details of empirical data analysis that was performed to determine if the quality characteristics most commonly used in quality assurance programs, such as those used in Michigan, affect pavement performance and can therefore be used to estimate the expected performance. If yes, then a quality assurance program can use suitable engineering and logistics model to rationally determine payment being made to the contractor and minimize losses because of poor quality construction by the contractor.

In this chapter, it was shown that analysis using empirical data generally cannot be used to develop a performance model. Such analysis can, at best, show some trends as to how a particular quality characteristic can affect certain aspects of pavement performance.

The analysis of data extracted from the LTPP database showed that pavements with low (< 4%) or high (>8%) in-situ air voids have higher probability of rutting than those in which air voids fall in the range of 4 to 8%. This indicates that in-situ air voids immediately after construction is an important test for a quality assurance program. Contrary to intuition, the asphalt content data did not show any clear trend in its effect on rutting. This is most likely because of not having data points corresponding to very low or very high asphalt content compared to the corresponding design values. Also, it is not possible to determine the contribution of the HMA layer to rutting directly from the LTPP; therefore some of the observed rutting may be caused by the unbound layers. The trends observed in the case of fatigue cracking and longitudinal cracking were not very clear. This is most likely due to the fact that several other factors are also affecting the fatigue performance, and these cannot be controlled in the analysis.

An argument was presented in this chapter as to how percent-within-limits for a particular quality characteristic should be related to pavement performance to rationally justify its use in pay factor equations. The analysis did not show clear relationships between PWL for in-situ density and fatigue cracking, longitudinal cracking and transverse cracking. Based on the plots showing PWL (plant air voids) versus cracking performance, it can be stated that while lower PWL does not necessarily mean poor performance, the probability of better performance certainly goes up with higher PWL. The analysis also showed that as PWL (plant air voids) become higher, i.e. air voids are within the specification limits to a greater extent, there is better longitudinal and transverse cracking performance. No observable relationship between PWL (plant air voids) and rutting was found in the analysis.

In conclusion, the results derived from the empirical data analysis presented in this chapter are mixed and unclear. Therefore, a mechanistic-empirical analysis needs to be performed to derive firm conclusions required for assessing quality assurance programs such as the one being used by the state of Michigan.

Chapter 7: Mechanistic-Empirical Analysis

7.1 Purpose

A major issue with this research effort was the lack of construction data from actual projects. As reported earlier in the report, asphalt pavement projects that are older than 10 years were not built with Superpave mixes. The current QA program as adopted by MDOT is based on Superpave. Therefore, even if some test data is available for these older projects they cannot be easily used to assess the current QA program. For example, in the case of older projects, no data would be available for air voids and VMA at N_{design}. Both these factors account for forty five percent of the total pay factor. When the projects are less than 10 years old, it is generally difficult to assess as to how they are performing because they have not gone through their design service life unless they are already performing poorly. In addition to these factors, even in the case of very well controlled experiments like SPS-1, it is not very easy to draw clear relationships between cause and effect because of the number of variables involved, several of which may not be controllable.

An alternative approach would be to assess the QA program by relating the QA parameters to expected performance if reliable predictions can be made for performance. The ME-PDG provides an opportunity to be able to predict pavement performance when sufficiently accurate inputs are provided. While ME-PDG predictions may not have a high level of accuracy, several studies have shown that M-E PDG predictions can be reasonable provided that proper (local) calibration is done. Also, it is important to mention that ME-PDG uses the best knowledge available today to the pavement engineering community. Given the lack of real construction data and recognizing that many other factors (other than QA factors) may be affecting in-service pavement performance over time, it is useful to attempt this exercise using the tools provided in ME-PDG software.

7.2 Analysis Approach

The Superpave mix design process requires specimen preparation and testing of the mixture being designed. This is done in order to determine the optimal asphalt content and achieve the desired 4 percent air voids. These tests lead to valuable information not just in the form of optimal asphalt content but also in terms of differences in mix characteristics that would result from the use of actual aggregate blends (not just theoretical blending) and varying asphalt content in the mix. These provide realistic inputs for predicting the change in performance as a result of such changes in the mix, thus simulating what actually happens during the construction process.

The following paragraphs present the steps that were taken to come up with inputs based on actual testing. These inputs represent the variation that would occur because of material variability, construction process itself, operator error etc. For instance, the bold line in Figure 7.1 shows gradation curve as target in a project. However, the actual gradation of the aggregates in the mix could be slightly different from the target and could look like any of the other gradation curves in the same figure. In reality, the other gradation curves were taken from other projects in this case. The QA program is meant to identify these deviations and also limit them so that the overall pavement quality does not diminish appreciably. In this strategy, we introduce such deviations in the mixture and use ME-PDG to quantify the change in performance. Assessing the type and magnitude of changes in performance provides knowledge about the level of variabilities that would be acceptable under a QA program. In all the analyses that follow, only MDOT mixes have been used. An example of this strategy is presented below.

Step 1: Identify the project, or a typical example of the type of project that needs to be analyzed with respect to the QA program. We chose the following three projects with varying performance.

(i) Section ID 29581W located in Lansing, MI and constructed in 1995
(ii) Section ID 18890N located in Ludington, MI and constructed in 1989
(iii) SPS-1 site 0117 located near Lansing, MI and constructed in 1994

Step 2: Match the mix used in the identified projects as closely as possible to a Superpave mix currently being used by MDOT. The criteria used for this was

(i) Traffic level
(ii) Climatic conditions
(iii) Asphalt grade
(iv) Aggregate gradation of the mix
(v) Asphalt content of the mix

Using these criteria, the following mixes were identified from the inventory of recently used Michigan mixes available to us.

(i) Section ID 29581W: Mix Design# 07MD161, 07MD090, 07MD0170 and 07MD0234
(ii) Section ID 18890N: Mix Design# 07MD046 and 07MD071
(iii) SPS-1 site 0117: Mix Design# 07MD042 and 07MD290

The list above gives project identification number/name followed by some mix design numbers. The first mix design number corresponds to the MDOT mix that was found to be closest to the original mix used in the project based on the criteria listed earlier in this section. The remaining mixes are similar in aggregate gradation, asphalt grade etc. but have some differences that would be representative of the difference that would exist in a real project because of variability in aggregate gradation etc. In summary, the first listed mix is expected to represent the original mix that was supposed to be used in the construction process; i.e., the target mix, referred to as "target mix" henceforth. The remaining mixes are the ones that could result because of material and construction variability.

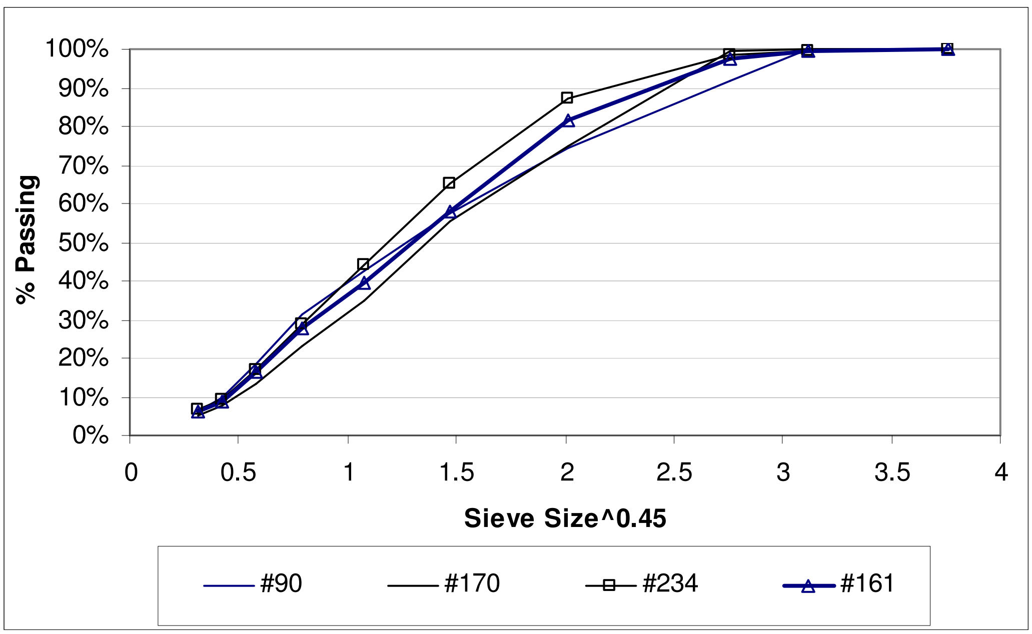


Figure 7. 1 Aggregate gradation for the four mixes chosen for the first project

Step 3: Use the target mix to determine inputs for ME-PDG software. The inputs available from the mix design would be for the mix as produced in the plant like asphalt content and aggregate gradation. Also, choose other levels of asphalt content that can result because of variability in the production process. Table 7.1 presents the asphalt content measurements from actual projects constructed in 2002. Superpave mix designs were used in all these projects. In this analysis, five levels of asphalt content were chosen:

(i) Optimum AC – 0.5%
(ii) Optimum AC – 0.25%
(iii) Optimum AC
(iv) Optimum AC +0.25%
(v) Optimum AC +0.5%

Table 7.1 Asphalt content measurements from actual projects

	Measured Asphalt Content (%)		Target (%)	Deviation from Target (%)	
	Min	Max			
Project 1	5.35	5.74	5.5	-0.15	0.24
Project 2	5.3	6.1	5.6	-0.3	0.5
Project 3	4.5	4.9	4.6	-0.1	0.3
Project 4	5.2	5.7	5.4	-0.2	0.3
Project 5	4.5	4.8	4.5	0	0.3
Project 6	5.3	5.7	5.4	-0.1	0.3
Project 7	5	5.4	5.2	-0.2	0.2
Project 8	5.2	5.4	5.3	-0.1	0.1
Project 9	5.2	5.6	5.4	-0.2	0.2
Project 10	4.4	4.8	4.7	-0.3	0.1

Step 4: Choose the range of air voids which can result because of the lay down of the mix, i.e. in-situ air voids. Table 7.2 presents the in-situ air voids from the same projects as those mentioned in Table 7.1. Based on these actual observations the following levels of in-situ air voids were chosen in this analysis.

(i) 12.0%
(ii) 9.5%
(iii) 7.0%
(iv) 4.5%
(v) 2.0%

Table 7.2 In-situ air void measurements from actual projects

	Measured In-situ Air Voids (%)		Standard Deviation (%)
	Min	Max	
Project 1	2.7	10.2	1.45
Project 2	2.9	9.1	1.80
Project 3	2.6	9.0	1.25
Project 4	4.0	8.6	0.98
Project 5	2.8	9.6	1.75
Project 6	2.0	10.0	1.84
Project 7	2.5	6.1	1.18
Project 8	4.4	11.9	1.15
Project 9	4.2	10.5	1.39
Project 10	3.0	10.1	1.23

Step 5: Make a matrix with all possible combinations of the levels chosen for asphalt content and air voids. This should be followed by calculating the other parameters with reference to the test results from the mix design procedure and using the volumetric relationships of the mixes. For example, for cases in the matrix with asphalt content less than the optimal value, one can refer to the volumetric properties corresponding to that from the table obtained during the Superpave mix design procedure. The said table lists air voids, VMA, VFA at N_{design} gyrations for the same mix but with varying amounts of asphalt content. This step helps ensure that the inputs selected for ME-PDG are characteristics of the original mix even though the mix components may not have been added in the right proportion according to the design. This is the same as what actually happens in real mix production and construction of the pavements.

Step 6: Step 5 should be repeated for each mix identified for each of the projects. However, when determining the inputs for the mix other than the target mix, optimal asphalt content for the target mix should be used. Tables 7.3, 7.4 and 7.5 present the matrices for the three cases that we analyzed.

Step 7: Run ME-PDG for each case within the matrix prepared in steps 5 and 6.

Step 8: Analyze a project by simulating quality characteristic test results from QA point of view and then relate them to performance.

It is important to determine the accuracy of performance predicted by MEPDG for this strategy of analysis to be successful. Figures 7.2 through 7.6 compare the performance predicted by MEPDG software with the actual performance observed. For the sake of brevity, only some cases have been presented here. Also, in some cases, the data for actual performance seem to be erroneous. Therefore, they could not be used for this verification purpose. These comparisons show that there is reasonable agreement between actual and predicted performance. It is also possible to calibrate the models used in MEPDG using local data. That would certainly further improve the accuracy of predictions.

Table 7.3 Matrix for the first project selected for analysis

Run No.	Design#	AC	Density (in-situ) (% Gmm)	% Retained on 3/4 inch	% Retained on 3/8 inch	% Retained on #4	P#200	Asphalt Grade	Pbe (vol.) (%)	In-Situ Air Voids (%)	Tot. Ut. Wt. (lb/ft3)
1	07MD161	JMF-0.5	88	0	2.4	18.4	6	PG 64-22	10.270	12.0	136.545
2	07MD161	JMF-0.5	90.5	0	2.4	18.4	6	PG 64-22	10.562	9.5	140.425
3	07MD161	JMF-0.5	93	0	2.4	18.4	6	PG 64-22	10.853	7.0	144.304
4	07MD161	JMF-0.5	95.5	0	2.4	18.4	6	PG 64-22	11.145	4.5	148.183
5	07MD161	JMF-0.5	98	0	2.4	18.4	6	PG 64-22	11.437	2.0	152.062
6	07MD161	JMF-0.25	88	0	2.4	18.4	6	PG 64-22	10.764	12.0	136.015
7	07MD161	JMF-0.25	90.5	0	2.4	18.4	6	PG 64-22	11.070	9.5	139.879
8	07MD161	JMF-0.25	93	0	2.4	18.4	6	PG 64-22	11.376	7.0	143.743
9	07MD161	JMF-0.25	95.5	0	2.4	18.4	6	PG 64-22	11.681	4.5	147.607
10	07MD161	JMF-0.25	98	0	2.4	18.4	6	PG 64-22	11.987	2.0	151.472
11	07MD161	JMF	88	0	2.4	18.4	6	PG 64-22	11.257	12.0	135.508
12	07MD161	JMF	90.5	0	2.4	18.4	6	PG 64-22	11.576	9.5	139.357
13	07MD161	JMF	93	0	2.4	18.4	6	PG 64-22	11.896	7.0	143.200
14	07MD161	JMF	95.5	0	2.4	18.4	6	PG 64-22	12.216	4.5	147.056
15	07MD161	JMF	98	0	2.4	18.4	6	PG 64-22	12.536	2.0	150.905
16	07MD161	JMF+0.25	88	0	2.4	18.4	6	PG 64-22	11.736	12.0	135.005
17	07MD161	JMF+0.25	90.5	0	2.4	18.4	6	PG 64-22	12.069	9.5	138.840
18	07MD161	JMF+0.25	93	0	2.4	18.4	6	PG 64-22	12.403	7.0	142.675
19	07MD161	JMF+0.25	95.5	0	2.4	18.4	6	PG 64-22	12.736	4.5	146.511
20	07MD161	JMF+0.25	98	0	2.4	18.4	6	PG 64-22	13.070	2.0	150.346
21	07MD161	JMF+0.5	88	0	2.4	18.4	6	PG 64-22	12.224	12.0	134.522
22	07MD161	JMF+0.5	90.5	0	2.4	18.4	6	PG 64-22	12.571	9.5	138.344
23	07MD161	JMF+0.5	93	0	2.4	18.4	6	PG 64-22	12.918	7.0	142.165
24	07MD161	JMF+0.5	95.5	0	2.4	18.4	6	PG 64-22	13.266	4.5	145.987
25	07MD161	JMF+0.5	98	0	2.4	18.4	6	PG 64-22	13.613	2.0	149.809
26	07MD090	JMF-0.5	88	0	8.1	25.7	6.1	PG 64-22	10.486	12.0	136.625
27	07MD090	JMF-0.5	90.5	0	8.1	25.7	6.1	PG 64-22	10.784	9.5	140.507
28	07MD090	JMF-0.5	93	0	8.1	25.7	6.1	PG 64-22	11.082	7.0	144.388
29	07MD090	JMF-0.5	95.5	0	8.1	25.7	6.1	PG 64-22	11.380	4.5	148.269
30	07MD090	JMF-0.5	98	0	8.1	25.7	6.1	PG 64-22	11.678	2.0	152.151
31	07MD090	JMF-0.25	88	0	8.1	25.7	6.1	PG 64-22	10.982	12.0	136.099
32	07MD090	JMF-0.25	90.5	0	8.1	25.7	6.1	PG 64-22	11.294	9.5	139.966
33	07MD090	JMF-0.25	93	0	8.1	25.7	6.1	PG 64-22	11.606	7.0	143.832
34	07MD090	JMF-0.25	95.5	0	8.1	25.7	6.1	PG 64-22	11.918	4.5	147.699
35	07MD090	JMF-0.25	98	0	8.1	25.7	6.1	PG 64-22	12.230	2.0	151.565
36	07MD090	JMF	88	0	8.1	25.7	6.1	PG 64-22	11.479	12.0	135.612
37	07MD090	JMF	90.5	0	8.1	25.7	6.1	PG 64-22	11.805	9.5	139.465
38	07MD090	JMF	93	0	8.1	25.7	6.1	PG 64-22	12.131	7.0	143.318
39	07MD090	JMF	95.5	0	8.1	25.7	6.1	PG 64-22	12.457	4.5	147.170
40	07MD090	JMF	98	0	8.1	25.7	6.1	PG 64-22	12.783	2.0	151.023
41	07MD090	JMF+0.25	88	0	8.1	25.7	6.1	PG 64-22	11.957	12.0	135.092
42	07MD090	JMF+0.25	90.5	0	8.1	25.7	6.1	PG 64-22	12.296	9.5	138.929
43	07MD090	JMF+0.25	93	0	8.1	25.7	6.1	PG 64-22	12.636	7.0	142.767
44	07MD090	JMF+0.25	95.5	0	8.1	25.7	6.1	PG 64-22	12.976	4.5	146.605
45	07MD090	JMF+0.25	98	0	8.1	25.7	6.1	PG 64-22	13.316	2.0	150.443
46	07MD090	JMF+0.5	88	0	8.1	25.7	6.1	PG 64-22	12.448	12.0	134.653
47	07MD090	JMF+0.5	90.5	0	8.1	25.7	6.1	PG 64-22	12.802	9.5	138.479
48	07MD090	JMF+0.5	93	0	8.1	25.7	6.1	PG 64-22	13.155	7.0	142.304
49	07MD090	JMF+0.5	95.5	0	8.1	25.7	6.1	PG 64-22	13.509	4.5	146.129
50	07MD090	JMF+0.5	98	0	8.1	25.7	6.1	PG 64-22	13.863	2.0	149.955

Table 7.4 Matrix for the second project selected for analysis

Run No.	Design#	AC	Density (in-situ) (% Gmm)	% Retained on 3/4 inch	% Retained on 3/8 inch	% Retained on #4	P#200	Asphalt Grade	Pbe (vol.) (%)	In-Situ Air Voids (%)	Tot. Ut. Wt. (lb/ft3)
1	07MD046	JMF-0.5	88	0	15.3	31.1	5.4	PG64-28	9.203	12.0	137.287
2	07MD046	JMF-0.5	90.5	0	15.3	31.1	5.4	PG64-28	9.464	9.5	141.187
3	07MD046	JMF-0.5	93	0	15.3	31.1	5.4	PG64-28	9.726	7.0	145.088
4	07MD046	JMF-0.5	95.5	0	15.3	31.1	5.4	PG64-28	9.987	4.5	148.988
5	07MD046	JMF-0.5	98	0	15.3	31.1	5.4	PG64-28	10.248	2.0	152.888
6	07MD046	JMF-0.25	88	0	15.3	31.1	5.4	PG64-28	9.706	12.0	136.805
7	07MD046	JMF-0.25	90.5	0	15.3	31.1	5.4	PG64-28	9.982	9.5	140.691
8	07MD046	JMF-0.25	93	0	15.3	31.1	5.4	PG64-28	10.258	7.0	144.578
9	07MD046	JMF-0.25	95.5	0	15.3	31.1	5.4	PG64-28	10.533	4.5	148.464
10	07MD046	JMF-0.25	98	0	15.3	31.1	5.4	PG64-28	10.809	2.0	152.351
11	07MD046	JMF	88	0	15.3	31.1	5.4	PG64-28	10.202	12.0	136.243
12	07MD046	JMF	90.5	0	15.3	31.1	5.4	PG64-28	10.492	9.5	140.114
13	07MD046	JMF	93	0	15.3	31.1	5.4	PG64-28	10.782	7.0	143.984
14	07MD046	JMF	95.5	0	15.3	31.1	5.4	PG64-28	11.072	4.5	147.855
15	07MD046	JMF	98	0	15.3	31.1	5.4	PG64-28	11.362	2.0	151.725
16	07MD046	JMF+0.25	88	0	15.3	31.1	5.4	PG64-28	10.696	12.0	135.732
17	07MD046	JMF+0.25	90.5	0	15.3	31.1	5.4	PG64-28	11.000	9.5	139.588
18	07MD046	JMF+0.25	93	0	15.3	31.1	5.4	PG64-28	11.304	7.0	143.444
19	07MD046	JMF+0.25	95.5	0	15.3	31.1	5.4	PG64-28	11.608	4.5	147.300
20	07MD046	JMF+0.25	98	0	15.3	31.1	5.4	PG64-28	11.912	2.0	151.156
21	07MD046	JMF+0.5	88	0	15.3	31.1	5.4	PG64-28	11.185	12.0	135.241
22	07MD046	JMF+0.5	90.5	0	15.3	31.1	5.4	PG64-28	11.503	9.5	139.083
23	07MD046	JMF+0.5	93	0	15.3	31.1	5.4	PG64-28	11.821	7.0	142.925
24	07MD046	JMF+0.5	95.5	0	15.3	31.1	5.4	PG64-28	12.139	4.5	146.767
25	07MD046	JMF+0.5	98	0	15.3	31.1	5.4	PG64-28	12.456	2.0	150.609
26	07MD071	JMF-0.5	88	0	10.1	27.8	5.3	PG64-28	9.488	12.0	135.612
27	07MD071	JMF-0.5	90.5	0	10.1	27.8	5.3	PG64-28	9.757	9.5	139.464
28	07MD071	JMF-0.5	93	0	10.1	27.8	5.3	PG64-28	10.027	7.0	143.317
29	07MD071	JMF-0.5	95.5	0	10.1	27.8	5.3	PG64-28	10.296	4.5	147.170
30	07MD071	JMF-0.5	98	0	10.1	27.8	5.3	PG64-28	10.566	2.0	151.022
31	07MD071	JMF-0.25	88	0	10.1	27.8	5.3	PG64-28	9.988	12.0	135.080
32	07MD071	JMF-0.25	90.5	0	10.1	27.8	5.3	PG64-28	10.272	9.5	138.918
33	07MD071	JMF-0.25	93	0	10.1	27.8	5.3	PG64-28	10.556	7.0	142.755
34	07MD071	JMF-0.25	95.5	0	10.1	27.8	5.3	PG64-28	10.839	4.5	146.593
35	07MD071	JMF-0.25	98	0	10.1	27.8	5.3	PG64-28	11.123	2.0	150.430
36	07MD071	JMF	88	0	10.1	27.8	5.3	PG64-28	10.480	12.0	134.597
37	07MD071	JMF	90.5	0	10.1	27.8	5.3	PG64-28	10.778	9.5	138.420
38	07MD071	JMF	93	0	10.1	27.8	5.3	PG64-28	11.076	7.0	142.244
39	07MD071	JMF	95.5	0	10.1	27.8	5.3	PG64-28	11.373	4.5	146.068
40	07MD071	JMF	98	0	10.1	27.8	5.3	PG64-28	11.671	2.0	149.892
41	07MD071	JMF+0.25	88	0	10.1	27.8	5.3	PG64-28	10.961	12.0	134.118
42	07MD071	JMF+0.25	90.5	0	10.1	27.8	5.3	PG64-28	11.273	9.5	137.928
43	07MD071	JMF+0.25	93	0	10.1	27.8	5.3	PG64-28	11.584	7.0	141.738
44	07MD071	JMF+0.25	95.5	0	10.1	27.8	5.3	PG64-28	11.896	4.5	145.548
45	07MD071	JMF+0.25	98	0	10.1	27.8	5.3	PG64-28	12.207	2.0	149.358
46	07MD071	JMF+0.5	88	0	10.1	27.8	5.3	PG64-28	11.454	12.0	133.628
47	07MD071	JMF+0.5	90.5	0	10.1	27.8	5.3	PG64-28	11.779	9.5	137.424
48	07MD071	JMF+0.5	93	0	10.1	27.8	5.3	PG64-28	12.104	7.0	141.221
49	07MD071	JMF+0.5	95.5	0	10.1	27.8	5.3	PG64-28	12.430	4.5	145.017
50	07MD071	JMF+0.5	98	0	10.1	27.8	5.3	PG64-28	12.755	2.0	148.813

Table 7.5 Matrix for the third project selected for analysis

Run No.	Design#	AC	Density (in-situ) (% Gmm)	% Retained on 3/4 inch	% Retained on 3/8 inch	% Retained on #4	P#200	Asphalt Grade	Pbe (vol.) (%)	In-Situ Air Voids (%)	Tot. Ut. Wt. (lb/ft3)
1	07MD042	JMF-0.5	88	0.1	24.4	58.6	4.2	PG58-22	8.516	12.0	138.243
2	07MD042	JMF-0.5	90.5	0.1	24.4	58.6	4.2	PG58-22	8.758	9.5	142.170
3	07MD042	JMF-0.5	93	0.1	24.4	58.6	4.2	PG58-22	9.000	7.0	146.098
4	07MD042	JMF-0.5	95.5	0.1	24.4	58.6	4.2	PG58-22	9.242	4.5	150.025
5	07MD042	JMF-0.5	98	0.1	24.4	58.6	4.2	PG58-22	9.484	2.0	153.953
6	07MD042	JMF-0.25	88	0.1	24.4	58.6	4.2	PG58-22	9.028	12.0	137.745
7	07MD042	JMF-0.25	90.5	0.1	24.4	58.6	4.2	PG58-22	9.284	9.5	141.658
8	07MD042	JMF-0.25	93	0.1	24.4	58.6	4.2	PG58-22	9.541	7.0	145.571
9	07MD042	JMF-0.25	95.5	0.1	24.4	58.6	4.2	PG58-22	9.797	4.5	149.485
10	07MD042	JMF-0.25	98	0.1	24.4	58.6	4.2	PG58-22	10.054	2.0	153.398
11	07MD042	JMF	88	0.1	24.4	58.6	4.2	PG58-22	9.536	12.0	137.200
12	07MD042	JMF	90.5	0.1	24.4	58.6	4.2	PG58-22	9.807	9.5	141.098
13	07MD042	JMF	93	0.1	24.4	58.6	4.2	PG58-22	10.077	7.0	144.995
14	07MD042	JMF	95.5	0.1	24.4	58.6	4.2	PG58-22	10.348	4.5	148.893
15	07MD042	JMF	98	0.1	24.4	58.6	4.2	PG58-22	10.619	2.0	152.791
16	07MD042	JMF+0.25	88	0.1	24.4	58.6	4.2	PG58-22	10.037	12.0	136.676
17	07MD042	JMF+0.25	90.5	0.1	24.4	58.6	4.2	PG58-22	10.322	9.5	140.559
18	07MD042	JMF+0.25	93	0.1	24.4	58.6	4.2	PG58-22	10.607	7.0	144.442
19	07MD042	JMF+0.25	95.5	0.1	24.4	58.6	4.2	PG58-22	10.892	4.5	148.325
20	07MD042	JMF+0.25	98	0.1	24.4	58.6	4.2	PG58-22	11.177	2.0	152.207
21	07MD042	JMF+0.5	88	0.1	24.4	58.6	4.2	PG58-22	10.529	12.0	136.186
22	07MD042	JMF+0.5	90.5	0.1	24.4	58.6	4.2	PG58-22	10.828	9.5	140.055
23	07MD042	JMF+0.5	93	0.1	24.4	58.6	4.2	PG58-22	11.127	7.0	143.924
24	07MD042	JMF+0.5	95.5	0.1	24.4	58.6	4.2	PG58-22	11.426	4.5	147.793
25	07MD042	JMF+0.5	98	0.1	24.4	58.6	4.2	PG58-22	11.726	2.0	151.662
26	07MD290	JMF-0.5	88	0	26	53.6	5	PG58-22	9.113	12.0	137.832
27	07MD290	JMF-0.5	90.5	0	26	53.6	5	PG58-22	9.371	9.5	141.748
28	07MD290	JMF-0.5	93	0	26	53.6	5	PG58-22	9.630	7.0	145.663
29	07MD290	JMF-0.5	95.5	0	26	53.6	5	PG58-22	9.889	4.5	149.579
30	07MD290	JMF-0.5	98	0	26	53.6	5	PG58-22	10.148	2.0	153.495
31	07MD290	JMF-0.25	88	0	26	53.6	5	PG58-22	9.617	12.0	137.321
32	07MD290	JMF-0.25	90.5	0	26	53.6	5	PG58-22	9.890	9.5	141.222
33	07MD290	JMF-0.25	93	0	26	53.6	5	PG58-22	10.164	7.0	145.124
34	07MD290	JMF-0.25	95.5	0	26	53.6	5	PG58-22	10.437	4.5	149.025
35	07MD290	JMF-0.25	98	0	26	53.6	5	PG58-22	10.710	2.0	152.926
36	07MD290	JMF	88	0	26	53.6	5	PG58-22	10.118	12.0	136.811
37	07MD290	JMF	90.5	0	26	53.6	5	PG58-22	10.406	9.5	140.698
38	07MD290	JMF	93	0	26	53.6	5	PG58-22	10.693	7.0	144.585
39	07MD290	JMF	95.5	0	26	53.6	5	PG58-22	10.981	4.5	148.471
40	07MD290	JMF	98	0	26	53.6	5	PG58-22	11.268	2.0	152.358
41	07MD290	JMF+0.25	88	0	26	53.6	5	PG58-22	10.609	12.0	136.295
42	07MD290	JMF+0.25	90.5	0	26	53.6	5	PG58-22	10.910	9.5	140.167
43	07MD290	JMF+0.25	93	0	26	53.6	5	PG58-22	11.212	7.0	144.039
44	07MD290	JMF+0.25	95.5	0	26	53.6	5	PG58-22	11.513	4.5	147.911
45	07MD290	JMF+0.25	98	0	26	53.6	5	PG58-22	11.814	2.0	151.783
46	07MD290	JMF+0.5	88	0	26	53.6	5	PG58-22	11.094	12.0	135.785
47	07MD290	JMF+0.5	90.5	0	26	53.6	5	PG58-22	11.409	9.5	139.643
48	07MD290	JMF+0.5	93	0	26	53.6	5	PG58-22	11.724	7.0	143.500
49	07MD290	JMF+0.5	95.5	0	26	53.6	5	PG58-22	12.039	4.5	147.358
50	07MD290	JMF+0.5	98	0	26	53.6	5	PG58-22	12.354	2.0	151.215

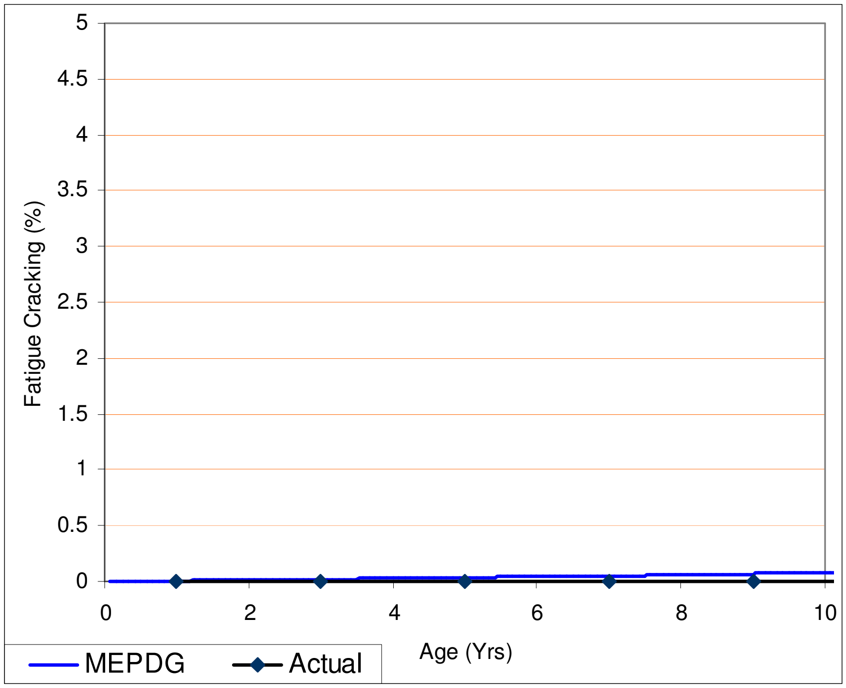


Figure 7. 2 Comparing actual fatigue cracking verses MEPDG prediction for Section ID 29581N

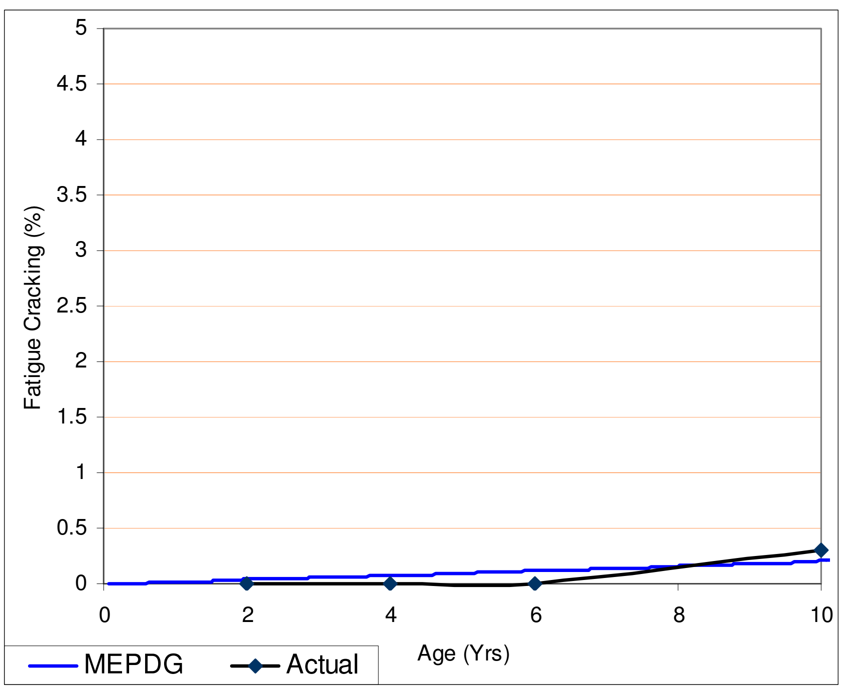


Figure 7. 3 Comparing actual fatigue cracking verses MEPDG prediction for Section ID 18890N

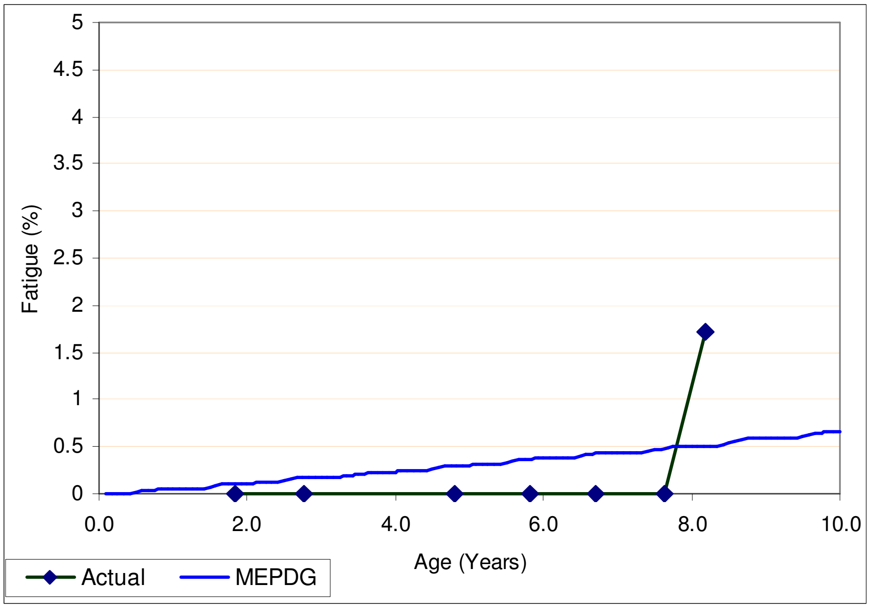


Figure 7. 4 Comparing actual fatigue cracking verses MEPDG prediction for SPS Site 117

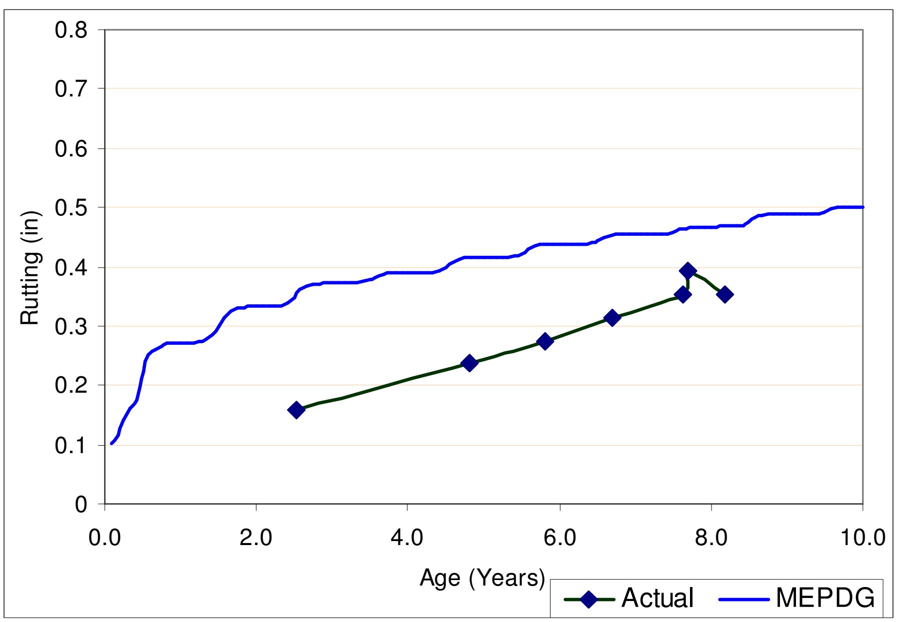


Figure 7. 5 Comparing actual rutting verses MEPDG prediction for SPS-1 Site 117

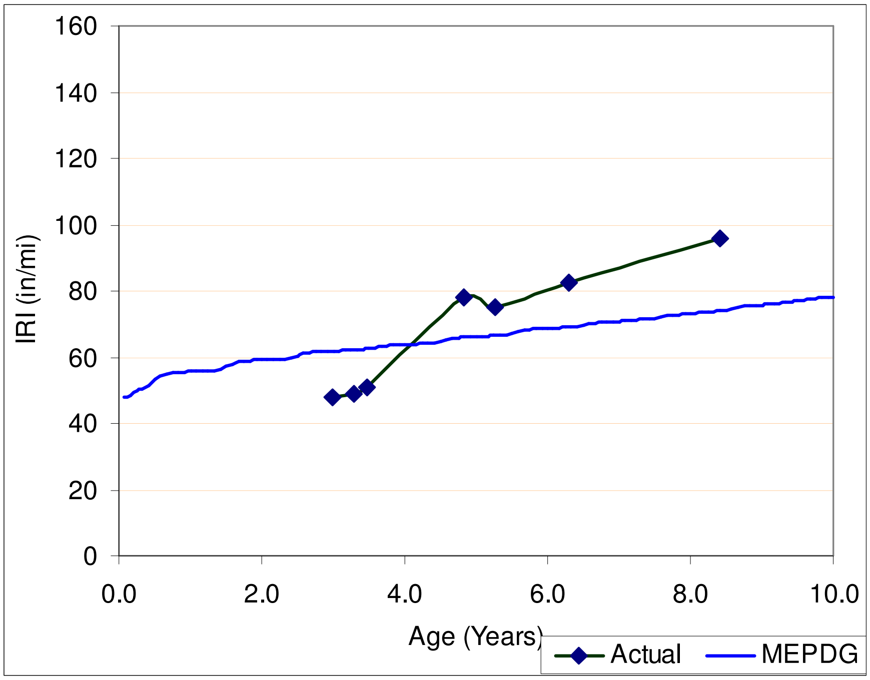


Figure 7. 6 Comparing actual IRI verses MEPDG prediction for SPS-1 Site 117

ME-PDG was run for all of the 150 cases corresponding to the three examples, and performance data was compiled. The real value of such analysis is to be able to study different scenarios and see how the current QA program is performing with respect to ensuring a quality product. Figures 7.7, 7.8 and 7.9 show predicted fatigue performance for all three cases with varying asphalt content and air voids.

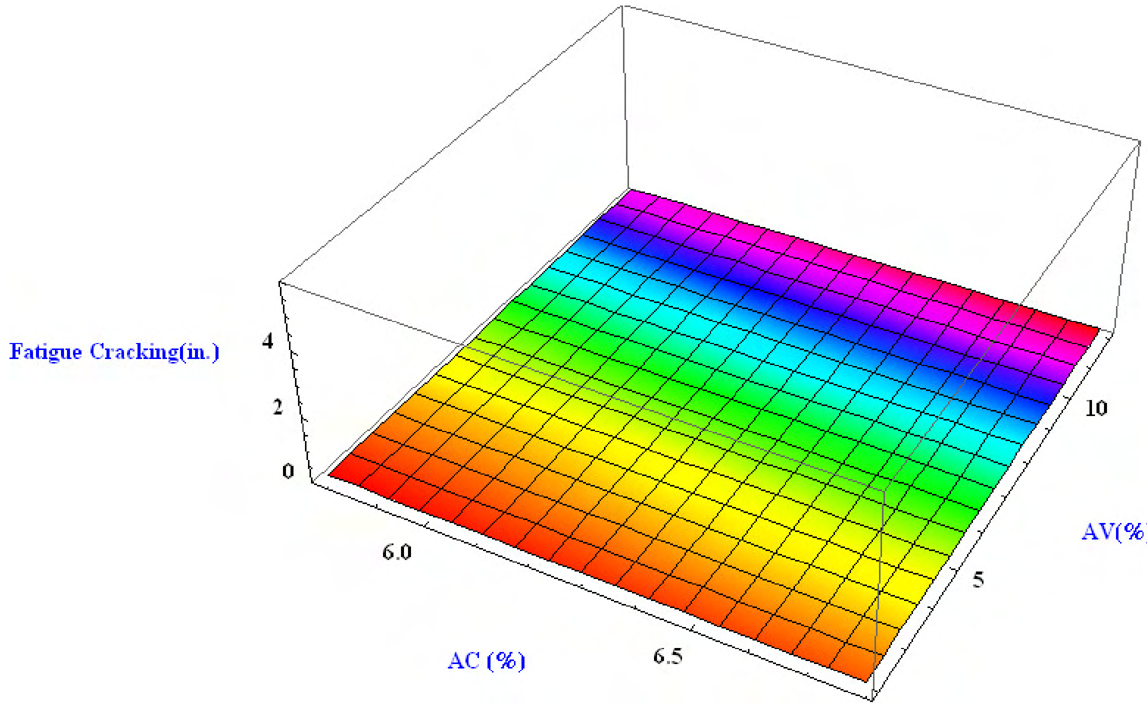


Figure 7. 7 Fatigue cracking (%) at the end of 20 years for Section ID 29581W with varying asphalt content and in-situ air voids

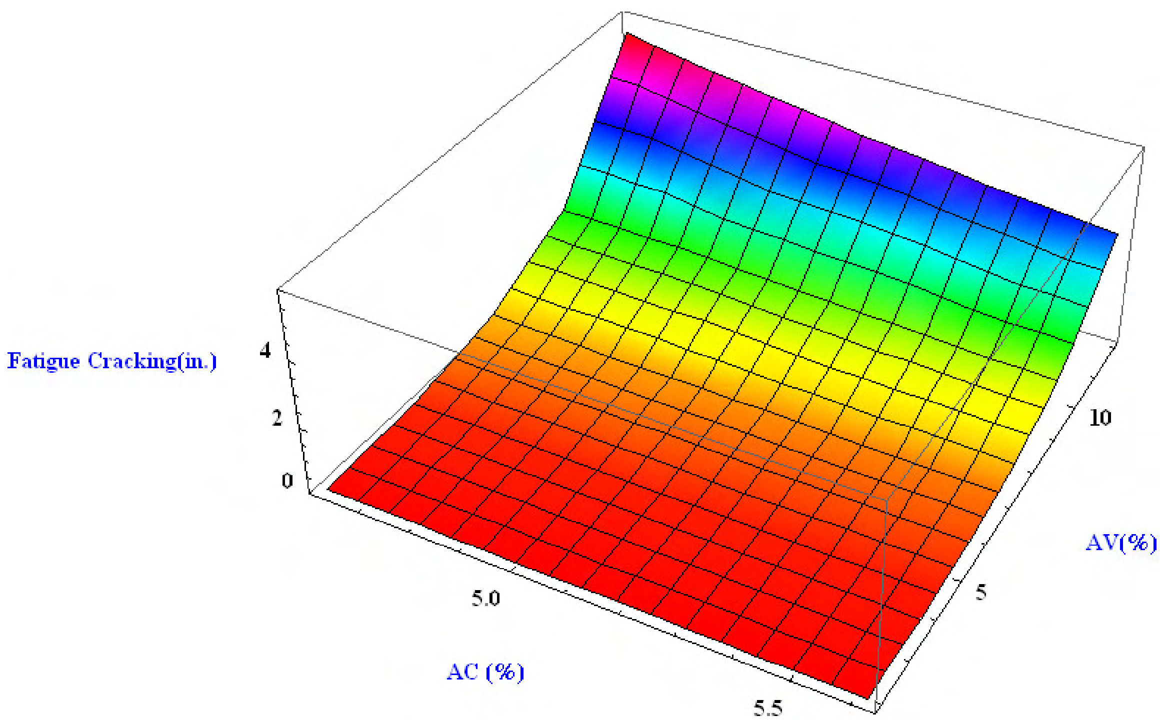


Figure 7. 8 Fatigue cracking (%) at the end of 20 years for Section ID 18890N with varying asphalt content and in-situ air voids

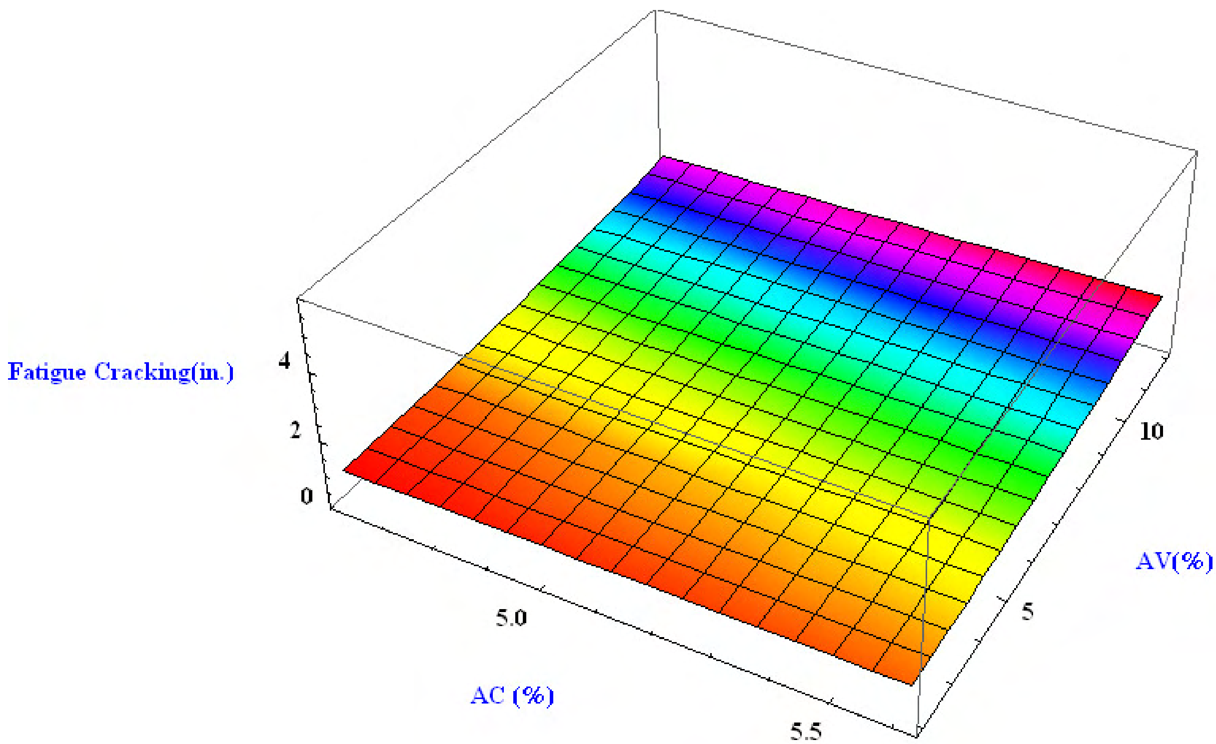


Figure 7. 9 Fatigue cracking (%) at the end of 20 years for SPS-1 site 0117 with varying asphalt content and in-situ air voids

To be able to assess how the current QA program is performing in terms of ensuring good quality, we need to present some contrasting case scenarios. For the three mixes that are being analyzed, three scenarios with respect to the quality of construction were considered. Table 7.6 presents the details of these scenarios. Note that the quality of construction is exhibited through the standard deviation; for example, while the "poor" case scenario has an acceptable air voids mean of 7% , taking into account two standard deviations from the mean, the in-situ air void will reach 10.7%, which is on the high side. It was assumed that the project would have 50 sublots.

Table 7.6 Air voids and asphalt content levels for the three scenarios for analysis

Scenario	In-situ Air Voids (%)		Asphalt Content (%)	
	Mean	Standard Deviation	Mean	Standard Deviation
Good	6	0.98	Target	0.05
Fair	6.5	1.45	Target	0.1
Poor	7	1.84	Target - 0.1	0.15

Performance curves obtained from all the MEPDG runs can be used to estimate performance for all these scenarios using interpolation methods. In this case, piecewise bi-cubic interpolation was used to accurately determine all the performance curves as required for the analysis. Figures 7.10 through 7.12 show fatigue performance for each of the sublots for all three scenarios and the three different mixes being analyzed. The mix corresponding to Section ID 29581N shows good performance even when construction quality is poor, while the same is not true for the other mixes. Also, while all the three mixes seem to perform well when the construction quality is good, the second and the third mixes start showing poor performance, or loss of service life, as construction quality becomes worse.

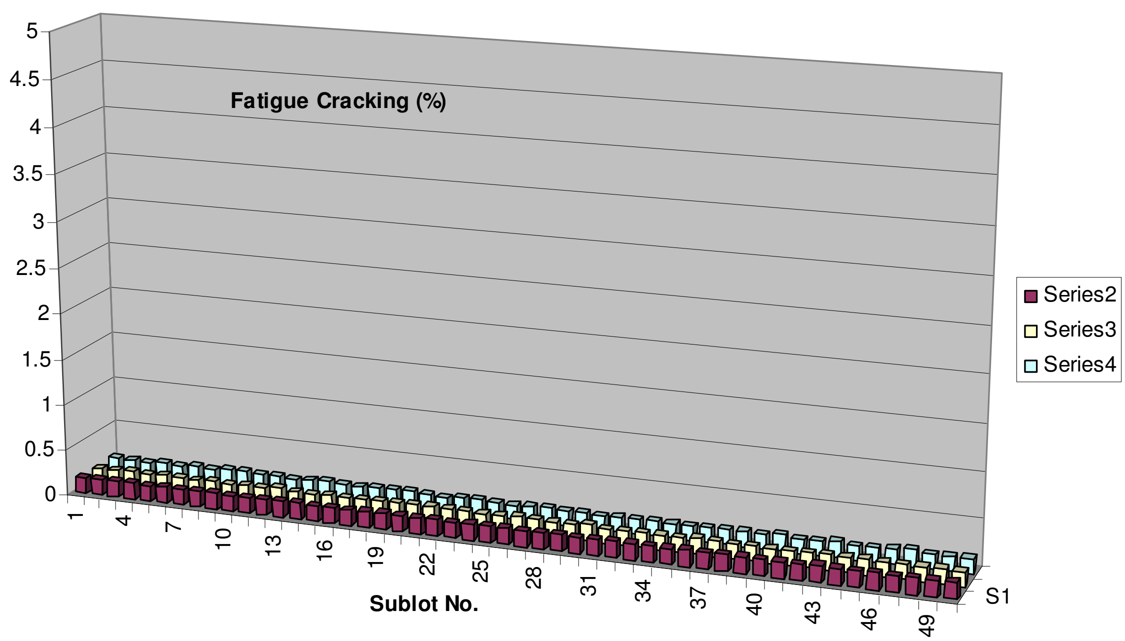


Figure 7. 10 Fatigue cracking for Section ID 29581N (Good, fair and poor)

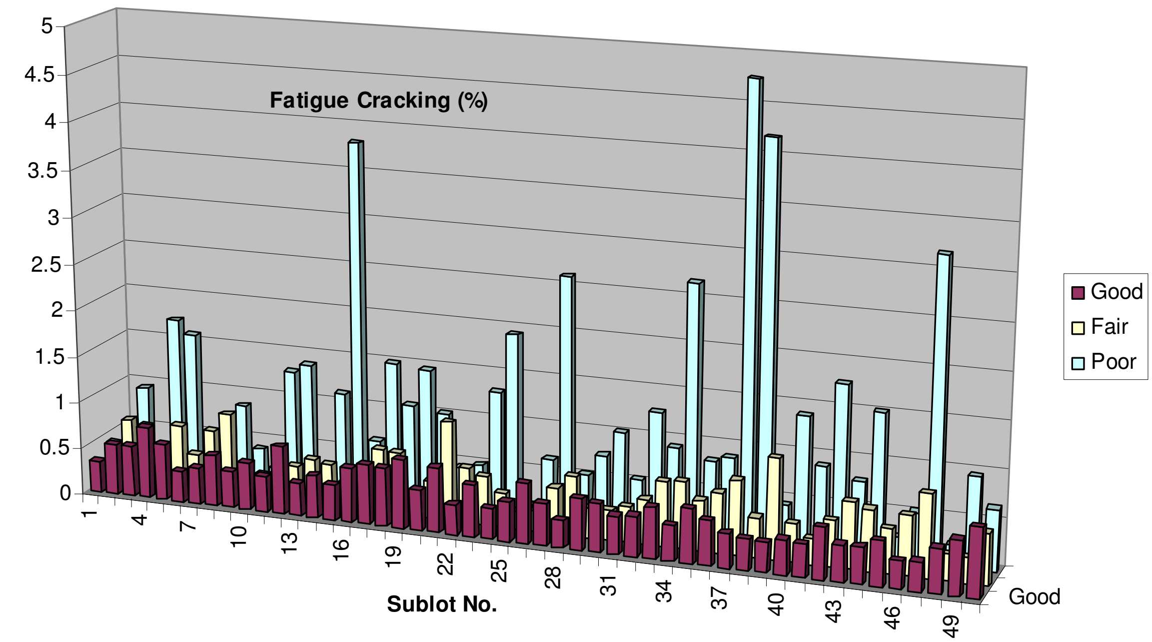


Figure 7. 11 Fatigue cracking for Section ID 18890N (Good, fair and poor)

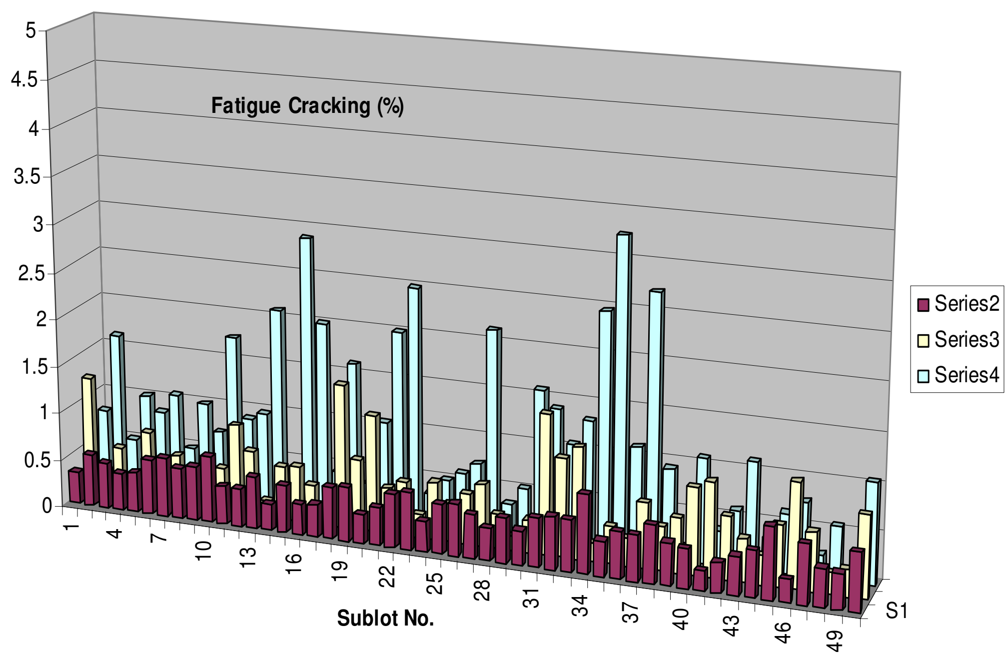


Figure 7. 12 Fatigue cracking for SPS-1 Site 117 (Good, fair and poor)

In a good QA program, properties that lead to poor long term results should translate into a lower pay factor. In the current QA program, pay factor is determined based on percent within limits achieved for plant air voids, plant VMA and in-situ density/air-voids. Table 7.7 shows the percent within limits and pay factor for density. It shows that, according to the current QA program, good construction quality in the example presented here would result in a pay bonus of 1.6%. Also, the pay factor for the fair construction is very close to 100% with a penalty of 0.3%. It also penalizes the poor construction much more severely with 4% penalty in pay.

Table 7.7 PWL and pay factor for density for the three scenarios.

Scenario	Density PWL	Density PF
Good	98	101.6
Fair	85	99.7
Poor	70	96.0

If the highway agencies were to have a reliable estimate of the performance of the pavements, they can also determine a rational QA program. Such a program would not only allow for rational payment to the contractor, it would also encourage them to achieve quality in construction that would lead to good performance. The analysis presented here is intended to show the possibility of doing this. The next step would be to perform exhaustive analysis considering all possible aspects of construction and determine their effect on performance. The following step would be to determine the change in service life as a result of construction quality.

7.3 Summary of Findings

There are three very important aspects of a QA program:

(1) The tests that are conducted within the QA program should relate to pavement performance, and sufficient and appropriate number of tests should be done to ensure quality in different aspects of mix preparation and construction process.

(2) The targets for the quality characteristics such as air voids, VMA and density should be set such that they ensure good performance. The window of variability allowed around the target should be achievable and at the same time tight enough to ensure consistency in pavement quality.

(3) The pay factor should be rational. In other words the penalty or bonus should be proportional to the gain or loss in life of the pavement as a result of good or poor construction quality.

Based on the limited number of verifications, MEPDG seems to give reasonable prediction trends for different pavement distresses. Therefore, MEPDG can be used to explore various aspects of HMA mix and construction process that influence pavement performance. However, pending to verification through other methods, other aspects of mix production or lay down that are not being accounted for in the current QA program can be identified through the strategy presented in this report. The proposed strategy can also be directly used to determine the targets and allowable windows for each of the quality characteristics being used in the QA program. MEPDG can be used to simulate the variability encountered in construction and the resulting pavement performance. A preliminary example of that has been presented in this report. When extended with possible life cycle cost analysis, such simulation process can lead to a rational pay factor formula/procedure. In the next section, several case scenarios are simulated using the MEPDG and the results are discussed.

7.4 Simulation Using MEPDG

The MEPDG includes many different models for predicting pavement properties and performance. This makes it possible to study the effect of different quality characteristics on pavement performance. Also, the effect of different input variables or quality characteristics can be studied together rather than individually because there is an interaction between their effects on performance. The models used in the MEPDG, like any other mechanistic-empirical model, have limitations and inaccuracies associated with them. This is more or less expected as we are trying to model natural materials which vary by their location and with changing environment. However, it should also be accepted that these models are the best that the pavement community can put together at this time.

There is a substantial amount of data in the LTPP database for different categories of input variables as well as performance from survey on real life pavements. However, any construction project is unique because of a variety of factors affecting it. For example, different construction projects would have different construction crews, different climatic conditions during the days of construction, different brands of equipment used, including the paver, different material transportation and discharge practices and conditions, different oversight managers etc. Also, during the life of the pavement one may have some unique factors influencing it like a traffic pattern which is not common to other pavements. The end result is that when distress survey data is collected from such project, the data is a result of *many* factors that it is almost impossible to separate their individual effects. Therefore, it is difficult to account for all the factors even in very carefully controlled road tests like the AASHO road test or Westrack.

Although many factors affect pavement performance, not all of them can be controlled at the time of construction, even though they do affect pavement performance. This can be easily known in those cases where quality control is very poor and the pavement develops premature distresses. However, in the majority of the pavements constructed, the quality control is fair to excellent. It is important to study how relatively minor changes in quality affect service life or performance of the pavement. MEPDG lends itself well to such analysis even though such simulation results are indicative rather than predictive. This section presents analysis performed using MEPDG to study the influence of QA variables, including plant air voids, in-situ density and asphalt content, on HMA pavement performance. Similar analysis on PCC pavements is also conducted and discussed later on.

In the previous sections, some analysis was performed where the effects of individual input variables on performance were studied. In this section, several case scenarios were simulated using MEPDG, and statistical analysis on the corresponding results are presented.

As stated earlier, the LTPP database has mean and standard deviation values for input variables like in-situ density and asphalt content along with the number of samples used to determine standard deviation, although individual data used for the calculation are not documented. Statistical methods were used to generate simulated values of in-situ air voids and asphalt content which would have the same means and standard deviations as those reported in the LTPP database. This was done by writing a computer program in Matlab. It is assumed that since these values simulate the LTPP means and standard deviations, they would also be representative of actual projects under a QA program.

The next step was to input each scenario in MEPDG and determine expected pavement performance in terms of fatigue cracking, longitudinal cracking, rutting etc. To get an appreciation for what this analysis entails, let us take the example of data plotted in Figure 7.13. Each data point in the plot represents the mean fatigue cracking value for one project. Each project has a number of samples tested for determining mean and standard deviation of in-situ density and asphalt content. Combining the number of samples from all the projects plotted in the figure for density and asphalt content, the total number of MEPDG runs required would be 7,884. Each run using MEPDG to predict distresses for 30 years would take an average of 50 minutes. This would mean that the total time required for carrying out all the runs for a single plot would be 273 days, assuming that there are no errors or crashes during the execution of the runs. For all practical purposes, it is almost impossible to execute this task. We present below an alternative, which can make this task much more efficient.

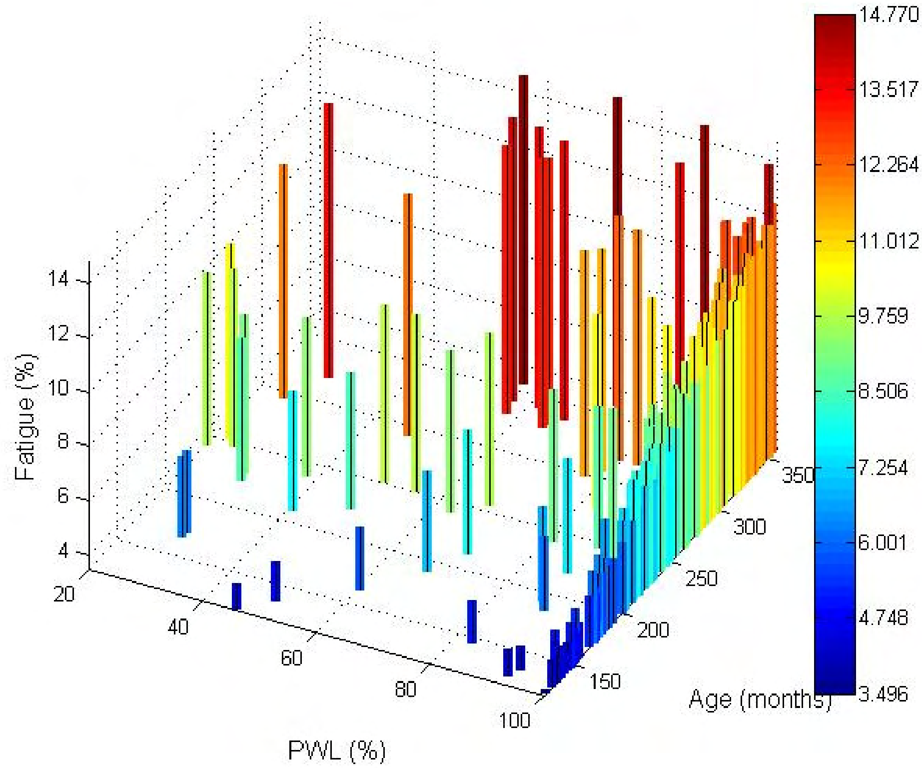


Figure 7. 13 Fatigue cracking Vs PWL (in-situ air voids) for mix 1

The alternative, in principle, is to run MEPDG to develop response surfaces and use these surfaces to determine performance for all 7,884 case scenarios required for one plot. For developing the response surface, the entire range of in-situ density and asphalt content should be identified. Then some points in between the maximum and minimum values should be identified. In this analysis, we identified 3 more values between the extremes, making for a total of 5 values each for density and asphalt content. Then a full factorial matrix is defined using all possible combinations of the values of asphalt content and density. This would make for 5*5= 25 runs. In order to keep the number of simulation runs manageable, we identified two different mixtures and input all of their characteristics in MEPDG. Then all 25 runs were executed for each of the mixtures. The results from the 50 runs were compiled based on performance category (fatigue, rutting, etc.). All the 25 cases defined here were real combinations of input variables derived using MDOT designs. One can then imagine a response surface where the four dimensions represent density, asphalt content, age and fatigue (or rutting etc.). Such response surfaces were created in Matlab.

Each of the required 7,884 runs of MEPDG correspond to one combination of density and asphalt content, and the response is obtained for varying ages in each case. The response surface created in the analysis actually contains each of these points as long as density and asphalt content fall within the range identified for running the 25 cases with MEPDG. Therefore, using the piecewise cubic spline interpolation technique in four dimensions the various performances (fatigue, rutting, etc.) were generated for all of the 7,884 cases. This was done using the MatLab computer program. Then, the performance (e.g., fatigue) corresponding to each project was calculated by averaging all the values corresponding to different samples.

Piecewise cubic spline interpolation technique was chosen in this case because it fits an n-dimensional cubic surface locally to each portion of the surface while maintaining continuity in magnitude and slope of the surface with the neighboring areas within the surface in all dimensions. This makes this technique extremely versatile to be used on diverse types of surfaces while maintaining excellent accuracy all across the surface. Regression or model fitting can lead to appreciable errors in certain ranges of input variables especially when it has to be fit in more than two dimensions.

7.4.1 Effect of varying air void distribution on average project performance

First, we present the results corresponding to the cases of varying in-situ air voids to achieve various PWL ranges, while fixing the asphalt content (at -0.4% of optimum). Figure 7.13 shows mean fatigue cracking estimated using the response surface for different case scenarios corresponding to data available in LTPP database as well as using additional input data to cover lower PWL ranges more uniformly. These cases were added to get a better picture of what happens when PWL is lower than 90%. The plot shows the expected trend of higher fatigue cracking with increasing age of the pavement. It is also clear that as PWL (in-situ air voids/density) increases the projects have lower expected fatigue cracking for the same age. To get a better appreciation of the magnitude of difference in cracking we estimated fatigue cracking at the end of 30 years for all the cases. The results are shown in Figure 7.14. The maximum difference in fatigue is only about 2% between the worst PWL and the acceptable PWL (greater than 90%); a negligible difference.

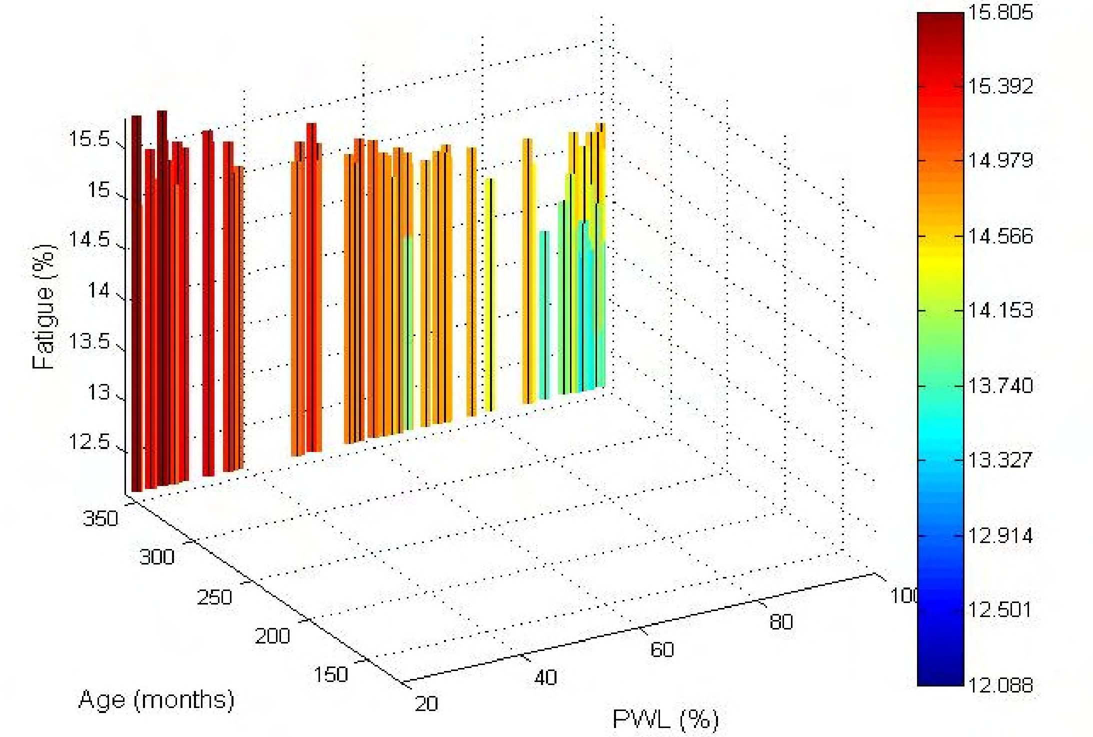


Figure 7. 14 Fatigue at 30 years Vs PWL (in-situ air voids) for mix 1

It is also noticeable that for very similar PWL values, different projects show different amounts of fatigue cracking. This is because fatigue cracking in each project is estimated by sampling. As long as the range, mean and standard deviation of two samples are the same they would have the same PWL. But the average fatigue cracking can vary depending where each of the samples falls within the specifications. For example, if a project has all densities equal to 93%, PWL would be 100%. If the density of all the samples were 98%, still PWL would be 100%. However, fatigue performance for the 98% density project would be much better than that for the project with mean density of 93%.

Rutting performance estimated through the simulation is shown in Figure 7.15. This figure shows that rutting performance improves as PWL approaches 100%. However, this improvement is very small. In other words, rutting performance is not so sensitive to PWL (in-situ air voids) according to MEPDG results. Figure 7.16 shows the same result with IRI. The plot shows that IRI also does not seem to be getting much affected by PWL (in-situ air voids).

It must be mentioned that all these results are for one type of mix. It is important to study different types of mixes to assess how PWL values affect performance for each one of them. It is quite possible that this effect may be much more pronounced in other types of mixes.

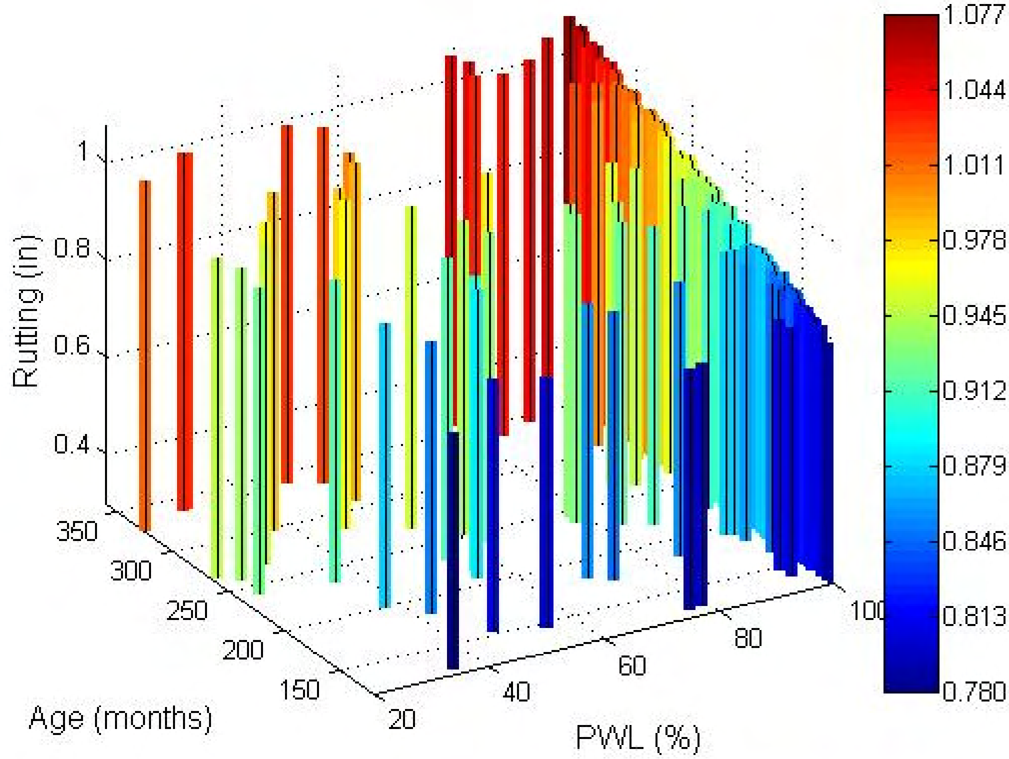


Figure 7. 15 Rutting Vs PWL (in-situ air voids) for mix 1

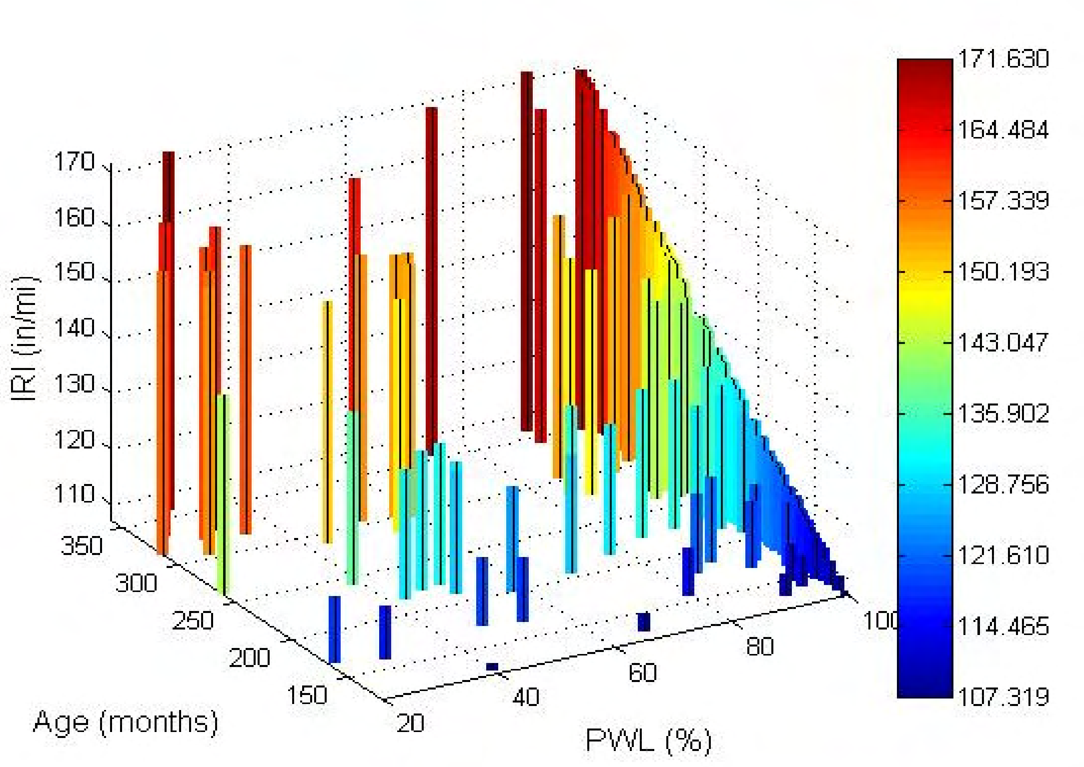


Figure 7. 16 IRI Vs PWL (in-situ air voids) for mix 1

7.4.2 Effect of varying both air void distribution and asphalt content on sublot project performance by sublot

The next exercise is to explore the effects of varying air voids and asphalt content together. This means that different sublots in the same project have varying air void and asphalt content. The previous analysis showed average performance for a given project. The observed effects of PWL on performance were not as pronounced as one would expect intuitively. The main reason for this, as will be shown in this section, is the attenuation of distress levels because of the averaging of performance across sublots. Therefore, the results presented in this separate analysis are shown in terms of performance by individual sublots. Figures 7.17 through 7.21 show the fatigue results for various scenarios (three air void and five asphalt content distributions). The figures show that:

1. performance of different sublots within a project can vary significantly;
2. within a given asphalt content range, there are more underperforming sublots when the air void distribution moves towards the upper specification limit ;

3. as the asphalt content range moves dry-of-optimum, more sublots are underperforming in fatigue, even at relatively high combined PWL [for example, compare Figures 7.17(a) through 7.21(a)].
4. as the asphalt content range moves wet-of-optimum, more sublots perform better in fatigue, even at relatively low combined PWL [for example, compare Figures 7.17(c) through 7.21(c)]. This may however lead to more rutting within the HMA layer.

7.5 Performance criteria

In the previous section, results from multi-factor analysis were presented for flexible pavements. The results showed how certain factors affect pavement performance. The results were presented in the form of comparative plots. However, Task 7 of this project would require firm decisions to be made whether a candidate QA variable should be included in the list of important QA variables. This section presents an objective strategy that can be used to make such decisions

Many different criteria were considered which could possibly be used to compare different project scenarios and make a decision whether a certain candidate QA variable is important for a QA program. Sets of possible case scenarios were developed to test these criteria. The results from these case scenarios would also give deeper insight into the workings of percent-within-limits statistical approach used in QA programs by the Michigan Department of Transportation and several other states. Each scenario represents one flexible pavement project with 50 sublots. Each of the sublots would have varying levels of in-situ air voids and asphalt content. Table 7.8 shows the asphalt content and in-situ air void levels for the 25 scenarios. This forms one set of scenarios. Three such sets were planned for this study which would have different levels of variability associated with asphalt content and air voids. Table 7.9 presents the variability for the three sets.

(a)

(b)

(c)

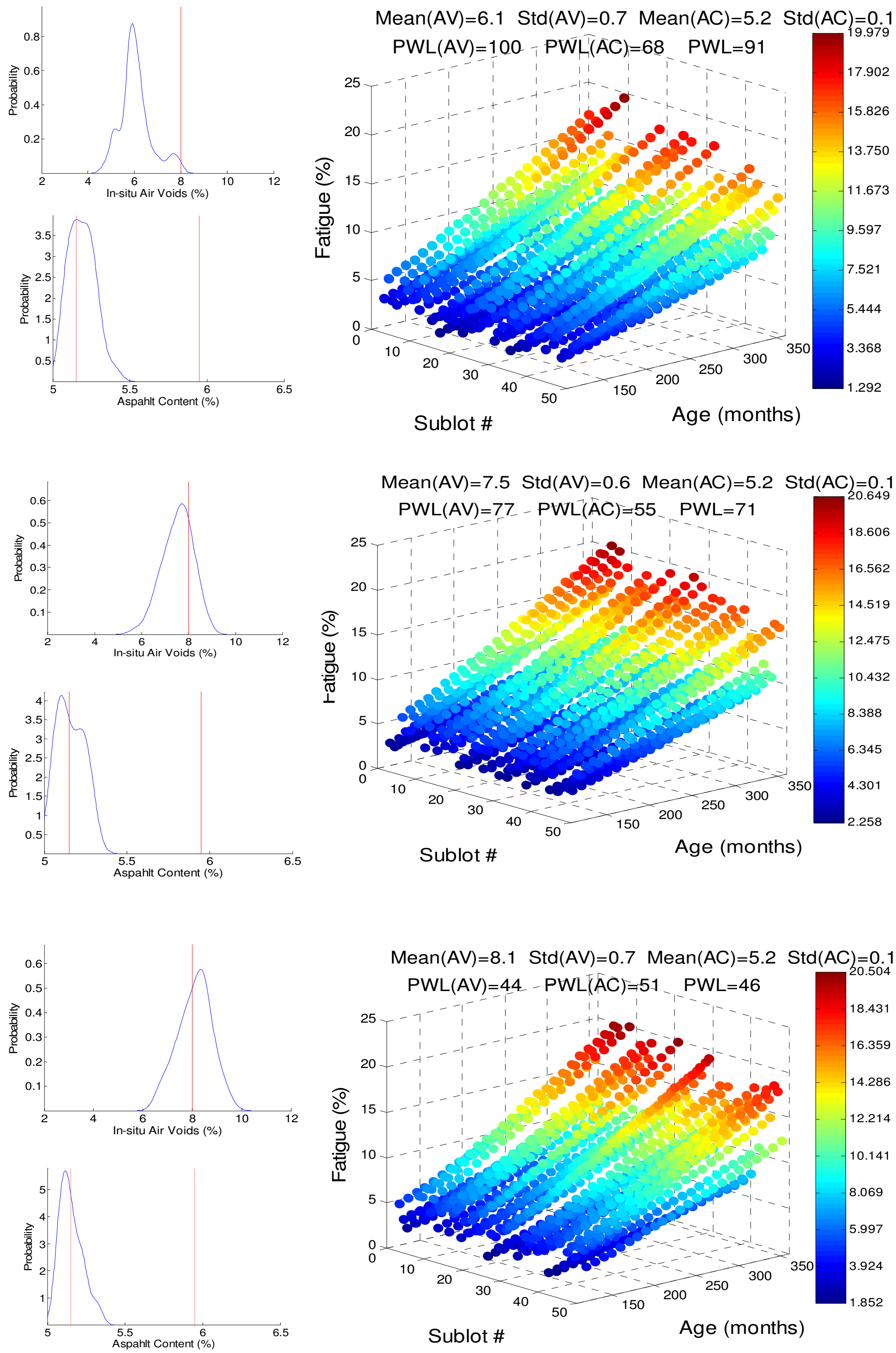


Figure 7. 17 Effect of AV distribution on fatigue performance of sublots for AC=Opt-0.4.

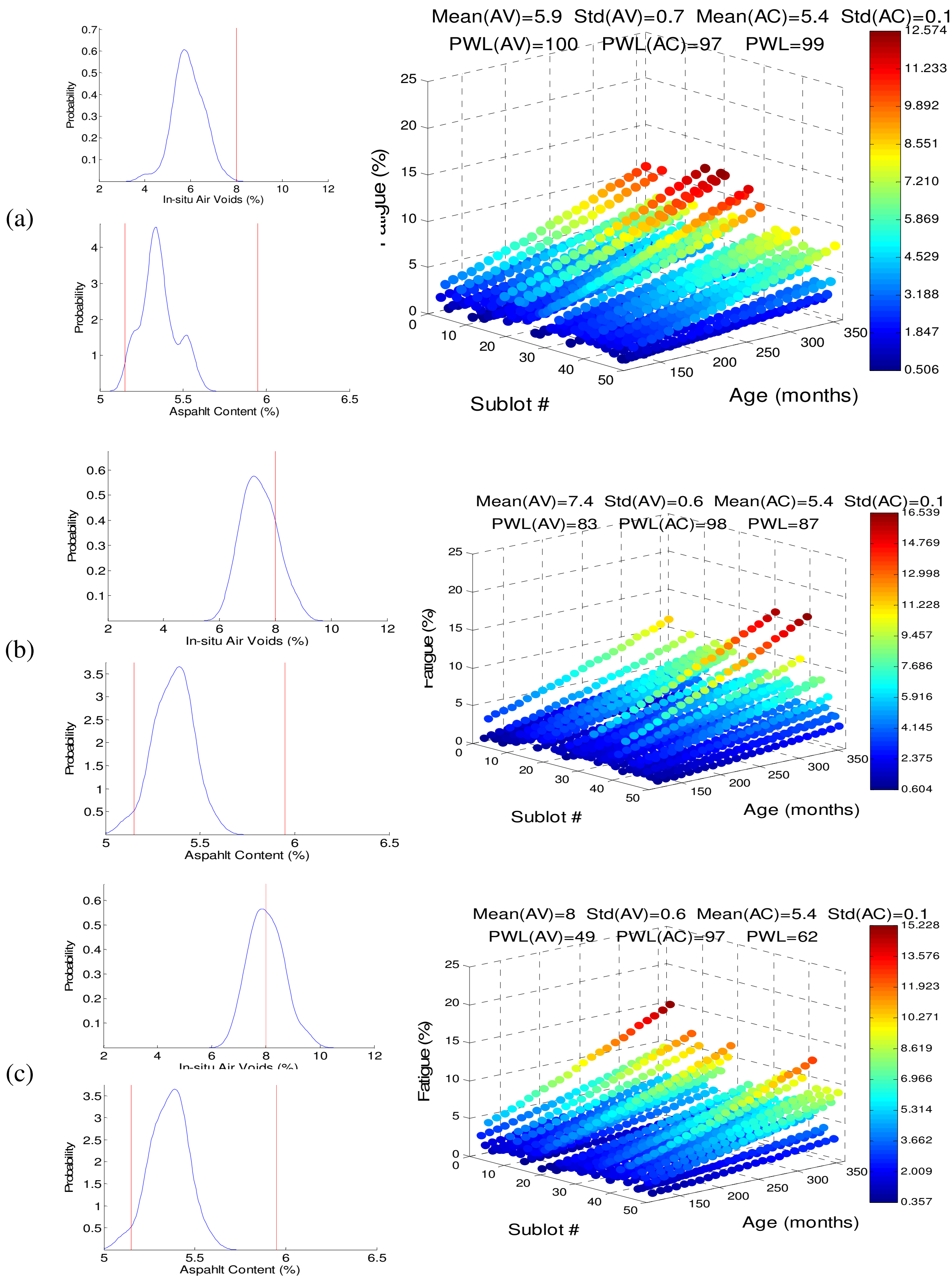


Figure 7. 18 Effect of AV distribution on fatigue performance of sublots for AC=Opt-0.2.

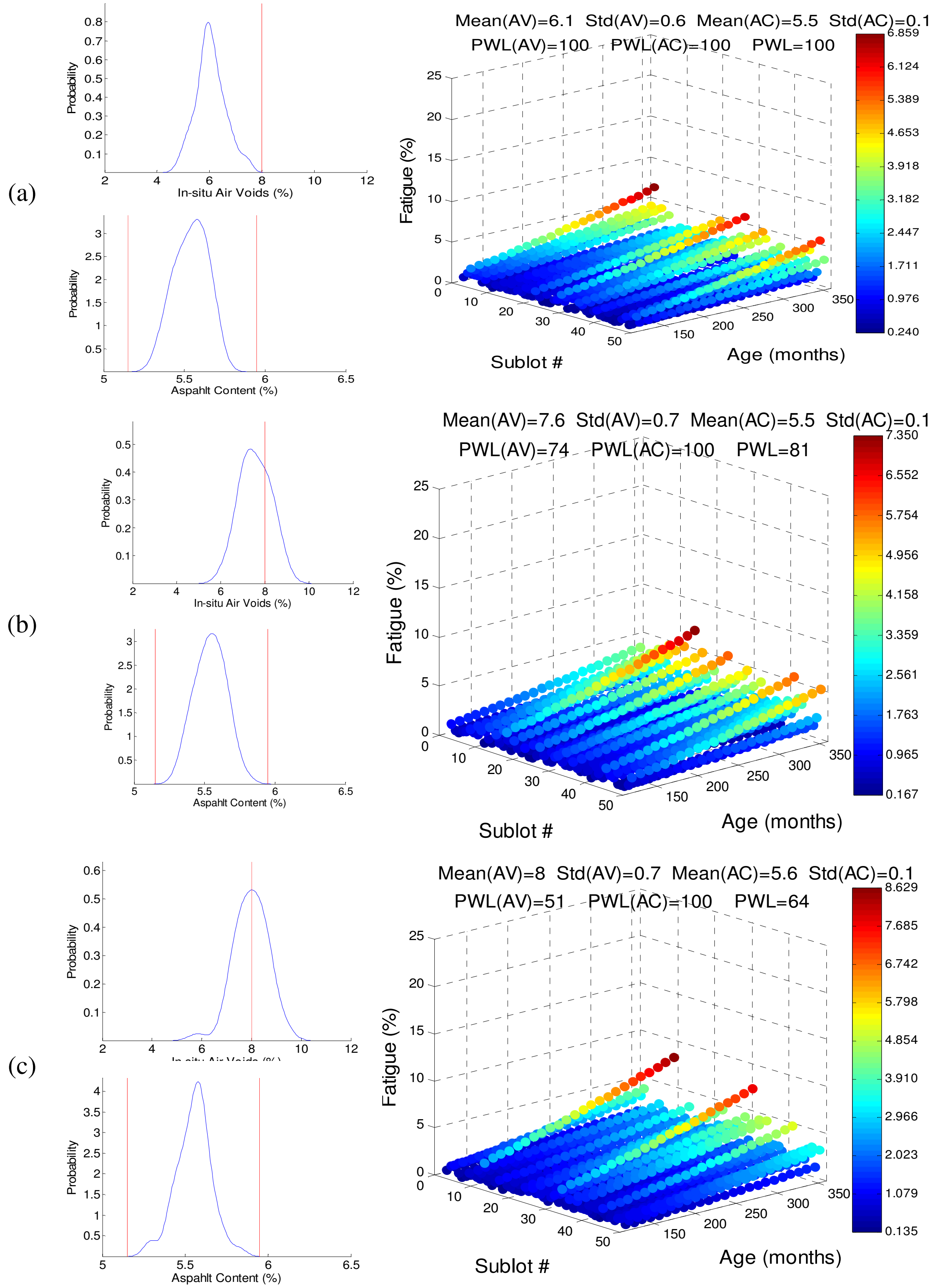


Figure 7. 19 Effect of AV distribution on fatigue performance of sublots for AC=Opt.

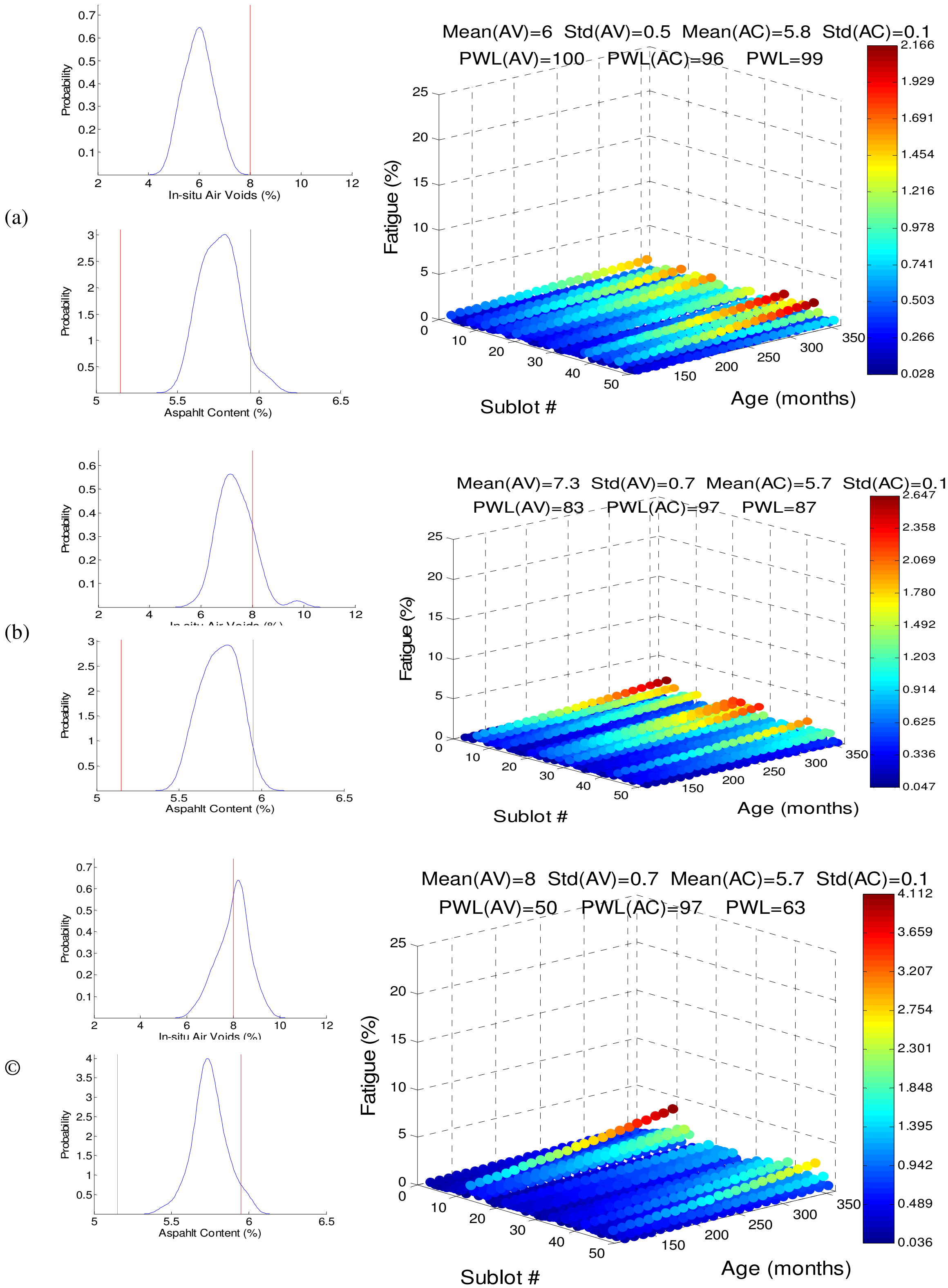


Figure 7. 20 Effect of AV distribution on fatigue performance of sublots for AC=Opt+0.2.

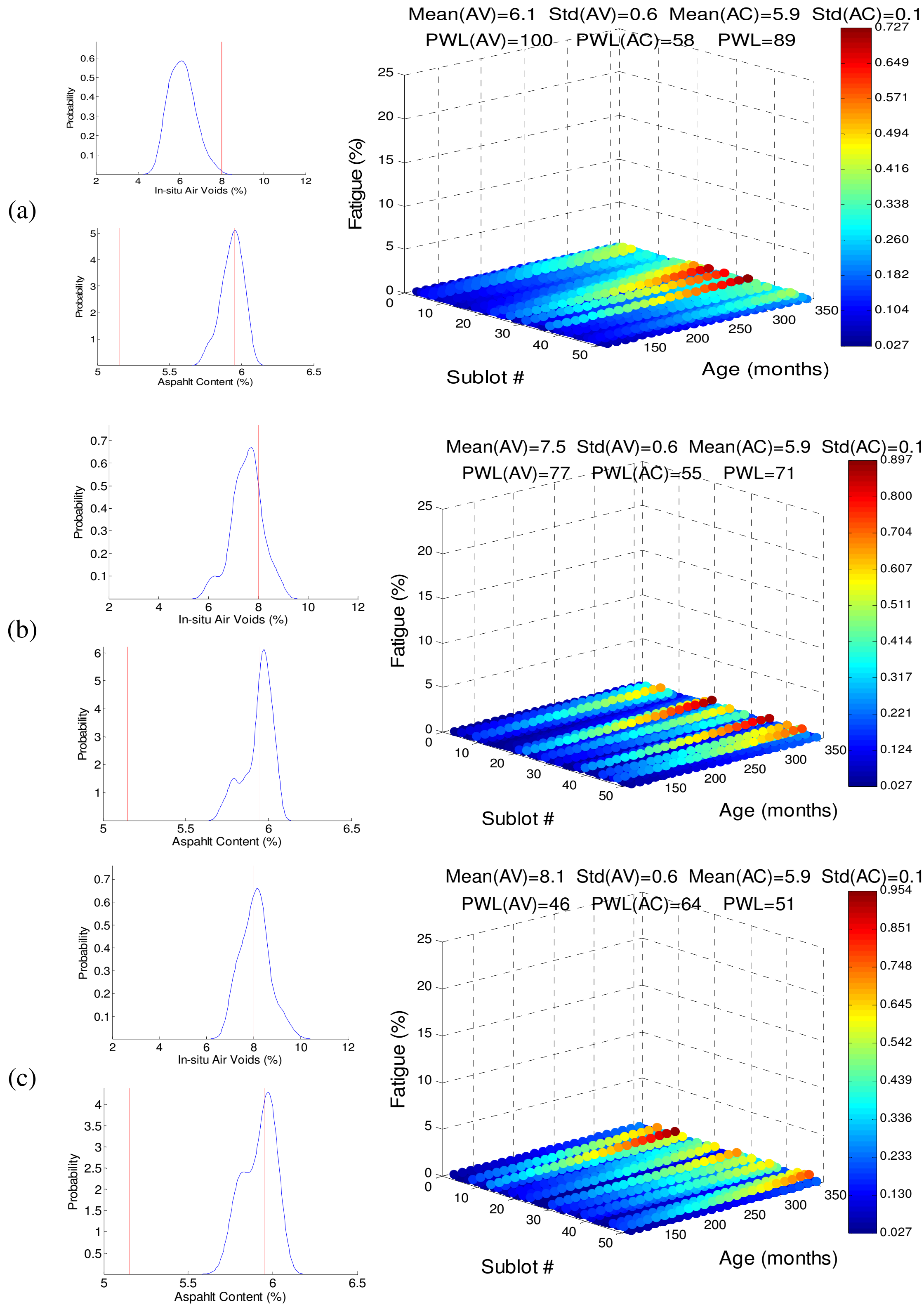


Figure 7. 21 Effect of AV distribution on fatigue performance of sublots for AC=Opt+0.4

Therefore, a total of 75 case scenarios were studied, each having 50 simulated sublots. This equates to running MEPDG software 3750 times. However, instead of running MEPDG, the strategy involving response surfaces was used. The project simulated in this study was a flexible pavement constructed in Ludington, MI in 1988. This pavement has 7.5 inch asphalt concrete layer over 4 inches thick base and 18 inches thick subbase.

Table 7.8 Asphalt content and air voids levels for the case scenarios

Run Number	Asphalt Content (%)	In-situ Air Voids (%)
1	Optimum-0.4	3.5
2		6.0
3		7.5
4		8.0
5		9.5
6	Optimum-0.2	3.5
7		6.0
8		7.5
9		8.0
10		9.5
11	Optimum	3.5
12		6.0
13		7.5
14		8.0
15		9.5
16	Optimum+0.2	3.5
17		6.0
18		7.5
19		8.0
20		9.5
21	Optimum+0.4	3.5
22		6.0
23		7.5
24		8.0
25		9.5

Table 7.9 Standard deviations for asphalt content and in-situ air voids for the three sets of scenarios

Set Number	Standard Deviation (AC) (%)	Standard Deviation (AV) (%)
1	0.10	0.75
2	0.15	1.00
3	0.15	1.20

Different criteria considered are enumerated below. Each project represents a set of 50 sublots with fixed means and standard deviations for air voids and asphalt content.

(i) Average distress (for example, fatigue cracking) at the end of 30 years for the 50 sublots

(ii) Average distress for the sublots considering distresses for each of the months during 30 year service life

(iii) Area beneath the curve showing distress at the end of 30 years versus sublot number

(iv) Total area beneath all the distress curves for each project

(v) 50th, 75th, 90th and 95th percentile of distresses at the end of 30 years for each project.

(vi) Area beneath the distress curve when distress is raised to a certain power. This was done to especially identify those scenarios which lead to unacceptably high distresses in even a few sublots within a project.

Tables 7.10 through 7.12 present the average fatigue and 75th and 90th percentile for different projects which represent different realistic scenarios. The rest of the criteria as mentioned in the list above were studied in a similar way for all of the scenarios, although the details are not being included in the report for the sake of brevity. Comparing all the above criteria it was found that average fatigue cracking after 30 years combined with 75th or 90th percentile are good to compare the projects (or scenarios).

Some of the rows in Table 7.10 have been highlighted. These cases have very similar combined percent-within-limits (PWL) values. Combined PWL is generally used directly to calculate payment for the contractor, which ideally is in accordance with the quality of the pavement constructed by him. Although the PWL values are very similar in these cases the average fatigue cracking at 30 years varies from 0.69% to 6.76%. This much of variation is very significant because they would be categorized as being in very good and nearly poor condition respectively. The 75th percentile value for the same cases varies from 0.83% to 8.68% and the 95th percentile varies from 1.1% to 9.86%. This clearly shows that the candidate QA variable causing this variation, asphalt content in this case, has significant influence on fatigue performance.

Variations in fatigue performance as a result of change in in-situ air voids in these cases is also appreciable, although not as large as that because of asphalt content. When the air voids increase from 3.52% to 7.97% between run number 1 and 4, the 75th percentile for fatigue cracking goes up from 13.9% to 17.87%. An advantage of percentile distresses, as used here, is that it can possibly identify those cases where a few sublots perform very poorly while rest of the sublots may have acceptable performance. Such a scenario should not be acceptable because the distressed sublots may force early repair work for the entire project.

Table 7.10 PWL values and fatigue cracking for the 1st set of MEPDG runs for performance criteria

Mean AV	Mean AC	Run No.	PWL(AV)	PWL(AC)	PWL Combined	Fatigue Avg. (%)	Fatigue 75th Percentile	Fatigue 90th Percentile
3.52	5.17	1	100.0	59.6	89.0	11.81	13.90	15.54
5.85	5.17	2	100.0	58.6	88.7	13.17	15.86	16.94
7.48	5.16	3	79.4	55.0	72.7	14.11	16.05	17.67
7.97	5.16	4	52.2	56.3	53.3	14.25	17.87	19.65
9.50	5.14	5	1.1	42.7	12.4	16.82	19.00	20.68
3.62	5.32	6	100.0	95.3	98.7	6.76	8.68	9.86
6.07	5.35	7	100.0	98.9	99.7	6.64	8.39	9.67
7.46	5.35	8	80.7	97.3	85.2	7.09	9.06	11.66
8.09	5.36	9	43.9	98.9	58.9	6.74	7.83	11.21
9.38	5.34	10	0.7	97.8	27.2	8.05	9.44	13.30
3.41	5.57	11	100.0	99.8	100.0	2.33	2.95	4.24
5.95	5.54	12	99.8	100.0	99.8	2.83	3.48	4.87
7.46	5.54	13	83.4	100.0	87.9	2.96	3.53	5.02
8.06	5.57	14	46.4	99.9	61.0	2.66	3.29	4.91
9.57	5.57	15	0.5	100.0	27.6	2.76	3.54	4.81
3.60	5.77	16	100.0	98.6	99.6	0.69	0.83	1.10
6.20	5.76	17	99.6	98.7	99.3	0.83	1.00	1.38
7.40	5.75	18	83.7	96.5	87.2	0.99	1.31	1.70
7.97	5.74	19	51.7	98.7	64.5	1.05	1.27	1.68
9.63	5.75	20	0.5	96.8	26.8	1.06	1.21	1.59
3.64	5.93	21	100.0	57.8	88.5	0.27	0.30	0.50
6.00	5.94	22	99.8	56.9	88.1	0.30	0.37	0.44
7.48	5.94	23	80.8	55.6	74.0	0.28	0.33	0.54
8.23	5.92	24	37.0	63.0	44.1	0.33	0.42	0.66
9.48	5.94	25	0.4	53.8	15.0	0.33	0.36	0.61

Tables 7.11 and 7.12 present the fatigue cracking results obtained for the 2nd and 3rd sets of scenarios, and show similar trends. Tables 7.13 through 7.15 present the rutting results for different scenarios. The range of rutting values corresponding to the average and 75th and 90th percentiles for all the cases is rather small. It should be noted that in these runs the asphalt content was allowed to vary only up to 0.5% more than the optimal. If higher asphalt contents were allowed, the increase in rutting would probably have been more significant. However, the

selected criteria, namely the average distress at 30 years, rutting in this case, and 75th and 90th percentiles are still good indicators of the effect of QA variables on performance.

The results also show that even though different projects may have very similar combined PWL values they may perform differently in reality. However, this does not necessarily mean that PWL methodology does not work. Different factors may have relatively different influences on performance and they may affect each others' effect as well. It is the goal for a good QA program to include this interaction when calculating the combined PWL for the entire project.

Table 7.11 PWL values and fatigue cracking for the 2nd set of MEPDG runs for performance criteria

Mean AV	Mean AC	Run No.	PWL av	PWL ac	PWL comb	Avg. Fatigue (%)	75th Percentile	90th Percentile
3.45	5.21	1	100.0	73.3	92.7	10.24	13.42	15.32
6.26	5.17	2	95.8	56.5	85.1	13.59	17.22	18.69
7.40	5.17	3	71.9	59.6	68.6	13.66	16.77	18.63
7.91	5.18	4	53.3	59.6	55.0	13.88	17.60	19.04
9.59	5.16	5	5.9	53.8	19.0	16.67	19.78	21.01
3.55	5.35	6	100.0	91.1	97.6	6.50	8.62	12.66
5.97	5.34	7	99.5	88.5	96.5	7.65	10.56	14.03
7.52	5.38	8	71.4	93.5	77.5	6.70	7.62	12.85
8.03	5.36	9	48.7	94.0	61.1	7.21	9.63	12.57
9.71	5.36	10	6.8	91.4	29.9	9.27	12.81	15.63
3.33	5.54	11	100.0	99.4	99.8	2.73	3.89	5.24
6.14	5.57	12	98.7	99.1	98.8	2.71	3.17	5.25
7.46	5.53	13	73.5	99.2	80.5	3.53	4.73	5.95
8.07	5.55	14	47.5	99.9	61.8	3.08	3.82	5.57
9.57	5.58	15	6.6	98.2	31.6	3.29	4.16	6.06
3.52	5.74	16	100.0	92.1	97.8	1.02	1.33	1.95
6.04	5.72	17	97.8	92.5	96.4	1.33	1.53	3.29
7.40	5.76	18	71.0	88.5	75.7	1.09	1.52	2.23
8.05	5.74	19	48.2	90.9	59.9	1.29	1.50	2.77
9.61	5.72	20	6.0	94.7	30.2	1.43	1.70	2.17
3.70	5.91	21	100.0	63.6	90.1	0.34	0.40	0.69
5.87	5.93	22	99.0	58.1	87.8	0.36	0.44	0.80
7.47	5.92	23	73.1	60.0	69.5	0.38	0.38	1.09
8.17	5.91	24	42.9	61.8	48.1	0.50	0.41	0.87
9.56	5.90	25	5.3	65.2	21.7	0.60	0.65	1.09

Table 7.12 PWL values and fatigue cracking for the 3rd set of MEPDG runs for performance criteria

Mean AV	Mean AC	Run No.	PWL av	PWL ac	PWL comb	Avg. Fatigue (%)	75th Percentile	90th Percentile
3.19	5.19	1	99.9	65.6	90.5	10.86	13.75	15.51
5.85	5.21	2	97.1	70.8	89.9	11.57	14.67	16.67
7.32	5.17	3	68.7	56.8	65.4	13.98	17.12	18.83
7.99	5.18	4	50.4	63.2	53.9	13.71	16.69	19.75
9.09	5.22	5	19.1	71.5	33.4	13.59	18.89	21.17
3.25	5.34	6	99.9	88.5	96.8	6.64	8.26	11.77
6.17	5.34	7	94.0	92.7	93.7	7.31	9.00	12.17
7.12	5.35	8	73.7	94.1	79.3	7.33	9.44	13.11
8.37	5.37	9	38.6	94.4	53.8	7.25	8.77	11.82
9.54	5.33	10	11.8	90.0	33.1	10.25	14.84	17.70
3.17	5.53	11	100.0	99.4	99.8	2.85	4.03	5.02
6.22	5.59	12	91.6	99.8	93.8	2.39	2.73	5.07
7.58	5.55	13	64.5	97.8	73.6	3.33	4.65	7.23
7.72	5.56	14	60.8	99.9	71.4	2.89	3.60	5.46
9.44	5.59	15	11.5	98.5	35.3	2.98	3.16	5.85
3.27	5.75	16	99.9	90.6	97.3	0.97	1.12	2.15
5.62	5.76	17	97.6	90.5	95.7	1.05	1.33	2.08
7.54	5.75	18	68.3	91.5	74.6	1.10	1.46	1.96
7.91	5.72	19	52.8	93.8	64.0	1.38	2.02	2.83
9.37	5.74	20	15.7	91.3	36.3	1.37	1.53	2.62
3.29	5.91	21	100.0	61.1	89.4	0.36	0.35	0.70
5.83	5.93	22	95.0	59.9	85.4	0.33	0.36	0.52
7.21	5.91	23	72.6	64.5	70.4	0.40	0.41	0.82
8.09	5.92	24	47.1	59.9	50.6	0.36	0.40	0.75
9.41	5.89	25	11.1	69.4	27.0	0.55	0.68	1.15

Table 7.13 PWL values and rutting for the 1st set of MEPDG runs for performance criteria

Mean AV	Mean AC	Run No.	PWL av	PWL ac	PWL comb	Avg. Rut (in)	h ntile	90th Percentile
3.49	5.17	1	100.0	60.8	89.3	1.06	1.08	1.10
6.15	5.17	2	99.9	61.8	89.5	1.04	1.07	1.09
7.43	5.17	3	77.4	60.7	72.9	1.04	1.07	1.08
7.95	5.17	4	53.3	61.7	55.6	1.03	1.06	1.08
9.54	5.18	5	0.3	64.7	17.9	1.02	1.05	1.07
3.43	5.34	6	100.0	97.5	99.3	0.98	1.01	1.03
6.07	5.36	7	99.9	97.1	99.1	0.96	0.98	1.03
7.47	5.32	8	83.0	95.4	86.4	0.97	1.00	1.02
7.94	5.36	9	54.1	98.7	66.2	0.95	0.98	0.99
9.46	5.36	10	0.4	97.4	26.8	0.95	0.97	0.99
3.54	5.54	11	100.0	100.0	100.0	0.91	0.94	0.95
6.22	5.59	12	99.9	100.0	99.9	0.89	0.90	0.92
7.37	5.54	13	81.9	100.0	86.8	0.90	0.91	0.93
7.90	5.55	14	55.7	100.0	67.8	0.89	0.91	0.92
9.57	5.53	15	0.1	100.0	27.3	0.89	0.91	0.92
3.64	5.76	16	100.0	98.2	99.5	0.85	0.86	0.87
6.15	5.75	17	99.9	98.4	99.5	0.85	0.86	0.86
7.58	5.73	18	73.1	99.3	80.3	0.85	0.86	0.87
8.06	5.74	19	46.3	98.4	60.5	0.84	0.85	0.87
9.49	5.74	20	0.4	98.1	27.0	0.84	0.85	0.87
3.47	5.95	21	100.0	52.6	87.1	0.82	0.83	0.83
5.83	5.94	22	100.0	55.5	87.9	0.82	0.82	0.83
7.28	5.93	23	92.2	60.2	83.5	0.82	0.82	0.83
8.07	5.93	24	46.1	60.5	50.0	0.82	0.82	0.83
9.63	5.93	25	0.7	62.0	17.4	0.81	0.81	0.83

Table 7.14 PWL values and rutting for the 2nd set of MEPDG runs for performance criteria

Mean AV	Mean AC	Run No.	PWL av	PWL ac	PWL comb	Avg. Rut (in)	75th Percentile	90th Percentile
3.56	5.20	1	100.0	69.3	91.6	1.04	1.08	1.10
6.02	5.17	2	98.9	58.6	87.9	1.04	1.08	1.10
7.42	5.18	3	69.4	60.4	66.9	1.04	1.08	1.09
8.03	5.20	4	48.6	67.8	53.9	1.02	1.07	1.08
9.57	5.18	5	7.0	62.0	22.0	1.02	1.06	1.07
3.60	5.37	6	100.0	92.1	97.9	0.97	1.01	1.06
5.99	5.35	7	95.4	91.6	94.4	0.97	1.00	1.07
7.48	5.39	8	67.8	91.0	74.1	0.95	0.99	1.05
7.90	5.35	9	54.4	92.0	64.7	0.96	0.99	1.03
9.72	5.35	10	4.9	91.1	28.4	0.95	0.97	1.01
3.37	5.55	11	100.0	98.5	99.6	0.91	0.93	0.97
5.96	5.54	12	97.2	99.6	97.8	0.90	0.93	0.95
7.66	5.58	13	61.2	97.7	71.2	0.89	0.91	0.96
8.11	5.55	14	46.0	99.6	60.6	0.89	0.93	0.94
9.58	5.57	15	4.3	99.7	30.3	0.88	0.90	0.91
3.65	5.74	16	100.0	94.4	98.5	0.86	0.87	0.89
5.87	5.75	17	99.5	94.4	98.1	0.85	0.87	0.88
7.47	5.76	18	70.3	91.1	75.9	0.84	0.86	0.88
8.04	5.72	19	48.4	95.9	61.4	0.85	0.87	0.88
9.43	5.75	20	7.5	89.6	29.9	0.84	0.86	0.87
3.60	5.91	21	100.0	64.6	90.3	0.83	0.84	0.85
5.80	5.91	22	99.7	67.6	91.0	0.82	0.83	0.84
7.40	5.91	23	72.8	63.5	70.3	0.82	0.82	0.84
7.89	5.91	24	54.4	65.4	57.4	0.82	0.82	0.84
9.60	5.93	25	2.4	58.8	17.7	0.81	0.82	0.83

Table 7.15 PWL values and rutting for the 3rd set of MEPDG runs for performance criteria

Mean AV	Mean AC	Run No.	PWL av	PWL ac	PWL comb	Avg. Rut (in)	75th Percentile	90th Percentile
3.62	5.18	1	99.8	60.8	89.2	1.05	1.09	1.11
6.12	5.20	2	96.0	66.2	87.9	1.03	1.07	1.09
7.71	5.19	3	59.6	64.5	60.9	1.03	1.07	1.08
8.12	5.19	4	45.6	64.6	50.8	1.03	1.07	1.09
9.37	5.18	5	12.7	60.0	25.6	1.02	1.06	1.08
3.53	5.35	6	99.9	90.1	97.3	0.98	1.03	1.05
6.04	5.35	7	94.2	91.2	93.4	0.97	1.00	1.04
7.74	5.33	8	60.1	92.0	68.8	0.97	1.01	1.04
7.95	5.36	9	51.5	91.6	62.5	0.96	1.00	1.03
9.37	5.38	10	11.5	93.5	33.9	0.94	0.97	1.01
3.56	5.53	11	100.0	99.6	99.9	0.91	0.95	0.97
5.97	5.53	12	93.2	98.4	94.7	0.91	0.94	0.97
7.40	5.55	13	67.2	98.9	75.8	0.90	0.91	0.96
8.19	5.56	14	43.7	99.8	59.0	0.89	0.91	0.93
9.44	5.58	15	6.7	98.4	31.7	0.88	0.90	0.94
3.53	5.76	16	100.0	91.8	97.7	0.86	0.87	0.90
5.80	5.76	17	96.1	91.5	94.9	0.85	0.86	0.88
7.48	5.78	18	69.7	91.0	75.5	0.84	0.86	0.88
7.87	5.75	19	54.1	93.1	64.7	0.84	0.85	0.89
9.49	5.77	20	6.9	85.4	28.3	0.84	0.87	0.89
3.64	5.91	21	100.0	65.4	90.5	0.83	0.84	0.85
6.12	5.93	22	89.1	57.3	80.4	0.82	0.83	0.84
7.49	5.92	23	68.0	61.0	66.1	0.82	0.83	0.84
7.66	5.90	24	60.3	67.9	62.4	0.82	0.83	0.84
9.48	5.92	25	12.3	63.0	26.2	0.81	0.82	0.83

7.6 Effect of varying HMA thickness on project performance by sublot

In this section, we present the results corresponding to the case of varying HMA surface layer thickness to achieve various PWL ranges and show its effect on rutting and fatigue performance. The pavement structure that was simulated to analyze the effect of variability in thickness on pavement performance is shown in Figure 7.22. This is from an SPS 1 site in Michigan. This section was chosen because it had shown poor performance. This would highlight the effect of variability. If the pavement was performing very well, the problems that will result because of thickness deficiency would get compensated by other strengths of the pavement. The results presented subsequently correspond to fatigue and rutting performance as well as IRI for a design life of 20 years.

Layer No.	Layer Description
5	Original Surface Layer (Layer Type:AC)1.8 Inch
4	AC Layer Below Surface (Binder Course) (Layer Type:AC)1.8 Inch
3	Base Layer (Layer Type:PATB)4 Inch
2	Base Layer (Layer Type:GB)8 Inch
1	Subgrade (Layer Type:SS)

Section 120

Layer No.	Layer Description
5	Original Surface Layer (Layer Type:AC)1.9 Inch
4	AC Layer Below Surface (Binder Course) (Layer Type:AC)2 Inch
3	Base Layer (Layer Type:PATB)4 Inch
2	Base Layer (Layer Type:GB)8 Inch
1	Subgrade (Layer Type:SS)

Section 121

Figure 7.22 Pavement structure used in simulation to study the effect of AC thickness variation on pavement performance

To generate and study different realistic case scenarios, the AC thicknesses were varied over a mean thickness of 7.6 inches as shown in Table 7.16. The variations correspond to mean thickness ± 3 standard deviations. Table 7.16 also summarizes the results for effect of AC thickness variation on fatigue performance. Since MDOT does not have thickness specification limits for HMA pavements, the lower thickness specification limit was set as the target thickness minus 0.75 inch for the purpose of this case study. Figures 7.23 through 7.25 capture the effect of AC thickness variation on fatigue performance using the response surface for the nine different case scenarios listed in Table 7.16.

Similarly, the effect of AC thickness variation on rutting performance is displayed in Figures 7.26 through 7.28 for the nine different case scenarios listed in Table 7.17. Figures 7.29 through

7.31 capture the effect of AC thickness variation on IRI of sublots for nine different case scenarios listed in Table 7.18.

Table 7.16 Summary of results for effect of AC thickness variation on fatigue performance

Mean Thickness (in)	Std_dev of Thickness (in)	PWL	Mean Fatigue (%)	Fatigue 75th %tile (%)	Fatigue 90th %tile (%)
7.1	0.10	99.2	40.6	42.3	43.5
7.1	0.31	78.0	41.1	46.1	50.0
7.1	0.51	67.2	41.7	50.2	56.2
7.6	0.10	100.0	32.1	33.4	34.2
7.6	0.31	99.2	32.3	36.0	39.3
7.6	0.51	92.3	32.9	39.4	45.3
8.1	0.10	100.0	24.5	25.6	26.4
8.1	0.31	100.0	24.9	28.3	31.2
8.1	0.51	99.2	25.3	31.3	35.4

Table 7.17 Summary of results for effect of AC thickness variation on rutting performance

Mean Thickness (in)	Std of Thickness (in)	PWL	Rutting (in)	Rutting 75th %tile (in)	Rutting 90th %tile (in)
7.1	0.09	99.8	0.86	0.86	0.87
7.1	0.27	83.7	0.86	0.87	0.89
7.1	0.45	73.0	0.87	0.87	0.93
7.6	0.09	100.0	0.85	0.86	0.86
7.6	0.27	99.8	0.85	0.85	0.86
7.6	0.45	95.7	0.85	0.85	0.87
8.1	0.09	100.0	0.83	0.83	0.83
8.1	0.27	100.0	0.83	0.83	0.85
8.1	0.45	99.8	0.82	0.84	0.85

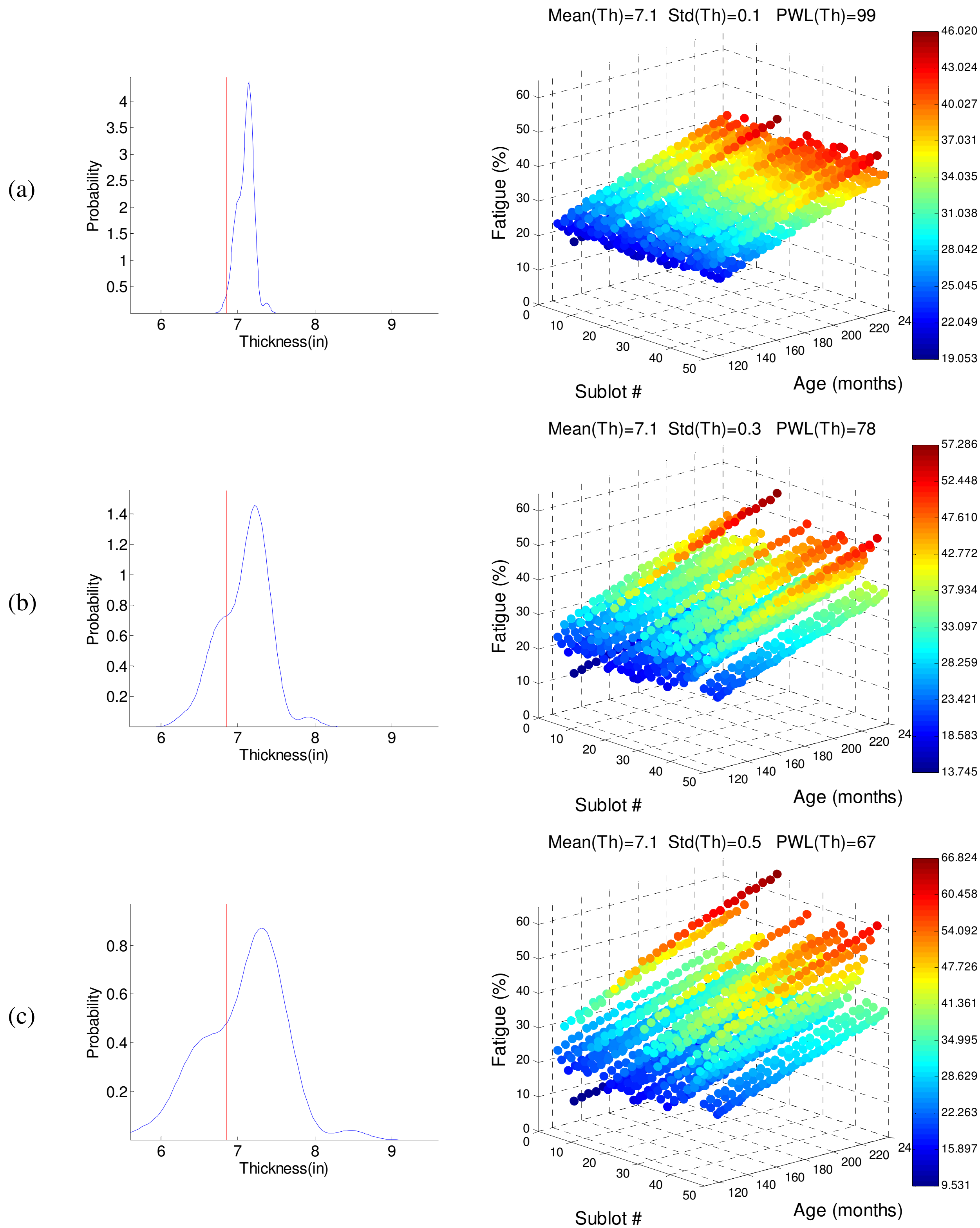


Figure 7.23 Effect of AC thickness variation on fatigue performance of sublots for Th=Opt-0.5.

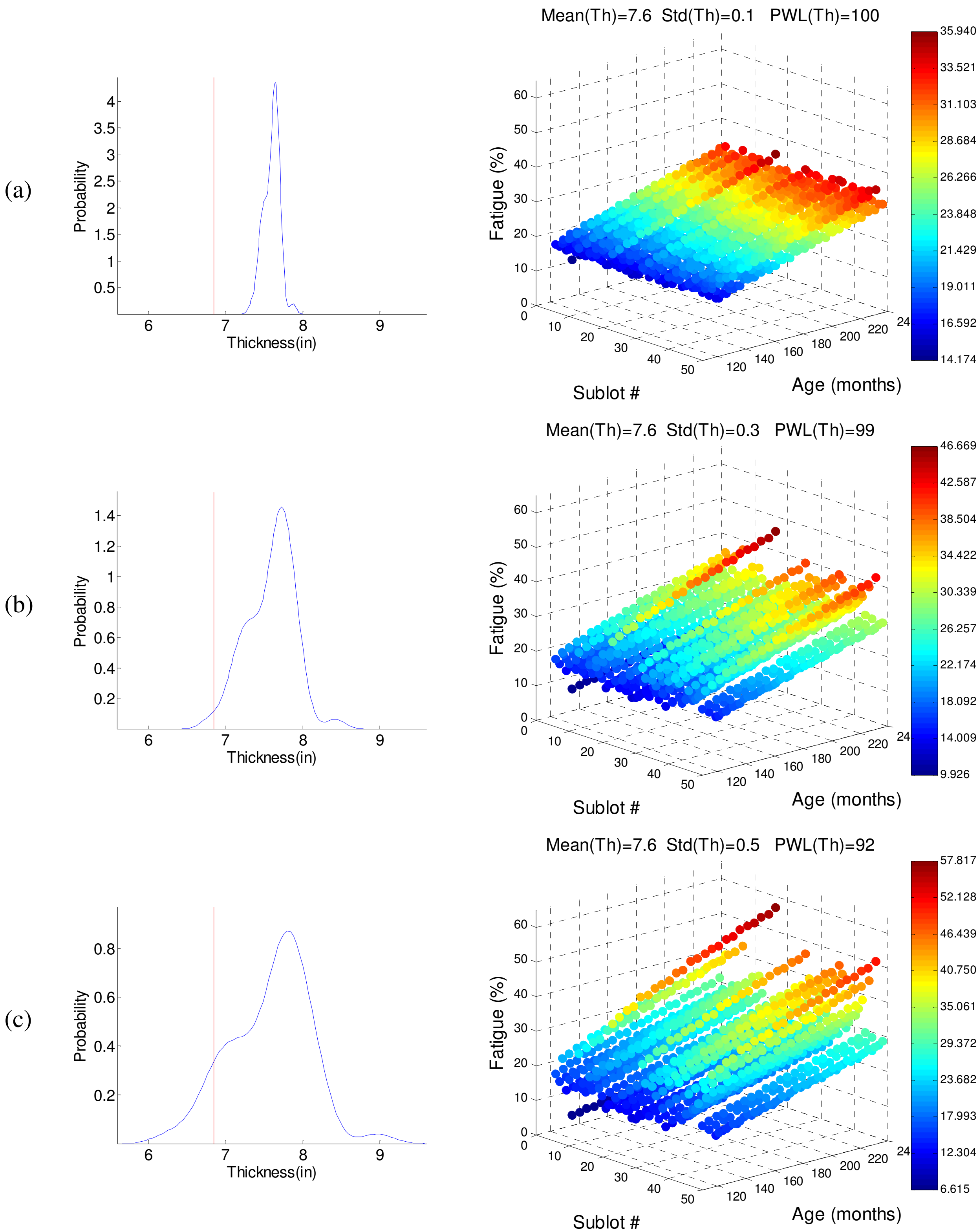


Figure 7. 24 Effect of AC thickness variation on fatigue performance of sublots for Th=Opt.

(a)

(b)

(c)

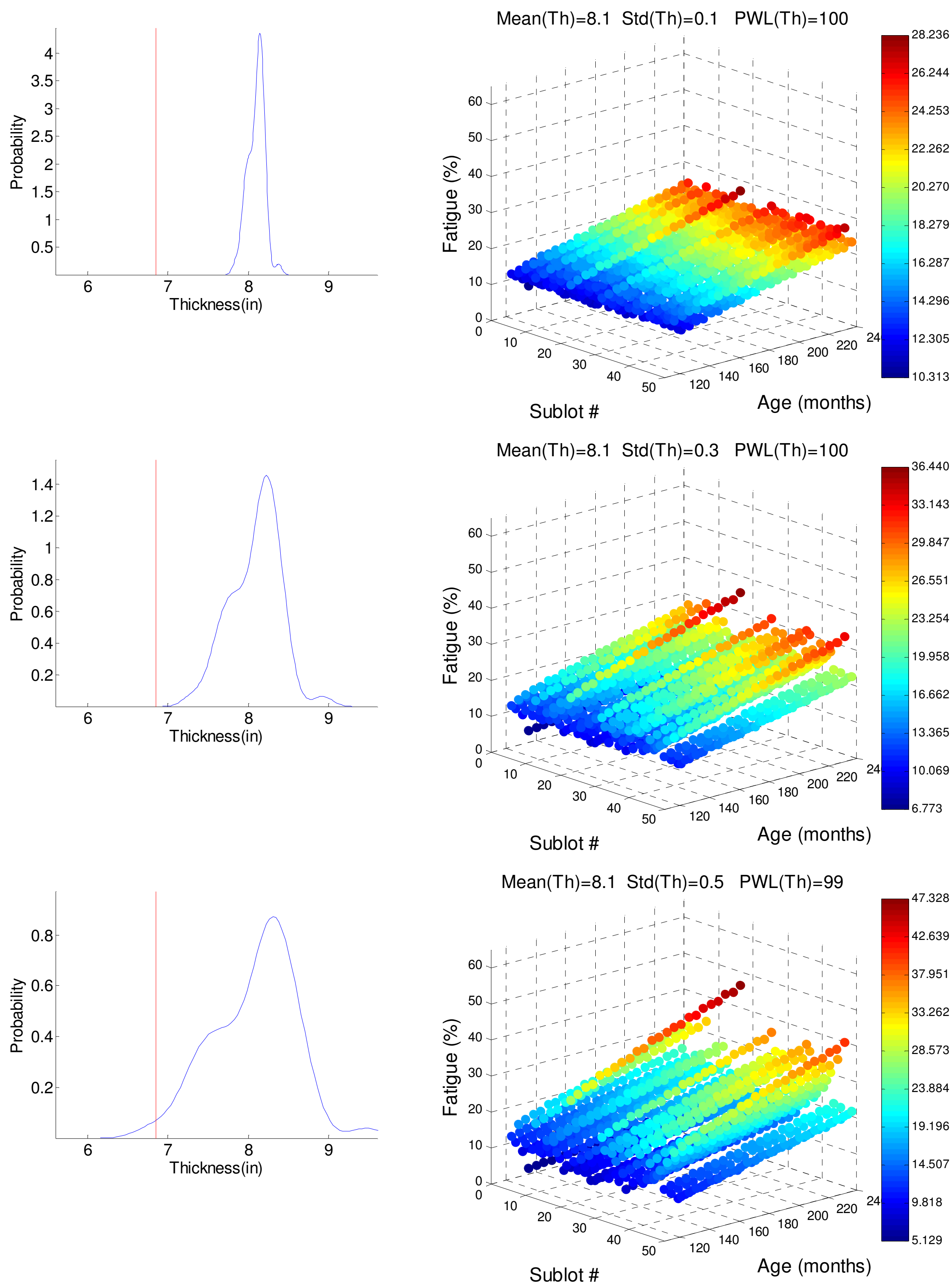


Figure 7. 25 Effect of AC thickness variation on fatigue performance of sublots for Th=Opt+0.5.

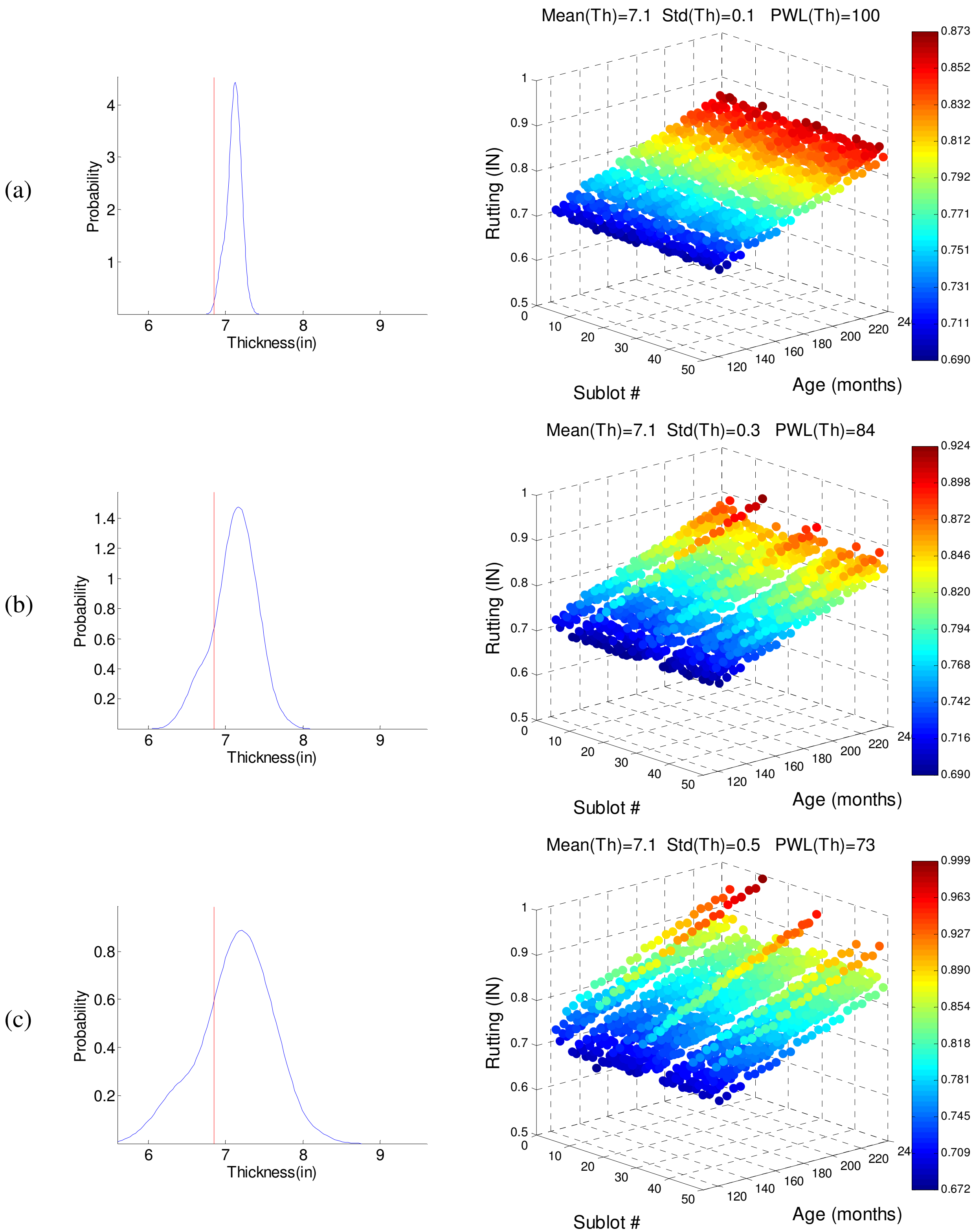


Figure 7. 26 Effect of AC thickness variation on rutting performance of sublots for Th=Opt-0.5.

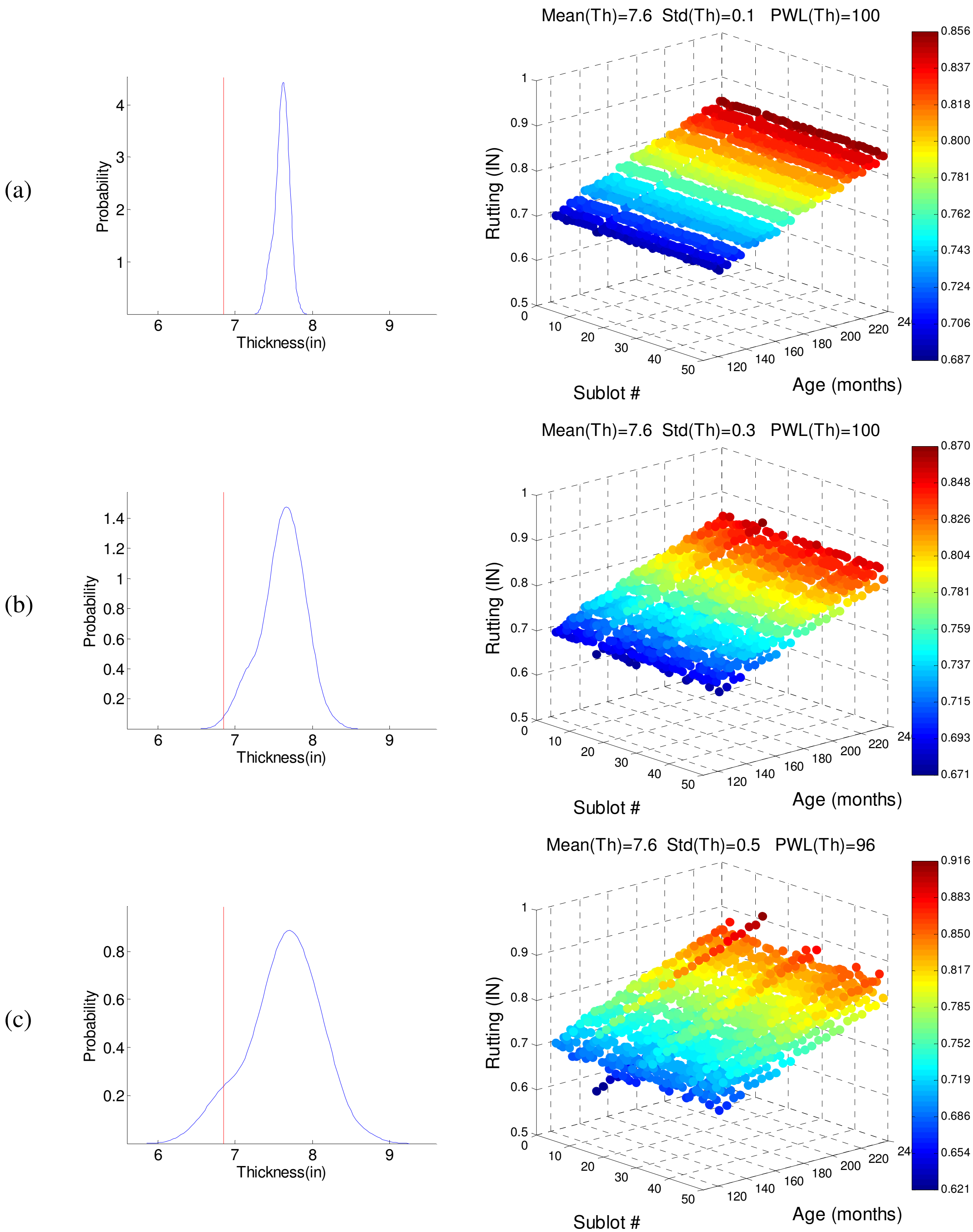


Figure 7. 27 Effect of AC thickness variation on rutting performance of sublots for Th=Opt.

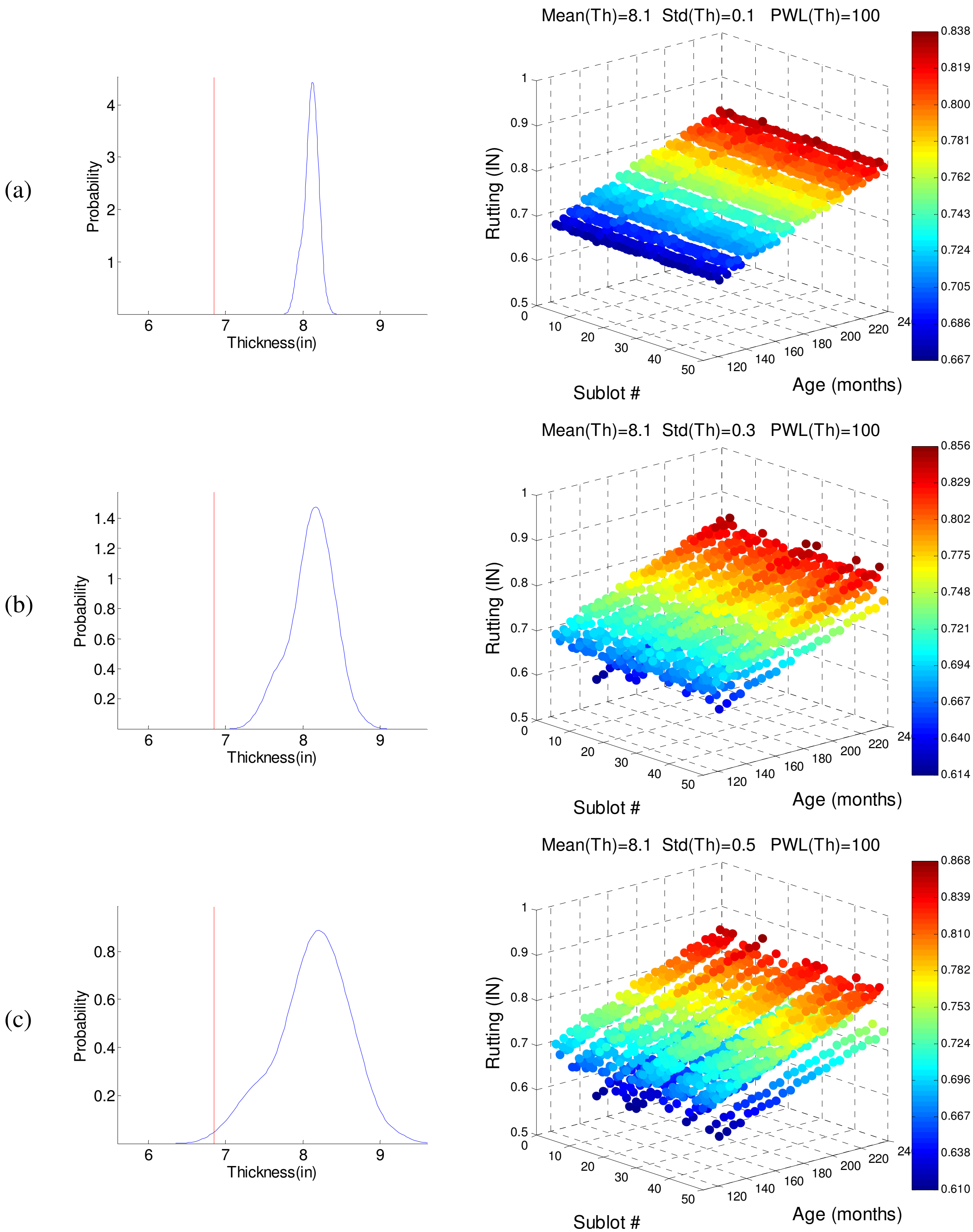


Figure 7. 28 Effect of AC thickness variation on rutting performance of sublots for Th=Opt+0.5.

Table 7.18 Summary of results for effect of AC thickness variation on IRI

Mean Thickness (in)	Std of Thickness (in)	PWL	IRI (in/mi)	IRI 75th %tile (in/mi)	IRI 90th %tile (in/mi)
7.1	0.09	99.5	158	159	161
7.0	0.27	76.0	159	164	170
7.0	0.45	63.2	162	170	182
7.6	0.09	100.0	149	150	150
7.5	0.27	99.5	150	152	156
7.5	0.45	92.6	151	155	164
8.1	0.09	100.0	142	143	143
8.0	0.27	100.0	143	145	148
8.0	0.45	99.5	143	148	152

Table 7.19 displays a sample table from the LTPP database with representative AC layer thickness variation, which also confirms that ranges used for the current study are reasonable and realistic.

It must be mentioned that all these results are for one type of mix and pavement structure. It is important to study different types of mixes and pavement structures to assess how PWL values affect performance for each one of them. It is quite possible that this effect may be much more pronounced in other types of mixes.

Table 7.19 Sample LTPP table with representative AC layer thickness variation

STATE_CODE	SHRP_ID	LAYER_NO	MEAN_THICKNESS	MIN_THICKNESS	MAX_THICKNESS	STD_DEV_THICKNESS
12	4136	4	1.4	1.3	1.5	0.1
12	4137	4	2.8	2.7	2.9	0.1
16	3017	4	4	3.4	4.9	0.1
19	1044	5	1.9	1.6	2.2	0.1
19	1044	6	0.9	0.7	1.2	0.1
24	0500	7	1	0.8	1.1	0.1
88	1647	5	1.6	1.4	1.8	0.1
12	4105	4	2.2	1.8	2.7	0.2
12	4135	4	1.6	1.5	1.8	0.2
19	6049	8	1.6	1.4	2.5	0.2
24	0500	5	2	1.8	2.3	0.2
13	4112	3	3.9	3.3	4.4	0.3
13	4113	3	3.9	3.3	4.4	0.3
19	6049	6	1.6	1.3	2.4	0.3
19	6049	7	1.7	1.1	3	0.3
24	0500	6	1.7	1.2	2	0.3
30	8129	5	3	3	4	0.3
40	1015	2	8.8	8	9	0.3
48	5328	3	4.3	4	4.7	0.3
88	1645	4	1.6	1	2	0.3
12	0900	4	2.3	1.2	3	0.4
13	4112	2	12.2	11.5	13	0.4
13	4113	2	12.2	11.5	13	0.4
19	5042	2	4	3.5	5	0.4
19	6049	3	1.9	1.1	3.5	0.5
19	9116	2	4	3.5	5.4	0.5
19	1044	4	12.6	12	14	0.6
88	1647	4	5.2	4	7	0.6
34	0500	5	2.4	1	3	0.8
34	0500	4	5.7	3	6	0.9
29	7054	5	1.8	-	-	1

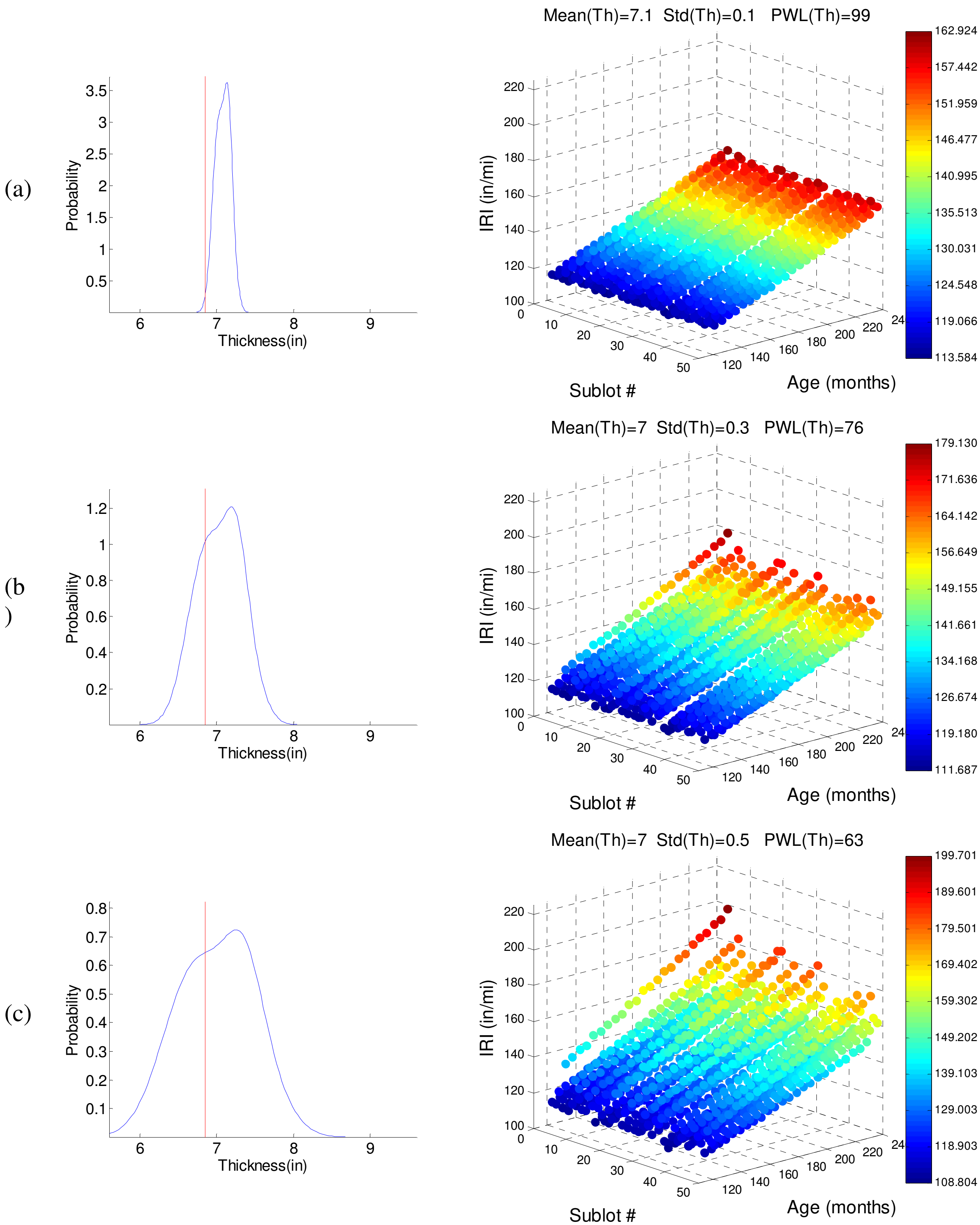


Figure 7. 29 Effect of AC thickness variation on IRI of sublots for Th=Opt-0.5.

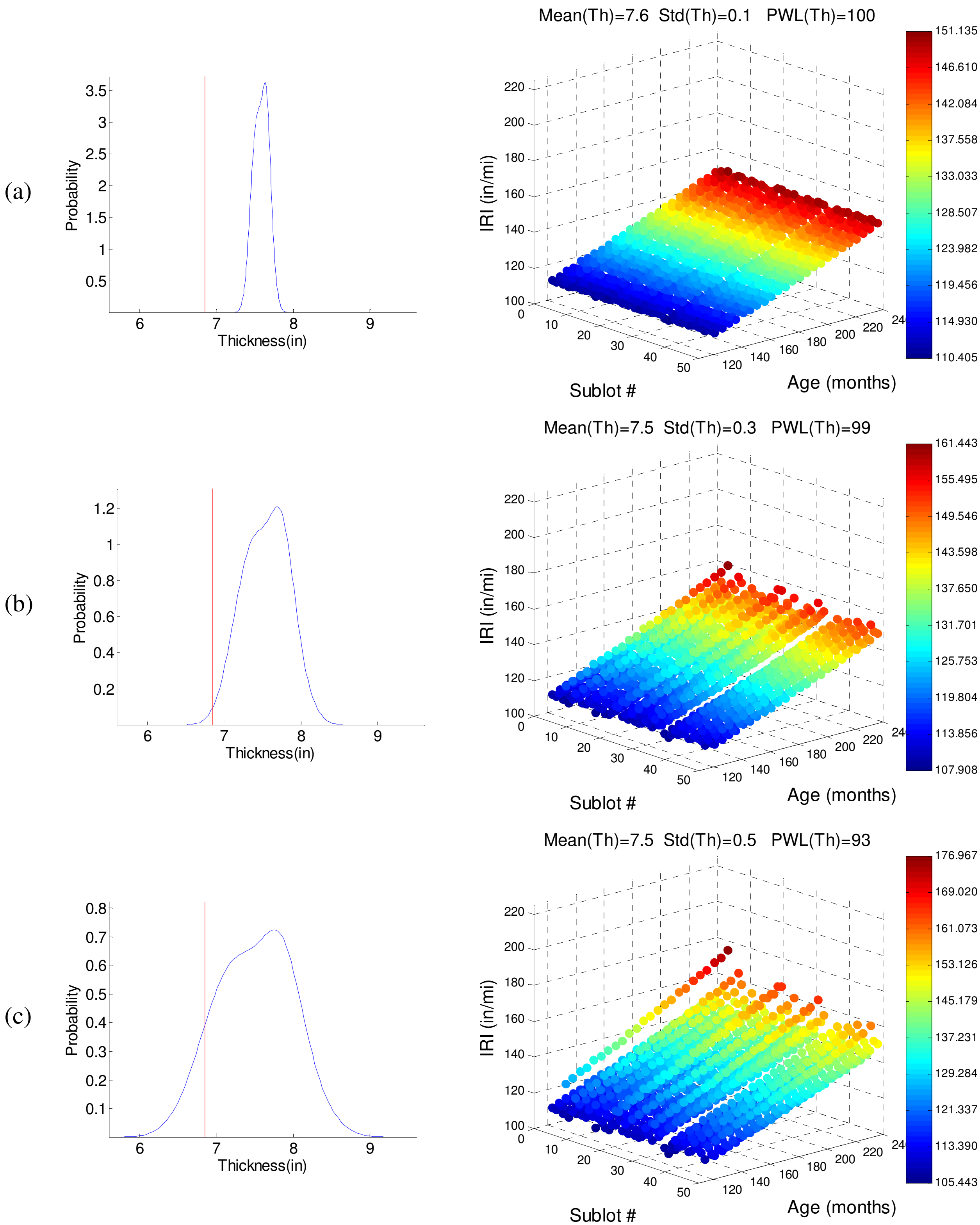


Figure 7. 30 Effect of AC thickness variation on IRI of sublots for Th=Opt.

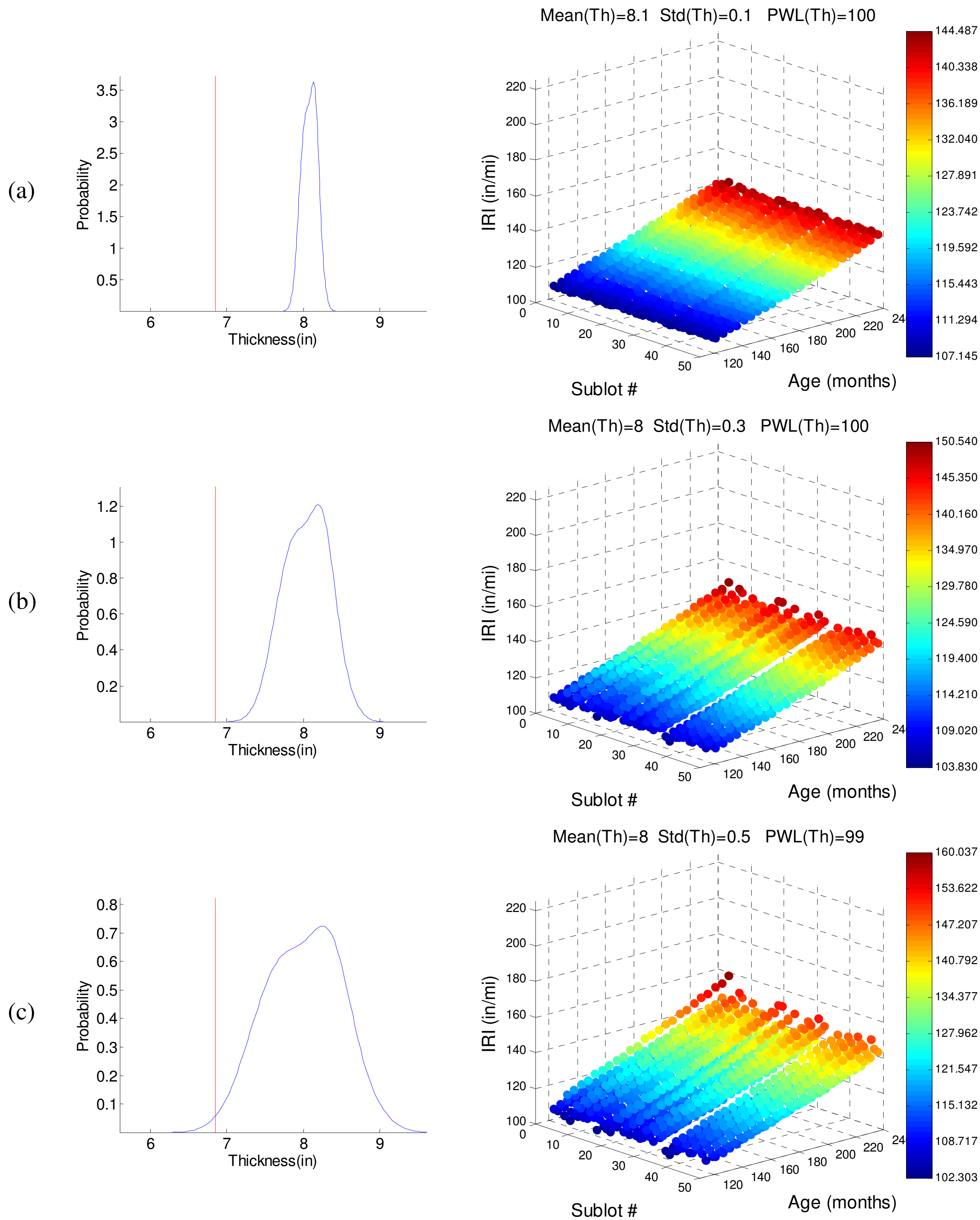


Figure 7. 31 Effect of AC thickness variation on IRI of sublots for Th=Opt+0.5.

Based on the results from the analysis of the effect of AC thickness variation on project performance, the following conclusions are drawn:

1. The performance of different sublots within a project can vary significantly.
2. AC mat thickness variation has a significant influence on fatigue performance. The mean fatigue ranges from 25.3% to 40.6% for AC mat thickness ranging from 7.1 in. to 8.1 in., respectively. Thus, the loss in fatigue performance can be significant with as little as 0.5-in. reduction in AC mat thickness.
3. There seems to be almost 10% increase in fatigue distress for every 0.5-in. decrease in AC mat thickness from 8.1 in. to 7.1 in.
4. The mean IRI ranges from 143 in./mile to 158 in./mile for thicknesses ranging from 8.0 in. to 7.1 in., respectively. It should be noted that in MEPDG, the IRI values are influenced by rutting and fatigue distresses.
5. These simulations and plots provide valuable information in helping us understand how performance varies within a project for such narrow ranges and variations in inputs, which is not possible to gain any other way.

Chapter 8: ERS Risk Analysis Using Simulation

8.1 INTRODUCTION AND BACKGROUND

Over the years many highway agencies in North America have made a valued commitment to End Results Specifications (ERS). As a direct result, it is believed that the quality of our roadways has improved (Smith, 1998, Benson, 1999). A quality Assurance (QA) program, which involves material testing, plays an important role in measuring this quality and is an integral ERS component. The results from the material testing are used to determine payment to be made to the contractor.

Aurilio et al. (2002) considered the effect of differences between laboratory test results on payment to the contractor. Further analysis of actual ERS project data indicated that in addition to test bias several other factors, like measurement variability, production variability etc. can have significant effect on payment. It has also been demonstrated that the concept of simulation program can be used to take into account all the parameters that could be identified to be affecting payment calculation (Aurilio et al. 2002, Manik and Buttlar, 2006). This chapter reports on the development of a Monte-Carlo based simulation program for assessing Michigan Department of Transportation's QA program and estimate the errors or risk involved with payment made to the contractor according to the provisions in the QA program. The chapter presents the details of the program, analysis and conclusions that can be derived from those. Such study would provide valuable insight into how QA programs can be formulated to reduce risk to the contractor as well as the agency and also balance risk.

8.2 END-RESULT SPECIFICATIONS

End-result specification places full responsibility of producing a pavement of a certain specified quality on the contractor. The contractor has full freedom to choose methodologies for construction process and take strategic decisions. He conducts quality control tests at a specified frequency to monitor the quality of the pavement being constructed. The responsibility of the state highway authority (SHA) is to check from their own side that the quality is acceptable, through quality assurance tests (AASHTO, 1996). The SHA can decide, based on criterion laid out in the specification, whether the quality is acceptable or rejectable, or that the pavement be accepted but with penalty to the contractor in terms of reduced pay. Adjustment in pay is one of the most significant aspects of ERS in present day practices. Rather than setting pass and fail criteria, a percent of the material produced is judged to be within acceptable limits and payment is determined accordingly. This calls for use of statistical methods (Box and Wilson, 1951)

The quality characteristics (defined as that characteristic of a unit or product that is actually measured to determine conformance with a given requirement) that are being used to determine "quality" of the pavement are generally air voids, binder content, voids in mineral aggregates, density etc. for flexible pavements. These quality characteristics are believed to be related to performance but the exact relationships are not yet firmly quantitatively established for all of them. Therefore, the pay adjustments are based on the values of the quality characteristics themselves and not on expected performance of the constructed pavement (Smith, 1998).

8.3 ESTIMATING RISK

In the past, researchers have attempted to develop statistical or simulation tools to help understand and balance risks in asphalt construction specifications. A computer simulation program called OCPLOT, developed in FHWA Demonstration Project 89 by Weed (Weed, 1996) is available for generating Operating Characteristics (OC) curves. OCPLOT was found to be user-friendly and very useful for initial assessment of relative risks, allowing the user to vary the following factors: sample size, pay factor equation, specification limits, and retest provisions. The program allows the user to assess the probability of acceptable material being rejected (defined as contractor risk) and the probability of rejectable quality material being accepted (defined as agency risk) over the long run (e.g., when considering the characteristics of the specification over a long period of time). However, a number of the factors that appear to be related to risk, including measurement variability and testing bias are not considered in OCPLOT. In addition, it can be argued that the most tangible measurement of risk should be linked to the financial impact on the project, i.e., how *risk* affects *what is actually paid versus what should have been paid.*

One of the necessary steps in the assessment of payment risk is to clearly define the risk metric. A very straight-forward and yet very effective way of defining risk could be as shown in equation 1, where baseline pay represents the ideal or correct payment. .

$$\text{Payment Risk} = \text{Payment made to the contractor} - \text{Base Line pay} \qquad (1)$$

Ideally, tests performed by different parties on the same material should give very similar results. However, in practice even split samples will show different results when the tests are carried out by two different agencies or in two different labs. Because of these uncertainties there is a risk of accepting rejectable quality and vice-versa. In the ERS approach, a percentage of acceptable quality (Percent Within Limits-PWL) is determined rather than pass/fail criteria used in typical QC/QA. Then, payment is made based on this percent within limits value (Patel, 1996). Because of the uncertainties involved with the test results, the payment made also may be more or less than what it would be if the actual quality of the construction would have been exactly determined (Weed, 1996; Willenbrock, 1976; Bowery and Hudson, 1976; Barros et al. 1983; Puangchit et al., 1983; Afferton et al., 1992; AASHTO, 1995). Overpayment of the contractor is often referred to as 'agency risk' while underpayment is often termed as 'contractor risk'. Throughout this report, *positive values of risk refer to the instance where the agency paid more than required (agency risk) and negative values of risk indicate that the agency paid less than what the contractor deserved (contractor risk).*

Buttlar and his coworkers at the University of Illinois at Urbana-Champaign have developed a series of risk simulation models that provide the user a virtual environment to quickly generate and analyze thousands of realistic ERS data sets. The first simulation model developed was ILLISIM (Buttlar and Hausman, 2000). This was followed by PaySim and BiasSim (Aurillio et al. 2002) which used different models and catered to different aspects of risk analysis and simulation. The latest model developed for the Illinois Department of Transportation is called Simulate Risk Analysis, or SRA, which combines the capabilities of all earlier programs into a single program, with added features to simplify the process of conducting sensitivity analyses (Buttlar and Manik, 2007). Using the same principles, a new simulation model called AMSim has been developed to analyze the MDOT QA program by the authors.

This chapter presents the details of this simulation model along with the analysis performed and conclusions derived from the analysis.

8.4 MAJOR FACTORS AFFECTING RISK

Analysis of data obtained from actual construction projects corresponding to various quality characteristics like density, binder content, air voids etc., have shown that such data are generally normally distributed *(Hall and Williams, 2002)*. Generally, the target values for these quality characteristics are fixed. This indicates that, as it would be expected in the real world, there are certain factors involved in the construction and quality characteristic measurement procedures which are not completely controllable, or even predictable. They tend to induce variability (Benson, 1995) in the quality around the targeted quality level.

Variability observed in the field, however, has at least two components, namely production variability and measurement variability. Production variability includes all variability introduced due to workability of concrete, variability in the quality and physical characteristics of source materials, changes in the relative proportions of ingredients in the mix, changes in plant operational characteristics, changes in equipment operators, changes in ambient temperature etc. Measurement variability is the variability which is introduced by the measuring devices, test procedures, and operator techniques and human error. In addition to variability around the actual value, a measurement bias may be introduced as well. Bias refers to a consistent shift in data and can be introduced by device calibration errors, human error, or by the intentional biasing of measurements and/or recorded data (Aurilio et al. 2002).

Every choice made in the development of an ERS comes with an associated risk. Risks are undertaken by both the contractor and the agency. The introduction or manipulation of certain specification attributes can shift the risk from the contractor to the agency and vice-versa. Other specification attributes can widen or narrow the range of risk. In summary, the key contributors to risk in ERS are:

- Contractor data versus agency data
- Frequency of testing and/or number of samples
- Variability and/or bias of test device and/or test procedure
- Specification parameters, including:
    - Specification limits
    - Pay factor equation
    - Pay “caps”

- Acceptance test logic and frequency and acceptance tolerance
- Third party testing provisions

In the procedure used for determining the pay factor in ERS, a sample of data with finite measurements is used to estimate the quality of a population which this sample belongs to. Therefore, mean and standard deviation of any quality characteristic of the sample is considered as equal to the mean and standard deviation of the entire material in the lot or pavement produced in that project. However, the finite sample being used may not have exactly the same mean and standard deviation as it could have been if a much larger number of samples were collected. Theoretically, actual quality of the material can be determined only if the sample collected is infinite. Such an infinite size sample, or rather population, would give payment called as “ideal payment”. Therefore, finite size samples would lead to a deviation from the ideal pay. In addition, the use of imperfect measuring devices would also lead to error in measurements. The error in turn would lead to deviation from the ideal pay. Therefore, to be able to determine the ideal pay, thousands of data with similar characteristics would need to be simulated, with each simulation representing an actual individual project. Pay calculated for each individual project coupled with the ideal or base-line pay for the entire population would provide distribution of risk on a project with those characteristics.

In order to simulate variability in an asphalt pavements material properties, one must be able to sequentially simulate, in this order: 1) production or construction variability; 2) results of random samples taken from that variable material; 3) the effects of measurement variability on the estimated properties; and finally; 4) the effects of bias on the final reported test measurement values. In order to estimate risk in terms of effects on pay, the software must also simulate the formulas and decision tree logic contained in the construction specification.

8.5 COMPOSITE RISK INDEX

A simulation tool like AMSim or SRA helps estimate and analyze risk in payment that can be expected in different scenarios using a certain set of end-result specifications. The main advantage of such a tool is that it can provide invaluable information in what-if scenarios without the need of a demonstration project or shadow specification. This can greatly help in determining the effect of different aspects or values in the specification used in end-result projects.

The main format in which AMSim would provide information would be risk plots. A risk plot presents the expected mean risk and associated confidence interval for the entire range of quality characteristic possible on a project. This means that a risk plot can give a very good understanding of how “well” a set of specifications would do for that quality characteristic.

A wealth of information can be gained from the risk plots generated by SRA. However, the interpretation of the risk plot could be subjective. This may make it difficult to compare risk scenarios arising because of two different specifications or any combination of other parameters affecting risk. In addition to this, if an algorithm needs to be developed for comparing risk plots for the purpose of comparing specifications etc. various quantitative characteristics of the plot would have to be used. Manik (2006) developed a composite risk index (CRI) to quantitatively characterize the risk plots. The concept of CRI was tested on a wide range of risk plots and was found to be very objective and promising in its purpose. The analysis presented in this chapter also uses CRI.

8.6 RISK ANALYSIS

One of the earlier sections in this chapter identified several factors associated with a QA program that affect the risk involved in payment made to the contractor through that program. It is very important to assess how exactly these factors affect payment risk. In addition to that, it is also important to determine how their contribution to other factors influence payment risk. This section presents risk analysis performed for the MDOT QA program with the following four factors in focus.

(1) Production Variability
(2) Measurement Variability
(3) Sample size and
(4) Bias

Three levels were identified for each of these four quality characteristics and a full factorial run matrix was constructed as shown in Table 8.1.

Table 8.1 Run Matrix for Risk Analysis of Flexible Pavements

Sample Size	Bias	Prod Var** = 0.45			Prod Var = 1.4			Prod Var = 2.3		
		Meas var*=0.1	Meas var=0.5	Meas var=1	Meas var=0.1	Meas var=0.5	Meas var=1.0	Meas var=0.1	Meas var=0.5	Meas var=1.0
10	0.1	1	10	19	28	37	46	55	64	73
	0.3	2	11	20	29	38	47	56	65	74
	0.5	3	12	21	30	39	48	57	66	75
40	0.1	4	13	22	31	40	49	58	67	76
	0.3	5	14	23	32	41	50	59	68	77
	0.5	6	15	24	33	42	51	60	69	78
70	0.1	7	16	25	34	43	52	61	70	79
	0.3	8	17	26	35	44	53	62	71	80
	0.5	9	18	27	36	45	54	63	72	81

* Measurement Variability in %

** Production Variability in %

AMSim simulates the entire MDOT QA program including the specification limits, sampling scheme and decision logic ending with pay factor calculations. The MDOT QA program has similar specifications for flexible pavement in-situ density, air voids, binder content and VMA. The sampling scheme for in-situ density is different because samples are collected from the mat immediately after compaction. In this case, randomization is used not only in the longitudinal direction but in transverse direction also. For the other factors, quality characteristics samples are collected in the form of mixture and, therefore, randomization is applied to the entire mixture in each sublot. The analysis presented here first corresponds to in-situ density followed by plant air voids. For all the four quality characteristics, pay formula is used instead of pay-schedule. In the past, pay schedules were very commonly used. However, with the increasing use of statistical methods in ERS, pay schedules have been replaced by percent-within-limits concept and pay formula (Buttlar and Harrell, 1998). Table 8.2 shows an example of what a pay schedule looks like. This is the pay schedule used by MDOT for thickness QA program for rigid pavements. Equations 1 and 2 show the pay formulae used for density QA program for flexible pavements. For PWL less than 50 corrective action is required as described below (MDOT specifications).

Table 8.2 Price Adjustment for Concrete Thickness Deficiency

Initial Core Type	Deficiency in Thickness (Inch)	Price Adjustment (Percent)
A	0.20 or Less	0
B	0.30	-5.0
B	0.40	-15.0
B	0.50	-25.0
B	0.60 To 1.0	-50.0
C	1.10 and Over	-100 [a]

A. If PWL for In-Place Density (PWL_D) is between 100 and 70, use the following formula to determine PF_D. Round the value of PF_D two decimal places.

$$PF_D = 55+(0.5xPWL) \qquad (1)$$

B. If PWL for In-Place Density is between 70 and 50 inclusive, use the following equation to determine PF_D. Round the value of PF_D two decimal places.

$$PF_D=37.5+(0.75xPWL) \qquad (2)$$

C. If PWL for In-Place Density is less than 50; the Engineer may elect to do one of the following:

(1) Require removal and replacement of the entire lot with new QA sampling and testing and repeat the evaluation procedure.

(2) Allow the lot to remain in place and apply an Overall Lot Pay Factor of 50.00.

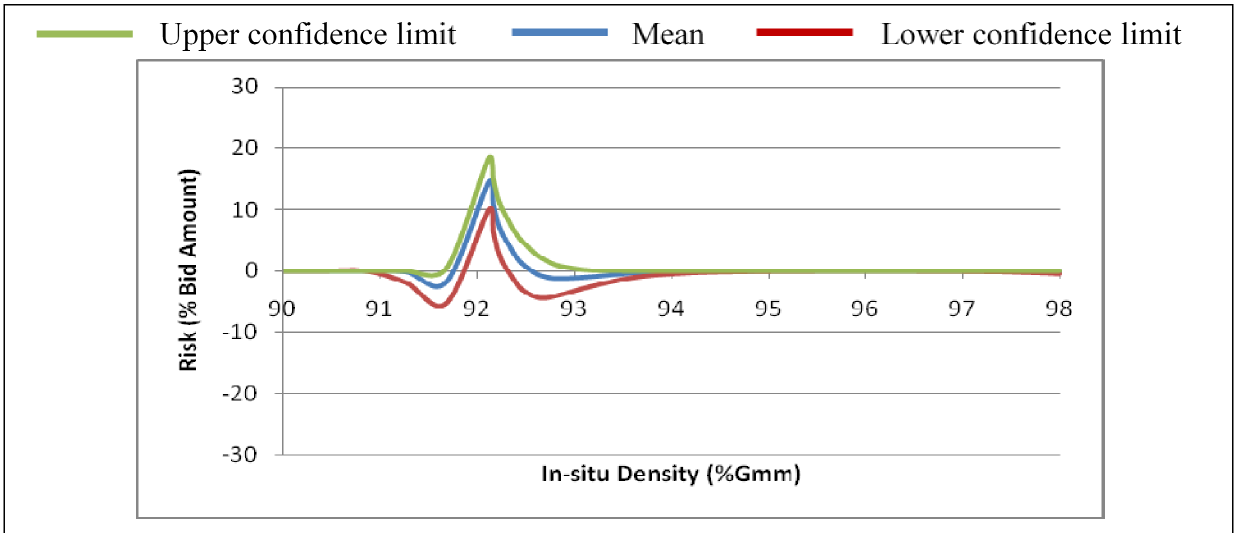
(3) Allow submittal of a corrective action plan for the Engineer's approval. The corrective action plan may include removal and replacement of one or more sublots. If one or more sublots are replaced, the sublot(s) will be retested and the Overall Lot Pay Factor will be recalculated according to this special provision. If the Engineer does not approve the plan for corrective action, subsections (1) or (2) above will be applied.

AMSim was run for all the 81 cases identified in the run matrix (Table 8.1). Figures 8.1 through 8.4 show sample results from the 81 runs performed with AMSim. It would be important to describe the concept of risk plot first. The x-axis in the risk plot has mean of the quality characteristic in the QA program which is being analyzed. The three risk curves shown in each of the figures correspond to the mean and upper and lower 90% confidence interval. Any point on the mean risk plot will show a magnitude of payment risk with 50% likelihood if the mean of the quality characteristic achieved in a specific project is equal to the corresponding in-situ density value.

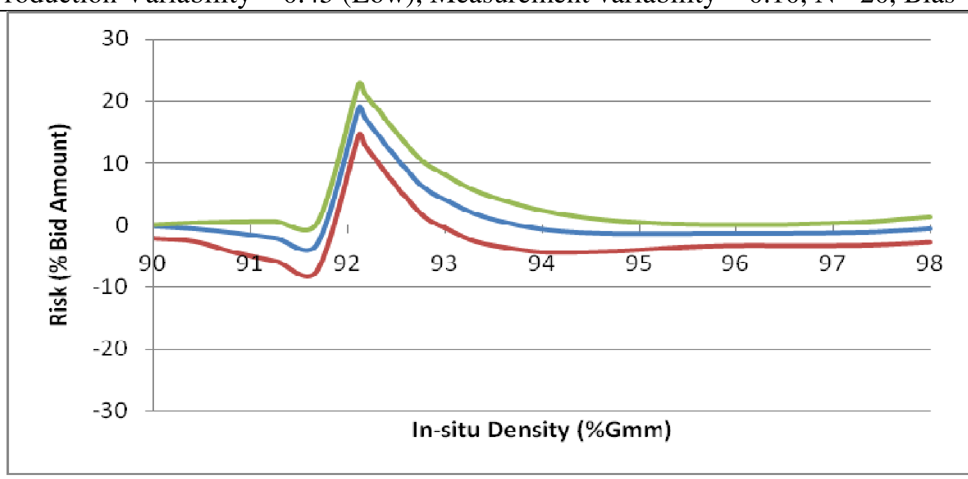
Figures 8.1 through 8.4 were selected to demonstrate some of the salient conclusions that can be derived through this analysis as listed below. This will be followed by analysis of variance and corresponding conclusions for the entire run matrix.

(1) The analysis presented in this chapter shows the effect of using pay formula instead of pay schedule. Since pay formula is a continuous function, risk is also smooth except for sharp points of inversion around the specification limits. The sharp inversion in the risk plot around the specification limits means that an error in the measured quality characteristic value putting it on one side of the specification limit would lead to a substantially different pay factor compared to that on the other side of the step for the same level of quality achieved. This is an undesirable feature of a QA program and efforts should be made to reduce the magnitude of risk and narrow the window in which inversion occurs even if it can be eliminated.

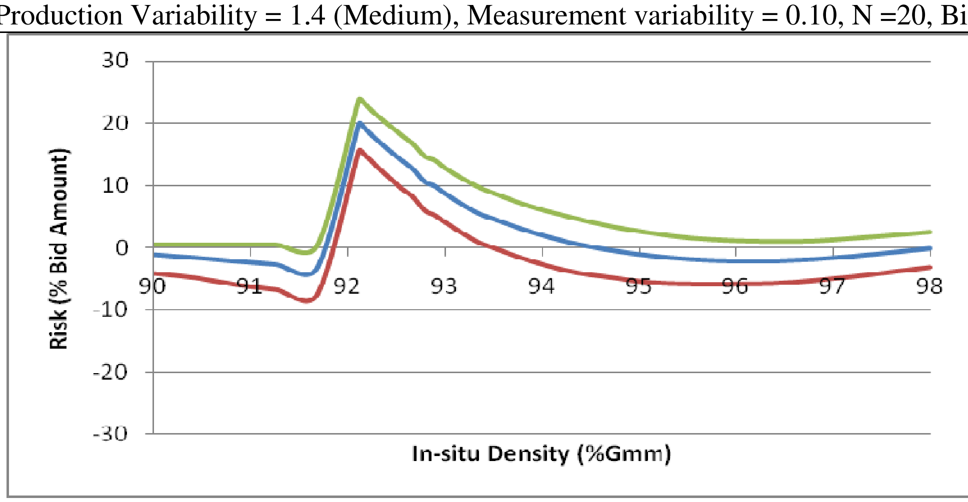
(2) Figure 8.1 shows the effect of production variability on risk. Plots a, b and c correspond to low, medium and high production variability with all other factors being the same.
   a. It is interesting to note that as the production variability increases from 0.45 to 1.3 the increase in payment risk is sharp.
   b. The increase in magnitude dampens for higher production variability values.
   c. With higher production variability the confidence interval around the mean risk widens considerably.
   d. More importantly there is appreciable amount of risk of overpayment or underpayment even when the contractor produces in the middle of the specification (allowable) window. In this case, the allowable window is 92% to 100%. This is highly undesirable. It has been observed that stricter allowable production variability on construction projects, within reasonable limits, can lead to lower production variability and thus reduction in this undesirable characteristic.
   e. It is important to note that risk is not equally balanced between the contractor and the agency. Especially, if the contractor's production mean is around the specification limit it is expected that the agency will overpay him. A good specification should have balanced risk. The advantage of balanced risk is that if pay factors are calculated on per lot basis, most of overpayment and underpayment would cancel each other for the aggregate payment for the entire project.



(a) Production Variability = 0.45 (Low), Measurement variability = 0.10, N =20, Bias=0 (2*)



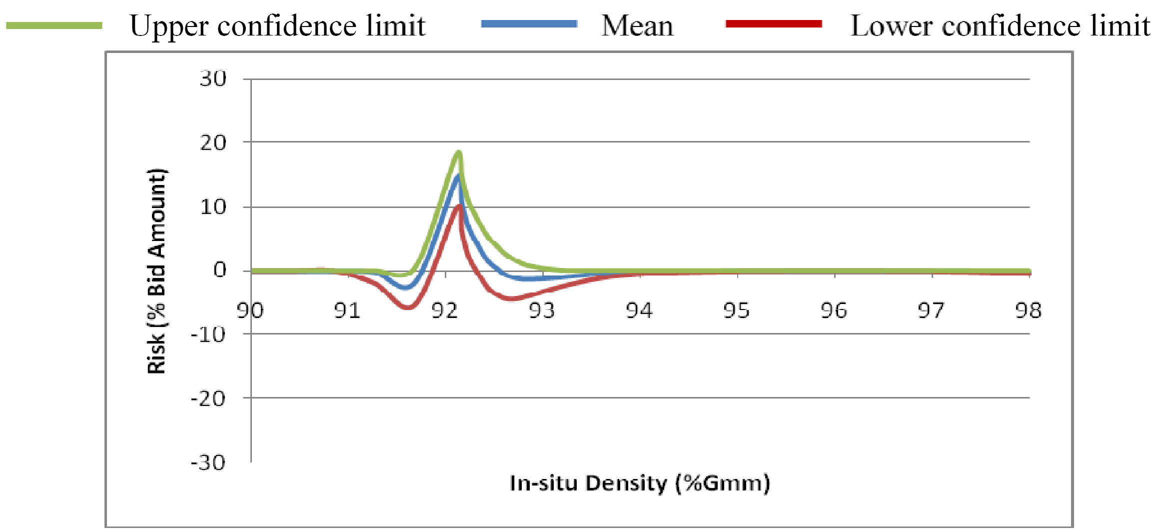
(b) Production Variability = 1.4 (Medium), Measurement variability = 0.10, N =20, Bias=0 (29)



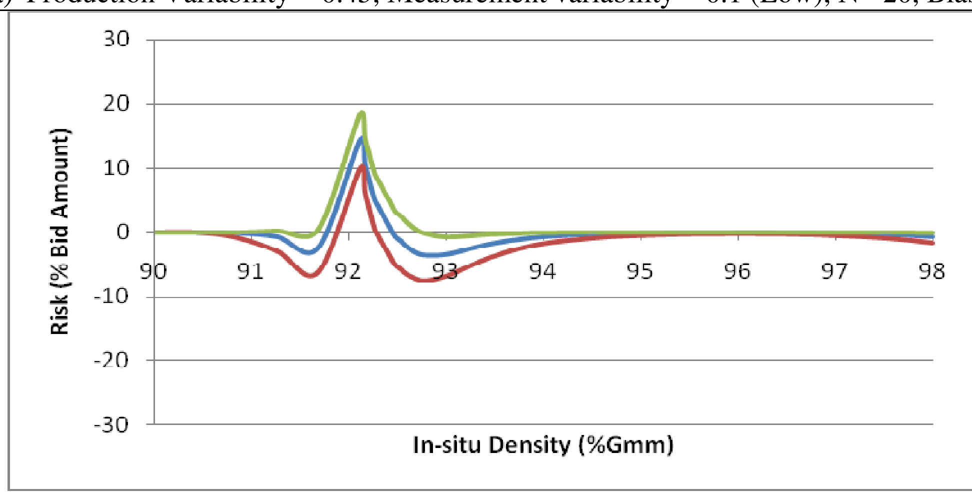
(c) Production Variability = 2.3 (High), Measurement variability = 0.10, N =20, Bias=0 (56)

Figure 8.1 Effect of production variability on risk for flexible pavements

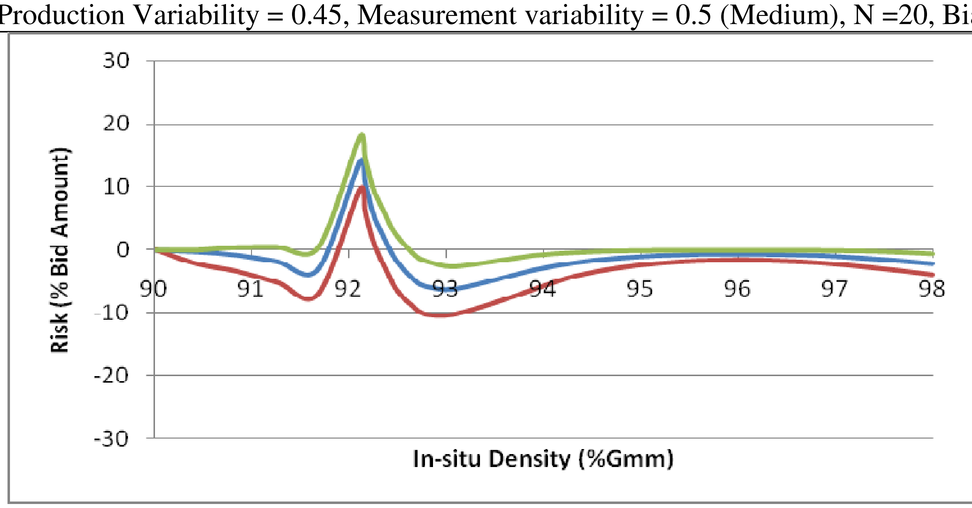
*Run number for the case in the run matrix (Table 8.1)



(a) Production Variability = 0.45, Measurement variability = 0.1 (Low), N =20, Bias=0 (2)

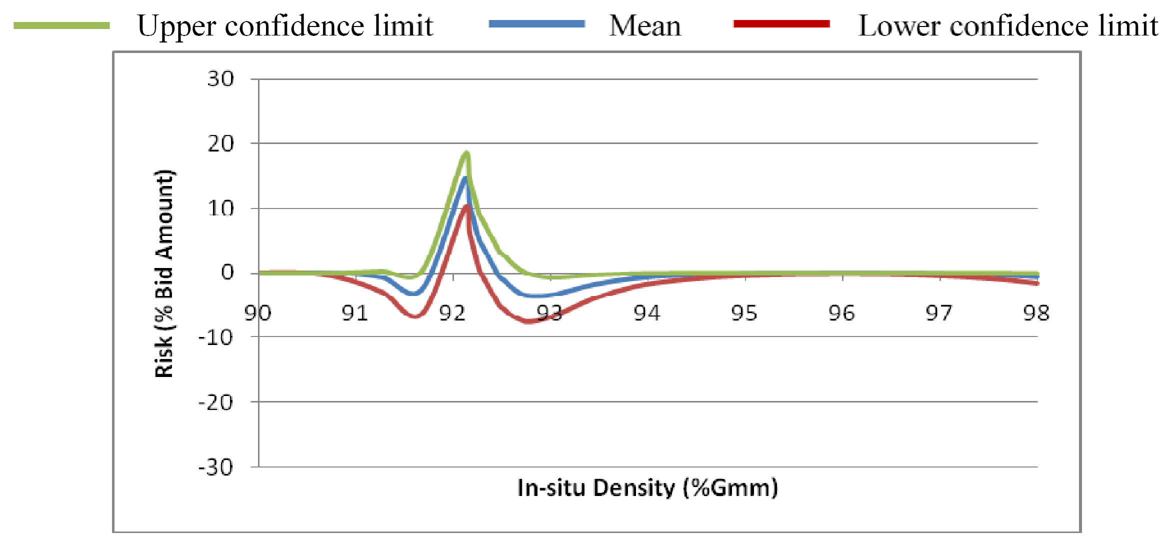


(b) Production Variability = 0.45, Measurement variability = 0.5 (Medium), N =20, Bias=0 (11)

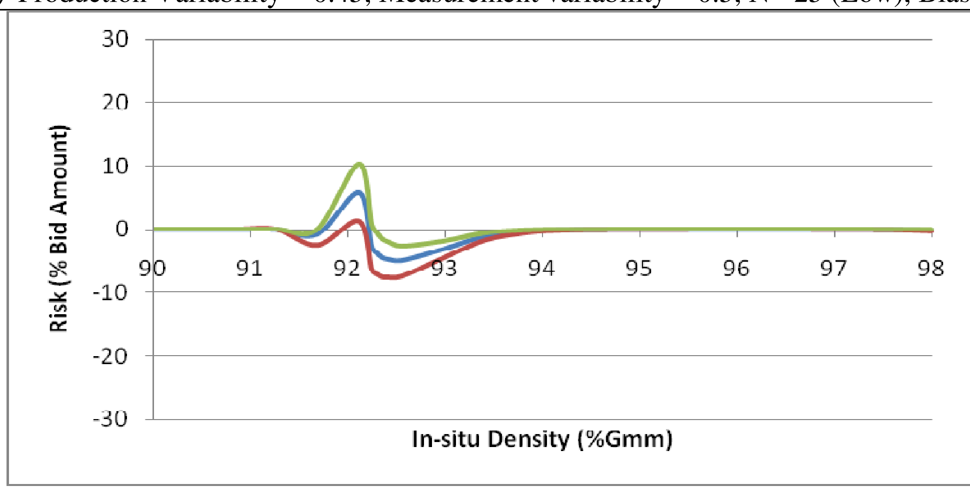


(c) Production Variability = 0.45, Measurement variability = 1.0 (High), N =20, Bias=0 (20)

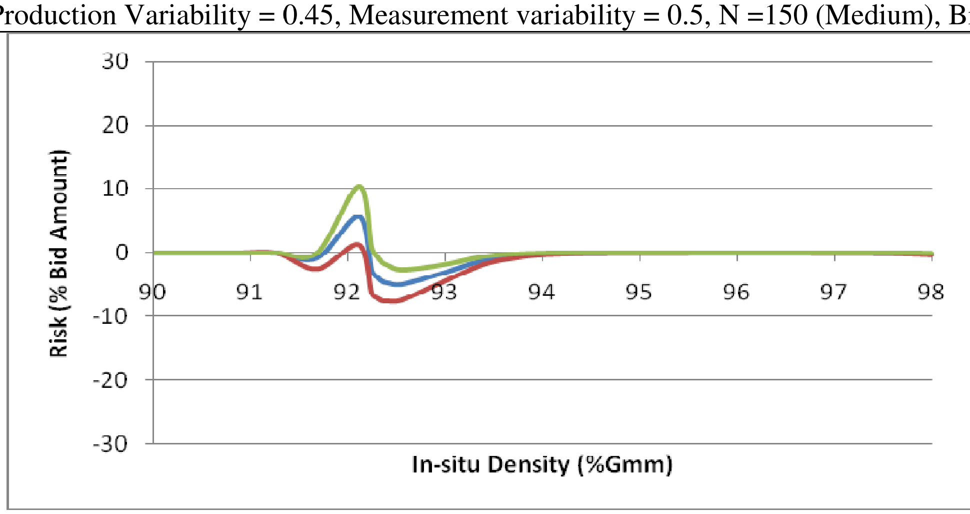
Figure 8.2 Effect of measurement variability on risk for flexible pavements



(a) Production Variability = 0.45, Measurement variability = 0.5, N =25 (Low), Bias=0 (11)

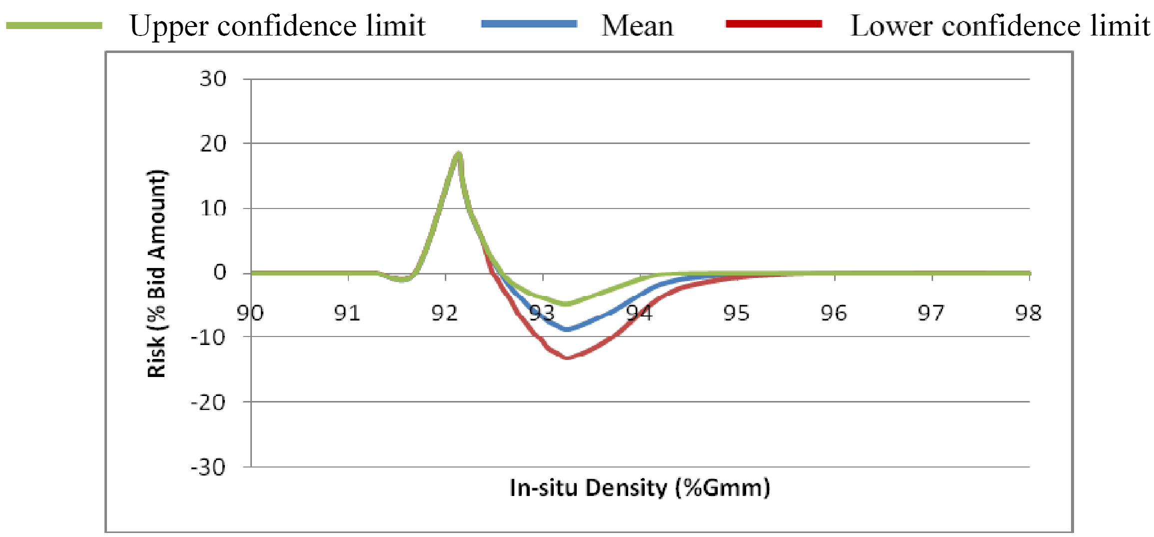


(b) Production Variability = 0.45, Measurement variability = 0.5, N =150 (Medium), Bias=0 (14)

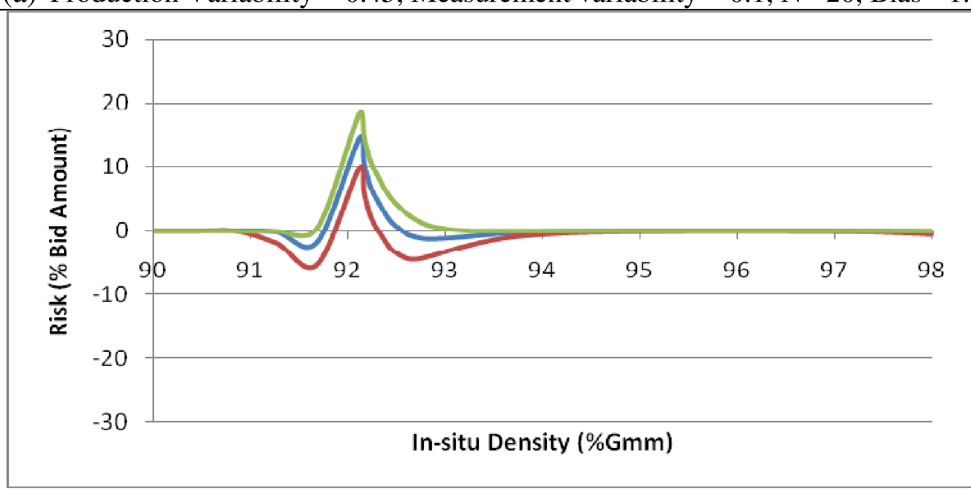


(c) Production Variability = 0.45, Measurement variability = 0.5, N =250 (High) , Bias=0 (17)

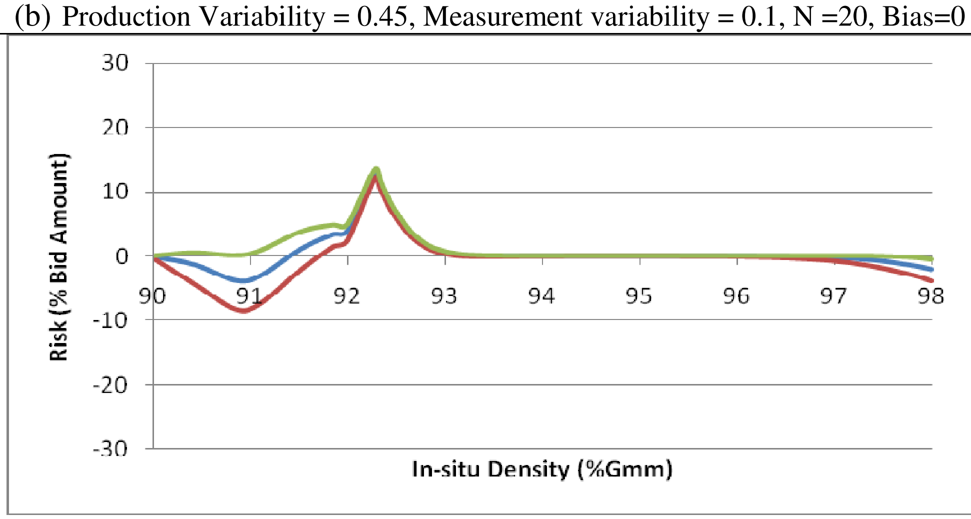
Figure 8.3 Effect of sample size on risk for flexible pavements



(a) Production Variability = 0.45, Measurement variability = 0.1, N =20, Bias=-1.2 (1)



(b) Production Variability = 0.45, Measurement variability = 0.1, N =20, Bias=0 (2)



(c) Production Variability = 0.45, Measurement variability = 0.1, N =20, Bias=1.2 (3)

Figure 8.4 Effect of measurement bias on risk for flexible pavements

(3) The plots in Figure 8.2 show the effect of measurement variability on payment risk.
   a. Increase in measurement variability leads to an increase in payment risk (compare plot (a) to plots (b) and (c)).
   b. The increase in risk because of measurement variability increasing from 0.1 to 0.5 is not as high as that because of increase in production variability. This is not always true. In this case, the measurement variabilities used are that of density measurement using pavement cores. This process has higher level of accuracy than most of the other tests used in quality assurance programs. This highlights the fact that more accurate test methods would lower risk in payment.

(4) Sample size has significant influence on payment risk as is evident from the plots presented in Figure 8.3. In all the three cases measurement variability was 0.5% which generally leads to appreciable level of payment risk. However, as the sample size becomes larger risk goes down considerably. In addition to the lowering of the risk, an increase in sample size also leads some amount of redistribution and therefore balancing of risk between the agency and the contractor. Redistribution of risk can be higher in other cases as was found with thickness specifications for the rigid pavements (see chapter 17).

(5) Figure 8.4 shows the effect of measurement bias on risk. The first plot in Figure 8.4 corresponds to a bias of -1.2%, the second plot to no bias and the third to a positive bias of 1.2%. A bias of -1.2% means that the agency consistently measures thickness to be lower than what it would even if measurement variability were present. In other words, the mean of a large sample of thickness measurements would be lower than the actual value by roughly 1.2%. If there were no bias and only measurement error was present, the mean of such a large sample of thickness measurements would be very close to the actual thickness. The three plots in Figure 8.4 clearly shows that bias can not only affect the magnitude of the risk, it can completely change the sign of the risk as well. When the bias is -1.2% (negative), risk just right of the lower specification limit is high and the contractor is expected to be underpaid. However, when bias is 1.2% (positive) the risk in the same region collapses to nearly zero and increases in the region just left of the specification window. Therefore, bias must be controlled carefully and eliminated from measured value through proper testing and calibration in the initial lot.

8.7 ANALYSIS OF VARIANCE (ANOVA)

The comparisons in the preceding section among different cases from the run matrix show the effect of the four variables namely production variability, measurement variability, sample size and bias. Analysis of variance can not only help quantify the effect of these factors on payment risk but also it can give insight into the interaction effects of these factors. However, to be able to run ANOVA, the risk plots by themselves cannot be used. The concept of Composite Risk Index (CRI) was presented earlier in this chapter. CRI helps assign one index value to a risk plot representing one case scenario considering several factors simultaneously. Without the use of such an index, thorough statistical analysis with such scenarios would be nearly impossible. Table 8.3 shows the values of CRI for all the 81 cases in the run matrix.

Table 8.4 shows the ANOVA table for CRI for all the 81 cases in the run matrix. Note that X1 through X4 represent the four factors being analyzed here and have been listed below the table. The following conclusions can be derived from the table.

(1) The p-values for main effects of the four factors show that all of them except production variability are statistically significant. This does not mean that production variability is not significant. One must look at interaction effects before making any conclusions.
(2) Looking at the main effects alone may indicate that sample size is relatively much more significant than the other three factors. Despite the fact that several interaction effects are also significant, high contribution to the total variance definitely indicate greater effect of that variable, sample size in this case, on the outcome of the experiment.
(3) Interaction effect between production variability and measurement variability leads to confounding results. For example, the solid line in Figure 8.5 shows that higher measurement variability leads to higher CRI which means higher risk when production variability is equal to 0.45 in this case. However, the trend is opposite when production variability is equal to 2.3 (high) while all other variables remain constant between the two cases.

Table 8.3 Calculated CRI Values for the Scenarios Identified in the Run Matrix.

Sample Size	Bias	Prod Var = 0.45			Prod Var = 1.4			Prod Var = 2.3		
		Meas var=0.1	Meas var=0.5	Meas var=1.0	Meas var=0.1	Meas var=0.5	Meas var=1.0	Meas var=0.1	Meas var=0.5	Meas var=1.0
20	-1.2	2.53	3.41	5.27	6.86	6.75	6.97	6.17	6.74	6.32
	0	0.92	1.79	3.36	3.51	2.51	4.96	6.00	5.65	5.55
	1.2	3.84	3.05	4.74	4.61	4.16	3.73	6.74	6.14	5.60
150	-1.2	2.98	3.99	10.55	4.53	8.32	11.60	6.98	8.25	9.67
	0	0.22	0.95	2.52	1.06	1.29	2.50	1.97	1.97	2.42
	1.2	3.75	3.98	4.11	4.12	4.49	4.14	4.91	4.76	4.81
250	-1.2	3.03	4.05	10.59	7.61	4.77	12.54	8.39	9.04	10.49
	0	0.16	0.99	2.18	0.94	1.08	2.28	1.48	1.70	2.59
	1.2	3.30	4.83	4.38	4.36	4.28	4.86	4.77	4.83	4.62

* Measurement Variability, in %

** Production Variability, in %

Table 8.4 ANOVA Table for CRI of all 81 Cases in the Run Matrix

Source	Sum Sq.*	d.f.	Mean Sq.	F	Prob>F
X1	8.6	2	4.3	1.6	0.22
X2	82.7	2	41.3	15.0	0.00
X3	56.4	2	28.2	10.2	0.00
X4	176.0	2	88.0	31.9	0.00
X1*X2	86.5	4	21.6	7.8	0.00
X1*X3	24.3	4	6.1	2.2	0.08
X1*X4	0.9	4	0.2	0.1	0.99
X2*X3	25.8	4	6.4	2.3	0.07
X2*X4	28.1	4	7.0	2.5	0.05
X3*X4	75.0	4	18.7	6.8	0.00
Error	132.4	48	2.8		
Total	696.6	80			

*Constrained (Type III) sums of squares.

X1: Production variability

X2: Measurement variability

X3: Sample size

X4: Bias

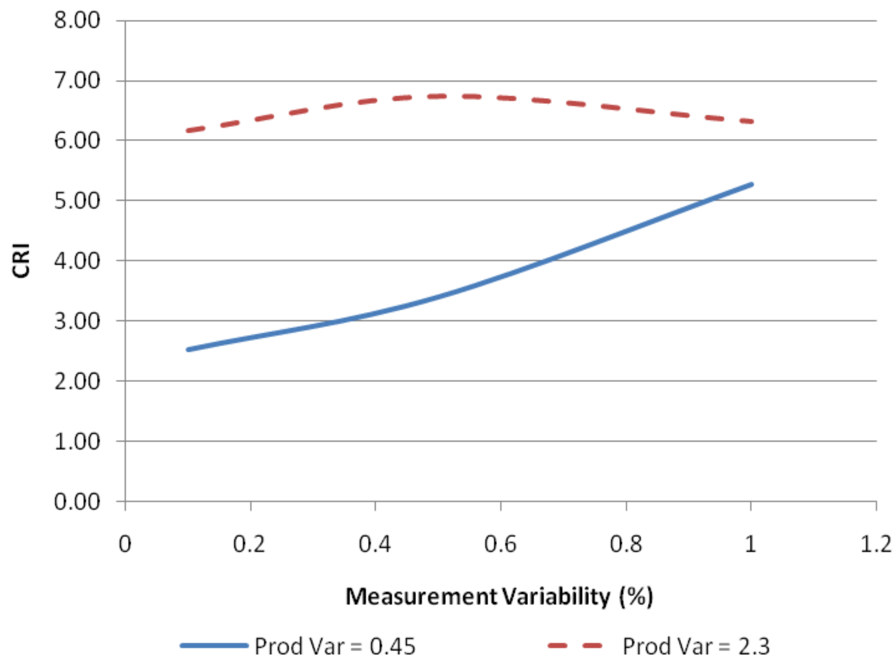


Figure 8.5 CRI as a function of production and measurement variability

(4) Measurement error can be controlled by the agency although it can most likely never be reduced to zero. If the measurement error is kept at minimum possible level, the risk in payment would go down while using the same QA program. Maintaining control over measurement variability is generally not too difficult. It would require that repeated measurements be taken in the beginning to assess the repeatability of the instrument/method and calibration be checked while doing the test section/initial lot in the beginning of the project.

(5) Bias seems to have the most drastic effect on payment risk because it can not only change the magnitude of risk but also alter the sign of risk. However, CRI does not catch this phenomenon because CRI treats the positive and negative risk as equally undesirable and does not discriminate between the two. This can be seen as a shortcoming of CRI. However, the authors have found through experience that it is very difficult, if not impossible, to design an index which is sensitive to the magnitude as well as sign of risk in the same plot. It means that most likely an accompanying risk index would have to be defined to cater to the needs of balancing risk between the agency and the contractor.

8.8 Conclusion

This chapter presents the details of the Monte-Carlo based simulation that was developed as part of this project to assess the current QA program of MDOT. The analysis conducted using the simulation showed that production variability, measurement variability, sample size and bias have significant influence on the risk in payment to be made to the contractor. This knowledge leads to identification of ways to reduce payment risk. The simulation can be used to analyze all other variables of a QA program and thereby improve it to achieve a lower risk of overpayment or underpayment. The analysis also showed that if production variability is high despite very low measurement variability and mean production being in the middle of the specification window risk exists. Therefore, not only the contractor should produce right around the target he should be encouraged to maintain low variability in production quality. This is also significant from the point of view of pavement performance, as has been shown in chapter 7.

Generally the test methods and instruments are standardized and calibrated in the beginning of the construction project. For longer projects, the instruments may develop bias with continued use over several days. Bias has a very significant effect on payment risk. Such situations can lead to disputes and even law suits. Therefore, bias must be avoided through suitable inspection of the functioning of the test instruments.

Chapter 9: Feedback Process to Design for HMA Pavements

The aim of a quality assurance program for pavement construction is to assess the quality of the pavement constructed by the contractor and pay the contractor accordingly. It invariably involves testing for various quality characteristics. The data collected through this effort should therefore represent the quality of the end product in comparison with the quality targeted through the design process. Therefore, the QA program cannot only be used for determining the payment to be made to the contractor but also to provide feedback to the design process itself.

A feedback process is required primarily to check if pavement materials and layers are being produced according to the design plan and if the variability is within expected and acceptable limits. The as-constructed QA data can then be used to update the main statistics of input design variables (mean and standard deviation), which can be fed back into the design system to revise the expected performance. Figure 9.1 schematically shows the feedback process.

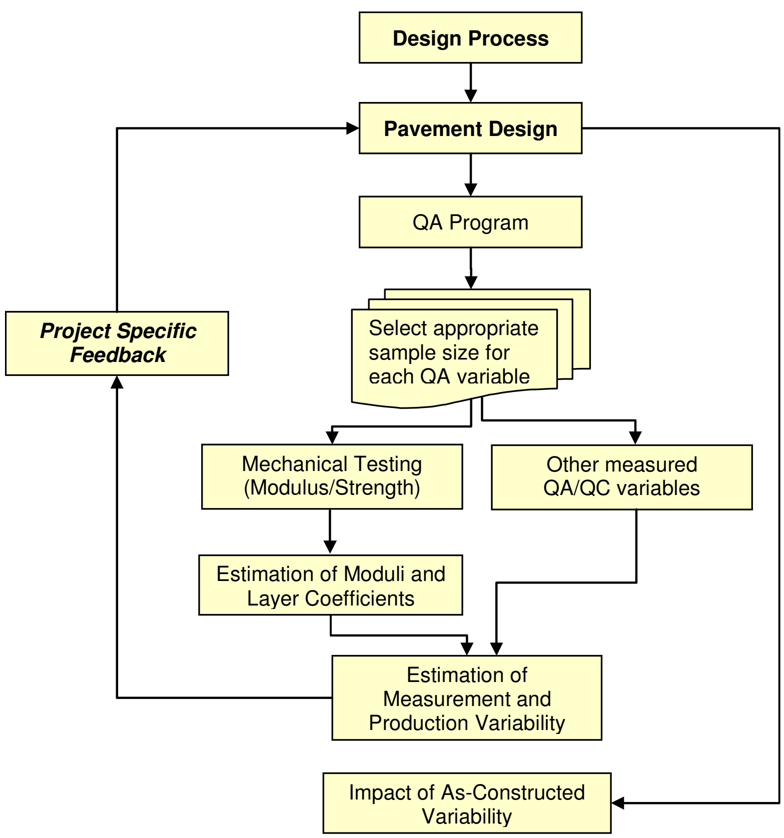


Figure 9.1 Flowchart showing feedback process for design

The following provides further description of the elements in this feedback process.

(1) *Selection of appropriate sample size:* One of the most important variables in the feedback process which needs to be optimized is the sample size. As the sample size becomes larger the confidence interval for a given design input (quality characteristic) tightens around the mean. The tighter the confidence interval the better the feedback process. This is discussed in detail in section 9.4.

(2) *Mechanical and/or material testing for modulus*: MDOT currently uses the AASHTO 1993 design guide for designing its pavements. Modulus values of the constructed pavement are required for the pavement structural design. The modulus values for the various layers can be indirectly measured through non-destructive testing in the field (e.g., FWD test) or directly measured through laboratory testing of cores obtained from the field. Alternatively, it can be estimated from material volumetrics and binder viscosity using established relationships (e.g., Witczak's E* equation). Note that the M-E PDG method also requires modulus testing in the form of dynamic modulus for the HMA layer and modulus of elasticity for the remaining layers.

(3) *Estimation of moduli and layer coefficients*: AASHTO 1993 uses the concept of layer coefficients (for asphalt pavements) to come up with the structural thickness. These layer coefficients need to be estimated from modulus values, which can be estimated from mechanical and/or material testing (see item (2) above). In the M-E PDG framework, the E* value is estimated through backcalculation using FWD test data (level 1) or from Witczak's equation (levels 2 and 3). Therefore, in the latter case, the feedback will consist of updating the input volumetrics and binder viscosity or dynamic shear modulus.

(4) *Use of other QA and QC data for design*: Quality characteristics data obtained through a QA program from pavement construction projects can be used as input for design either directly (in the MEPDG) or indirectly through correlations (in the current AASHTO 1993 design procedure). An effort can also be made to collect contractor's QC data as long as they are deemed comparable. This is described in sections 9.1 through 9.3.

(5) *Estimation of measurement and production variability*: The overall variability that construction data shows has two components, namely (a) measurement variability and (b) production variability. Production variability is the actual variability in the constructed pavement because of variability in material, construction practices and equipment, and climatic conditions. When various tests are used to determine the level of quality achieved, production variability gets masked with measurement variability because of error in the test equipment and/or process. However, only production variability affects pavement performance. Therefore, in the beginning of a construction project, measurement variability should be estimated for the various test methods to be used under the quality assurance program. This will help estimate actual production variability in the constructed pavement.

(6) *Impact of as-constructed variability*: Production variability will lead to variability in pavement performance. In the AASHTO 1993 design procedure, the loss/gain in design life (ΔPSI) can be directly back calculated using the design equation or by iteration using the Darwin design software. In the M-E PDG framework, the software can be used directly to predict the loss/gain in pavement life.

The following sections describe in detail the use of actual QC and QA data from MDOT projects to (1) validate the assumption of normality in QC/QA data, which is required for all the analyses used in this report, (2) demonstrate the applicability of using QC data in addition to QA data, and (3) investigate the effect of sample size on the feedback process to design.

9.1 MDOT Data Analysis

Michigan uses a standard format for collecting data for its QA program from pavement construction projects. All the data is stored in the form of Microsoft Excel Workbooks. An effort is also made to collect contractor's QC data as well in the same workbook. All the QA data from the projects contracted in 2008 were collected by the authors. There were 200 such files in total. Figure 9.1 shows a sample of the QA data from one of the project workbooks.

A typical pavement project is divided into lots which are further subdivided into sublots. Most of the sublots are composed of roughly equal amount of material used in the bound pavement layers. A specific number of samples is collected from each sublot for each quality characteristic. In the Excel worksheet shown in Figure 9.2, a set of rows are assigned for each sublot, and each row represents a sample within a sublot. The snap shot of the table represents lot number 1; each additional lot would be represented by a similar table. The worksheet has been designed in a way that one worksheet should be used for one mix type on any project. Most of the time, one mix type represents one HMA layer within the pavement. In those instances where the mix design formula is changed during the course of the project, separate worksheets may be used for different mix designs. However, it was found that the data entry in the actual worksheets was not as systematic as intended. For example, in many instances the same mix type for a project had data entered in more than one workbook with few lots worth of data in each of them. There is a possibility that not all of the data was documented in these sheets.

The first step in the analysis process was to consolidate and extract good data. Consolidation of data involved identifying all the data corresponding to one mix type of one project from different Excel files and putting them together in one place. Filtering involved eliminating duplicate data and in some cases those which seemed to be in error. This is a fairly arduous process since it needs to be done for each of the mix types and each of the quality characteristics separately, which means checking more than 1000 sets of data and moving them as required. At the end of the consolidation and filtering process a total of 127 mix worksheets from different projects were obtained. The next step was to analyze this set of data for various characteristics.

9.2 Assumption of Normality

This report documents the development of simulation methods to analyze MDOT QA specifications and provide for ways to improve it. An underlying assumption behind these simulations is that construction project data are normally distributed. This assumption is made based on observations made by other researchers in the past (Hall and Williams, 2002). However, this assumption is so crucial that it should be verified in every case as much as possible. Since the authors had access to MDOT QA data, this exercise was performed.

Figures 9.3 and 9.4 show plots from QA data corresponding to several mixtures from different projects to verify the assumption of normality. These plots have been presented for visual appreciation of the characteristics of QA data, which is very important from a simulation point of view. This will be followed by Chi-square goodness of fit test results for all the data being analyzed. In Figures 9.3 and 9.4 blue stars represent cumulative probability distribution functions for the actual QA data. The red dots represent the same for ideal normal distribution. If the two sets follow the same trend the data being tested can be assumed to be normally distributed.

MDOT HMA PWL Worksheet v1.10 - FY 2008 5/27/08
(effective for Contracts after Oct 07)

Control Section	13081	Binder Method	Back Calculated
Job Number	60527A	Plant Location	Kalamazoo
Testing Lab	MDOT/SW Reg	Plant ID	110-05
Lab Location	Kalamazoo	Layer	Top
Mix Type	3E50	Stress Level	Standard
Mix Design #	08MD160	Application	Mainline
Contractor	Aggregate Industries		

=Sublot RQL
=Lot RQL
=Density Outlier
=Action Limit
=Sus. Limit

	Binder	AV	VMA	Density
Low Spec	- 0.40 of JMF	3.00	13.00	92.00
High Spec	+ 0.40 of JMF	5.00	15.00	----------
Target	----------	4.00	----------	----------
AQL	90	90	90	90
RQL	50	50	50	50

							Target		MDOT QA					Contractor Random QC				MDOT QA Dispute Resolution ***Fill out only if there is a dispute***				
Sample Date	Mix Type	JMF	Production Type	Lot	Sublot / Core #	Sublot Tons	Binder Content	AV	QA Binder Content	QA AV	QA VMA	QA Density	QA Density Sublot Avg	QC Binder Content	QC AV	QC VMA	QC Density	Dispute Binder Content	Dispute AV	Dispute VMA	Dispute Density	Dispute Density Sublot Avg
5/8/2008	3E50	08MD160	UDP	1	1A	750.00			4.85	3.77	13.64	93.30		4.80	4.25	13.96	94.00					
5/8/2008	3E50	08MD160	UDP	1	1B							93.50					93.30					
5/8/2008	3E50	08MD160	UDP	1	1C							92.70										
5/8/2008	3E50	08MD160	UDP	1	1D							94.80	93.58									
5/8/2008	3E50	08MD160	UDP	1	2A	750.00			4.80	3.61	13.38	92.90		4.70	3.77	13.31	91.50					
5/8/2008	3E50	08MD160	UDP	1	2B							91.30					96.40					
5/8/2008	3E50	08MD160	UDP	1	2C							91.80										
5/8/2008	3E50	08MD160	UDP	1	2D							91.40	91.85									
5/12/2008	3E50	08MD160	UDP	1	3A	750.00			4.88	4.05	13.95	94.90		4.98	4.45	14.53	93.40					
5/12/2008	3E50	08MD160	UDP	1	3B							93.70					94.90					
5/12/2008	3E50	08MD160	UDP	1	3C							95.40										
5/12/2008	3E50	08MD160	UDP	1	3D							93.00	94.25									
5/14/2008	3E50	08MD160	UDP	1	4A	750.00			5.04	4.02	14.27	92.40		5.12	4.69	15.05	92.60					
5/14/2008	3E50	08MD160	UDP	1	4B							92.40					92.50					
5/14/2008	3E50	08MD160	UDP	1	4C		4.99	4.00				93.00										
5/14/2008	3E50	08MD160	UDP	1	4C							92.90	92.68									
5/14/2008	3E50	08MD160	UDP	1	5A	750.00			4.75	3.72	13.37	93.30		4.86	4.88	14.65	94.60					
5/14/2008	3E50	08MD160	UDP	1	5B							92.80					93.30					
5/14/2008	3E50	08MD160	UDP	1	5C							93.00										
5/14/2008	3E50	08MD160	UDP	1	5D							91.90	92.75									
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		
		08MD160	UDP	1																		

Figure 9.2 Sample MDOT QA data worksheet

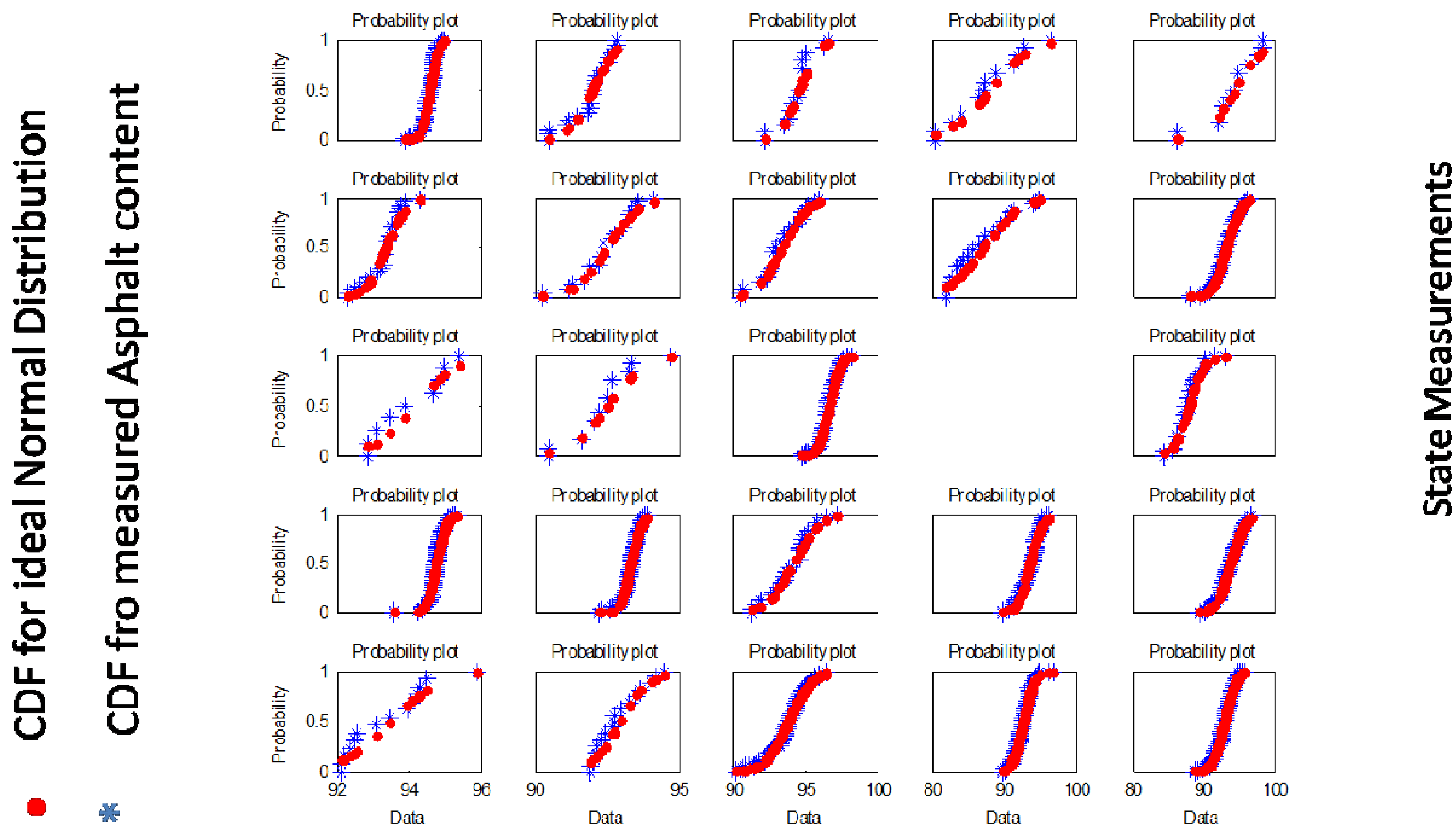


Figure 9.3 Test of assumption of normality – Asphalt content QA data

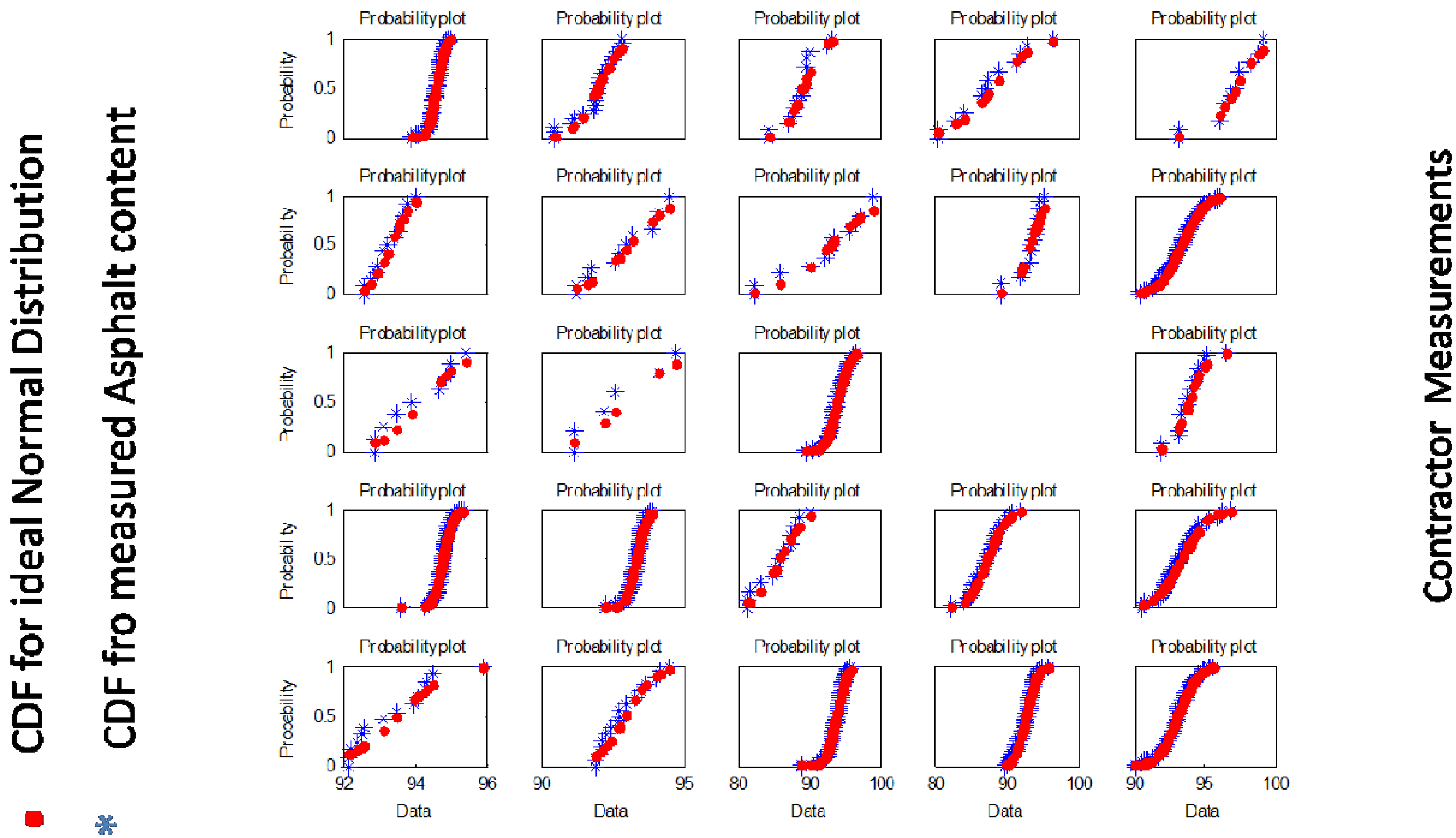


Figure 9.4 Test of assumption of normality – Asphalt content QA data (Continued)

An advantage of the visual representation is that one can see the parts of the data which deviate from normal behavior and account for that in the simulation so that the artificially generated data has the same characteristics as the real data. In the case of asphalt content, the QA data seems to be very close to ideal normal distribution in the majority of cases.

Figures 9.5 through 9.8 show results from Chi-square goodness of fit test for asphalt content, density, VMA and air voids QA data. The null hypothesis in this test is that the data are a random sample from a normal distribution with mean and variance estimated from the data, against the alternative that the data are not normally distributed with the estimated mean and variance. The result "h = 1" corresponds to p-value less than 5%, which means that the null hypothesis can be rejected at the 5% significance level. The result "h = 0" corresponds to p-value more than 5%, which means that the null hypothesis cannot be rejected at the 5% significance level. Therefore, an h-value of 0 means that the data is normally distributed at 5% significance level. It can be seen from the plots that the vast majority of the mix data are normally distributed in all the cases. For the sake of brevity cumulative density plots for other quality characteristics have not been included.

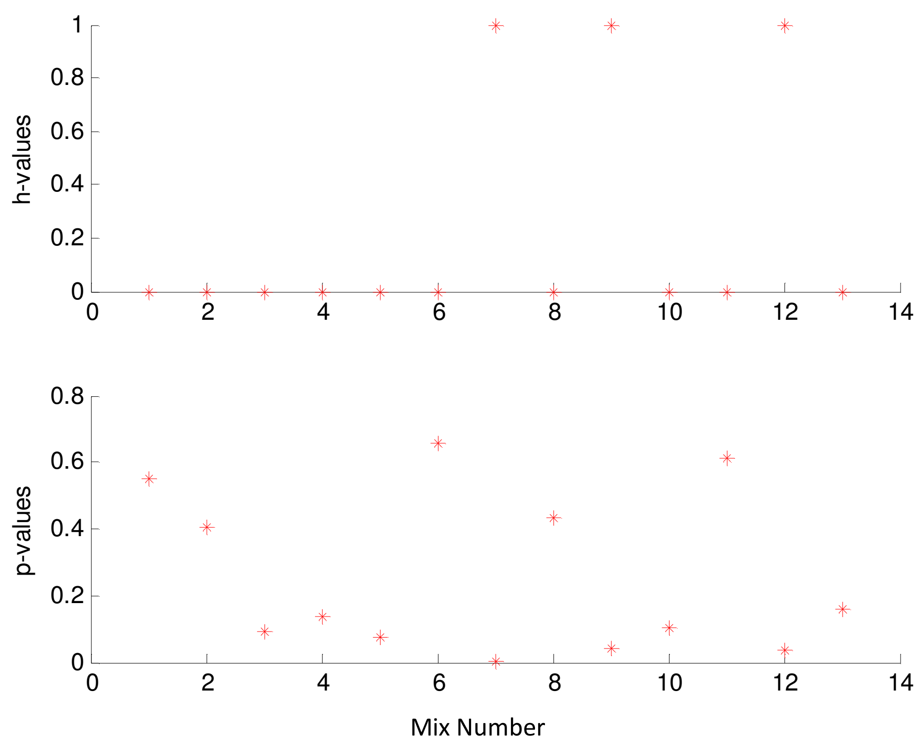


Figure 9.5 p and h – values from Chi-square goodness of fit for asphalt content QA data

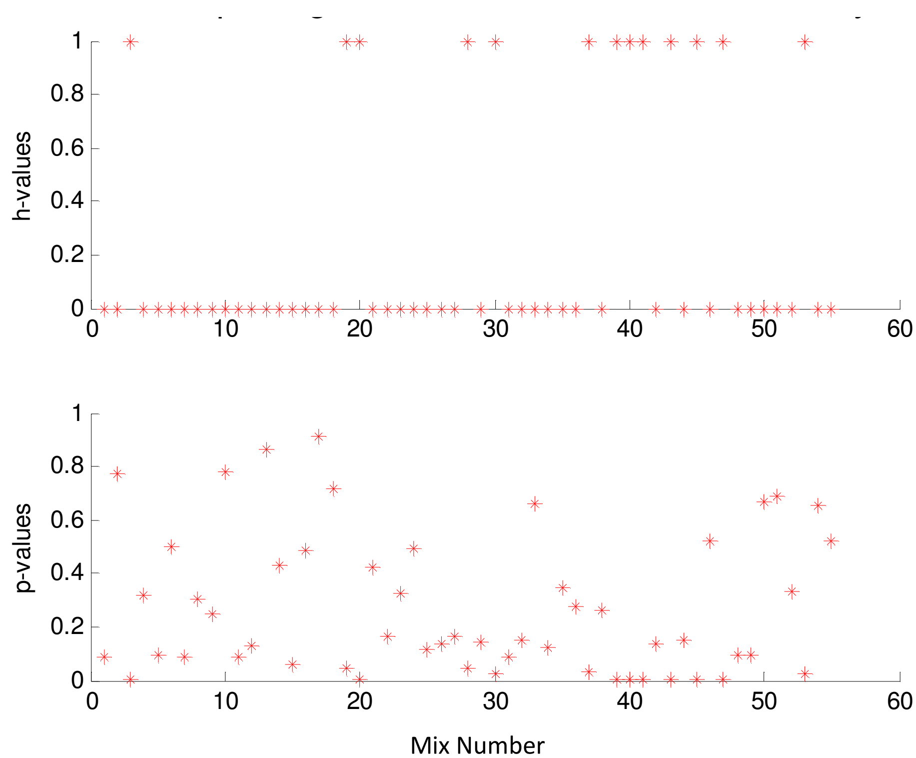


Figure 9.6 p and h – values from Chi-square goodness of fit for density QA data

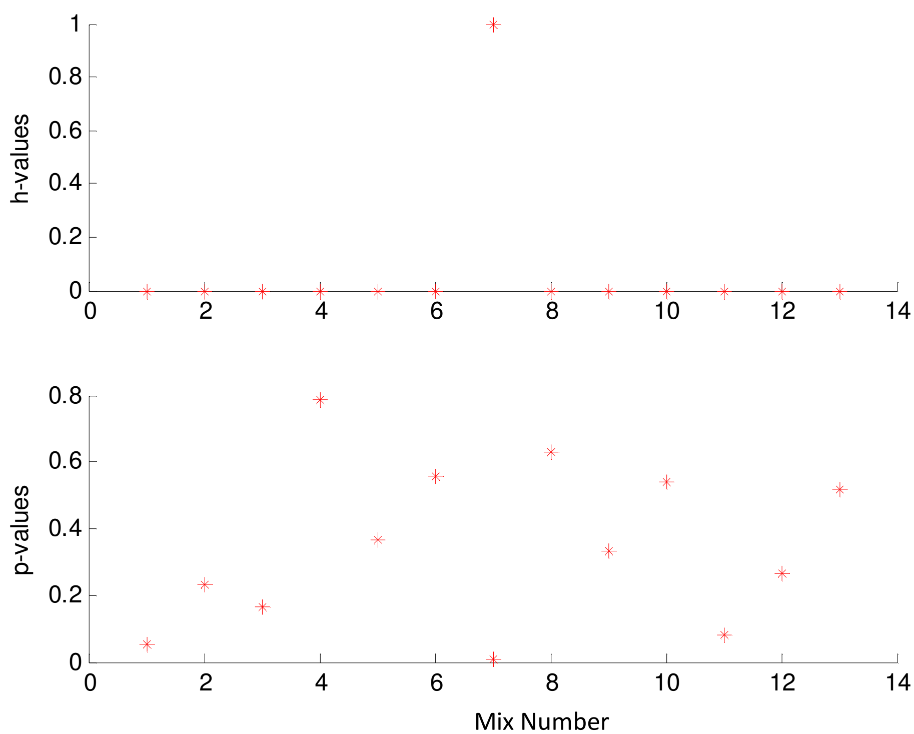


Figure 9.7 p and h – values from Chi-square goodness of fit for VMA QA data

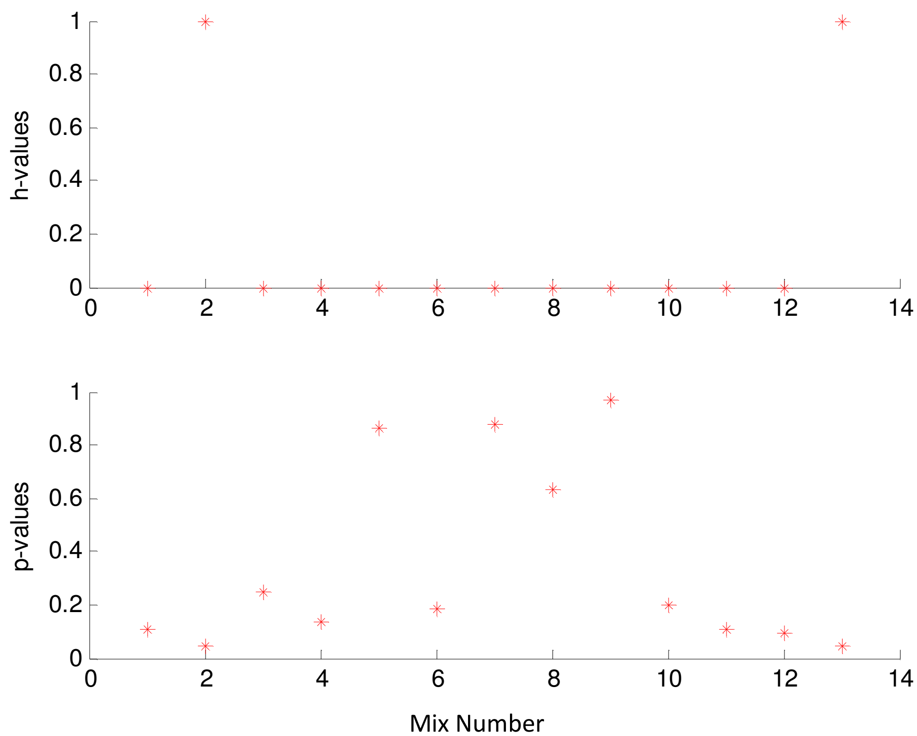


Figure 9.8 p and h – values from Chi-square goodness of fit for Air voids QA data

9.3 Comparison of QA and QC Data

The lot and sublot identification used for the QA testing remain the same for QC testing. Therefore, it is expected that the two sets of data should be similar, i.e. two samples should belong to the same population. Most of the times QC data is strictly used by the contractor only for quality control purposes and the state or the highway agency performs its own set of tests for QA. The state or highway agency determines payment for the contractor using QA data only. However, some states have procedures in place to check the correctness of QC data and use it to determine a pay factor. The main advantage of such a process is that the sample size used for pay factor calculation is larger and, therefore, can lead to lower risk in pay. MDOT does not use QC test data in pay factor calculation. An exercise was performed in this research to do a preliminary feasibility study to use QC test data (in addition to QA data) in pay factor calculation.

The first exercise in the feasibility study would be to test the assumption that QC and QA data are indeed two samples from the same population. If this assumption does not hold for any reason QC data cannot be used in pay factor calculation. Figures 9.9 through 9.12 present comparisons between state and contractor data for the corresponding mixes to get a visual appreciation of the similarity or differences.

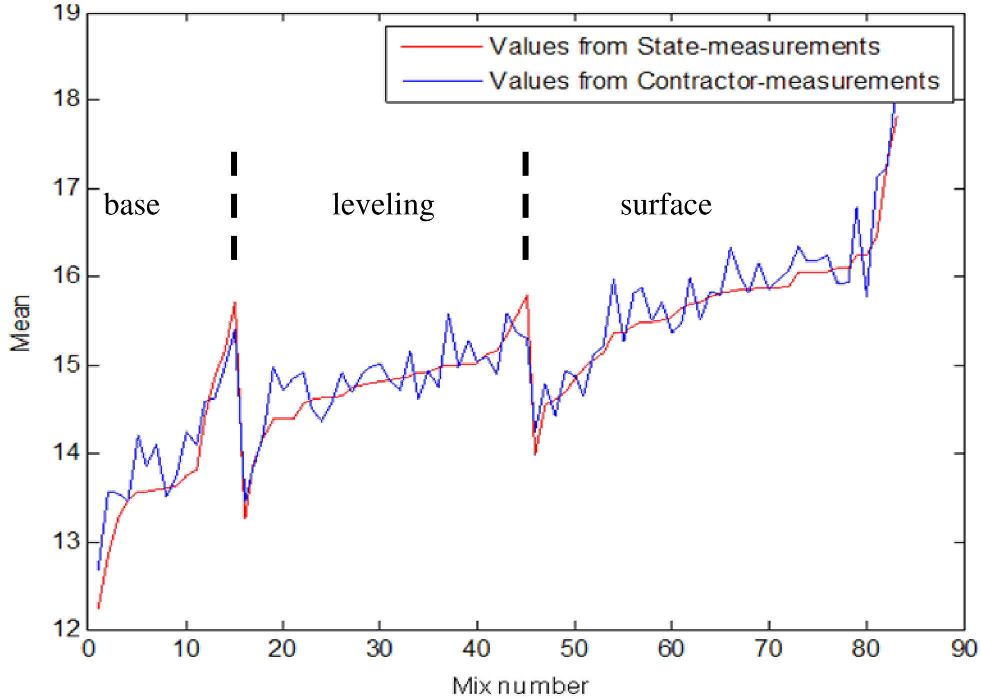


Figure 9.9 Comparison of state and contractor measured mean VMA for different mixes

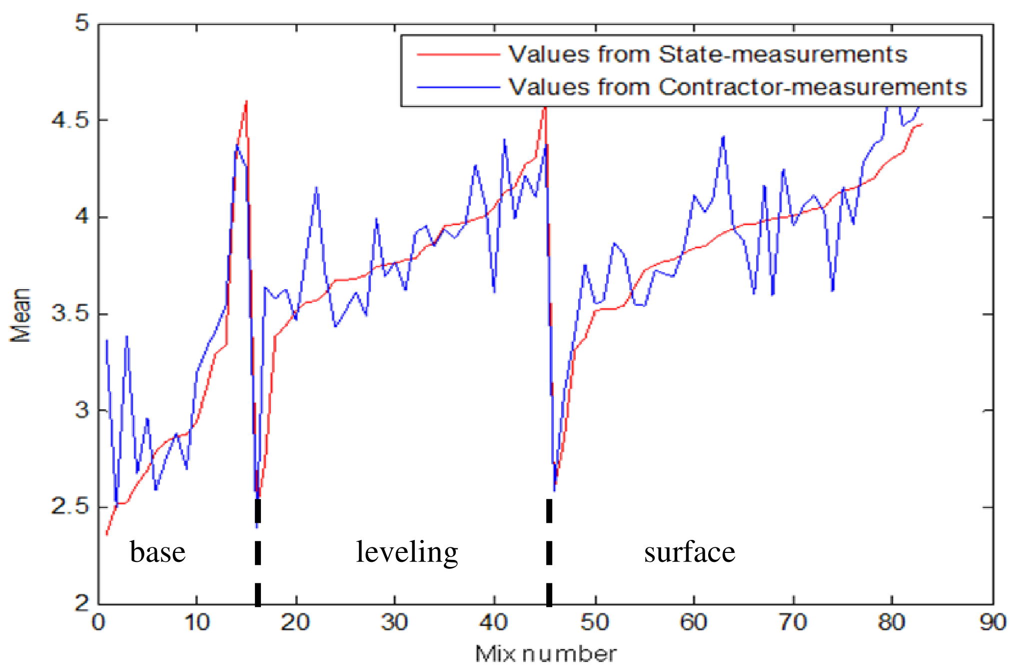


Figure 9.10 Comparison of state and contractor measured mean Air Voids for different mixes

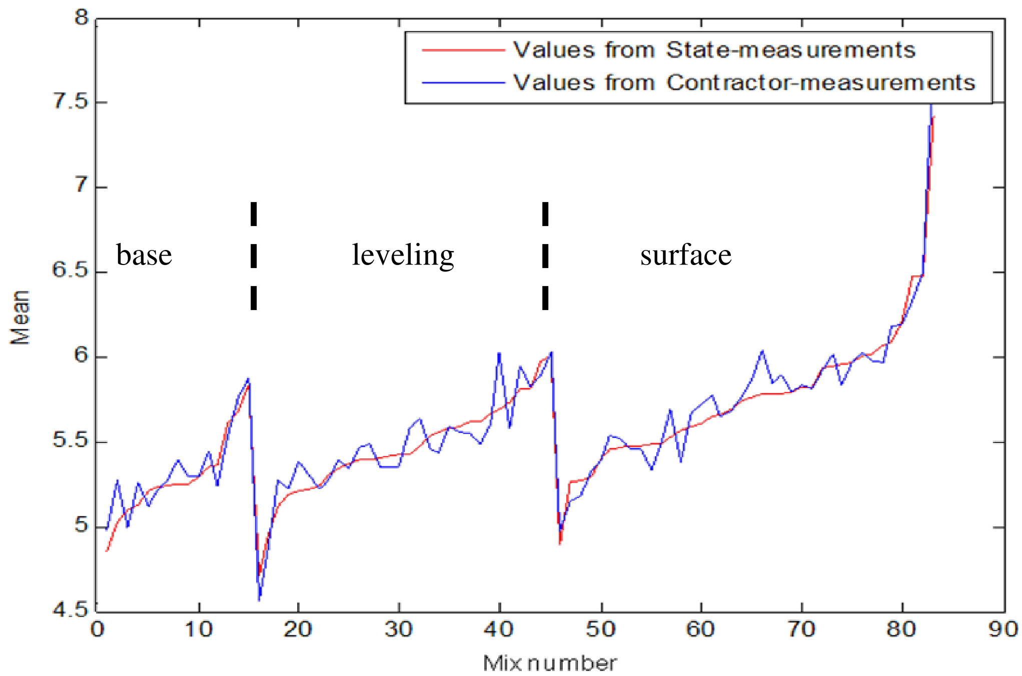


Figure 9.11 Comparison of state and contractor measured mean Asphalt Content for different mixes

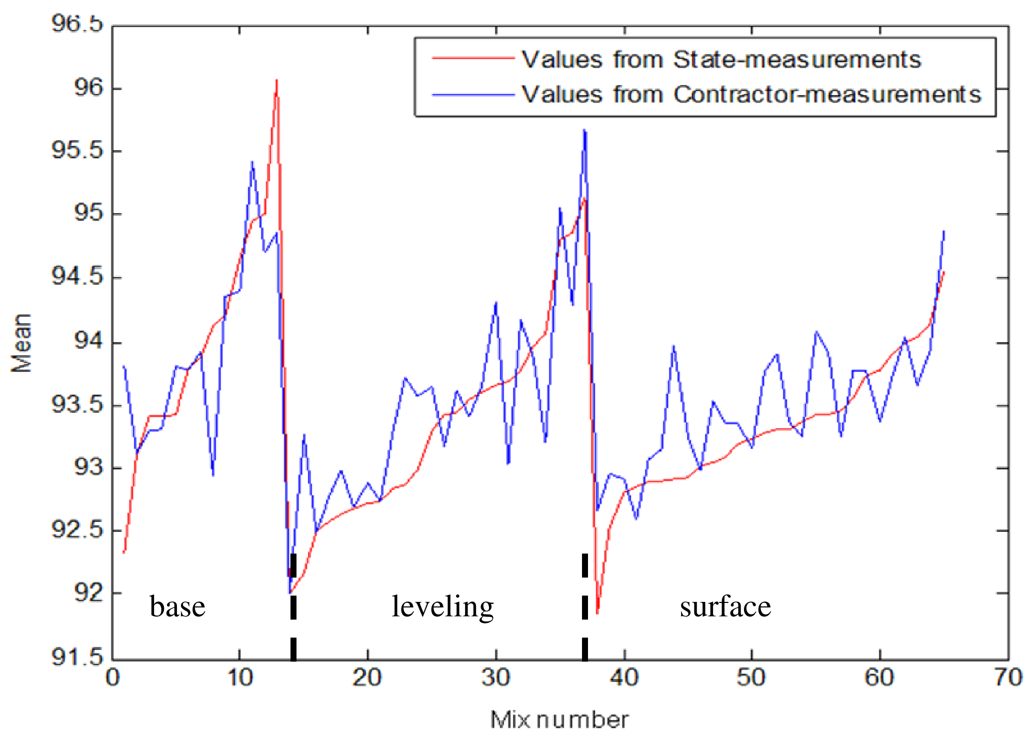


Figure 9.12 Comparison of state and contractor measured mean In-situ density for different mixes

The data used in this analysis were collected from different projects which were not related to each other. Therefore, there is no expected trend between any two projects or mixes. However, data for similar layers should be analyzed together. For example, one could put the base courses in one category, or lump the binder and surface courses in another category. If the state and contractor data corresponding to different mixes are plotted in any random sequence, both series will look more or less like the blue lines in the plots above. Because of the high fluctuation in values from mix to mix, it will be difficult to see any trend in comparing the two data sets. Therefore, the following steps were performed to obtain the above plots:

(1) Categorize data for the base course, leveling course and surface course (3 categories).

(2) Within each category sort the state measured data in ascending order and plot it along with the corresponding contractor data for that mix under the same mix number.

The plots for all four quality characteristics show the contractor data to be close to the corresponding state measured values. However, in some cases (e.g., relative in-situ density for the surface course, Figure 9.12), the contractor values seem to be higher. This is an example of bias.

Figures 9.13 and 9.14 show comparisons of standard deviation, 5th and 95th percentiles, and sample size between the state and contractor data. The purpose of presenting these plots is not only to compare the data but also to demonstrate the range of these statistics observed in the field. For example, in the case of air voids, more than 95% of the projects have a standard deviation less than 1%. Assuming an average standard deviation of 0.6%, if the QA specifications have an allowable window of ±1% (i.e., 1.67 times the average standard deviation) then percent-within-limits for an average project would be 90%, which translates into 100% pay. However, half of the projects which have higher standard deviation would attract penalty.

Sample size for the vast majority of mixes is less than 30. Section 9.4 in this chapter will show the impact of sample size on the error when estimating the variability and the resulting difference in pavement performance and error in pay factor calculation.

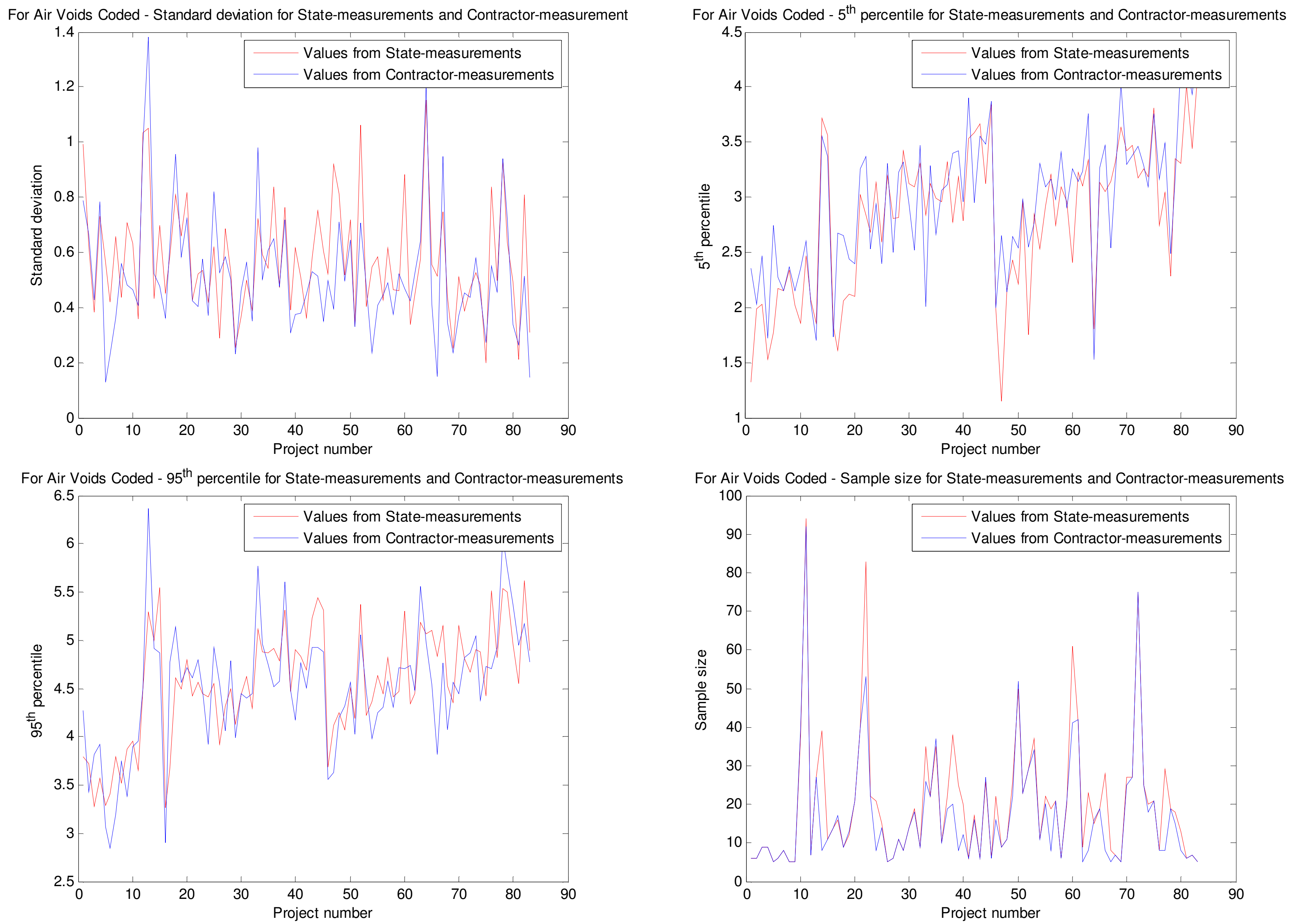


Figure 9.13 Comparison of state and contractor measured Air Voids for different mixes – standard deviation, 5th & 95th percentile and sample size

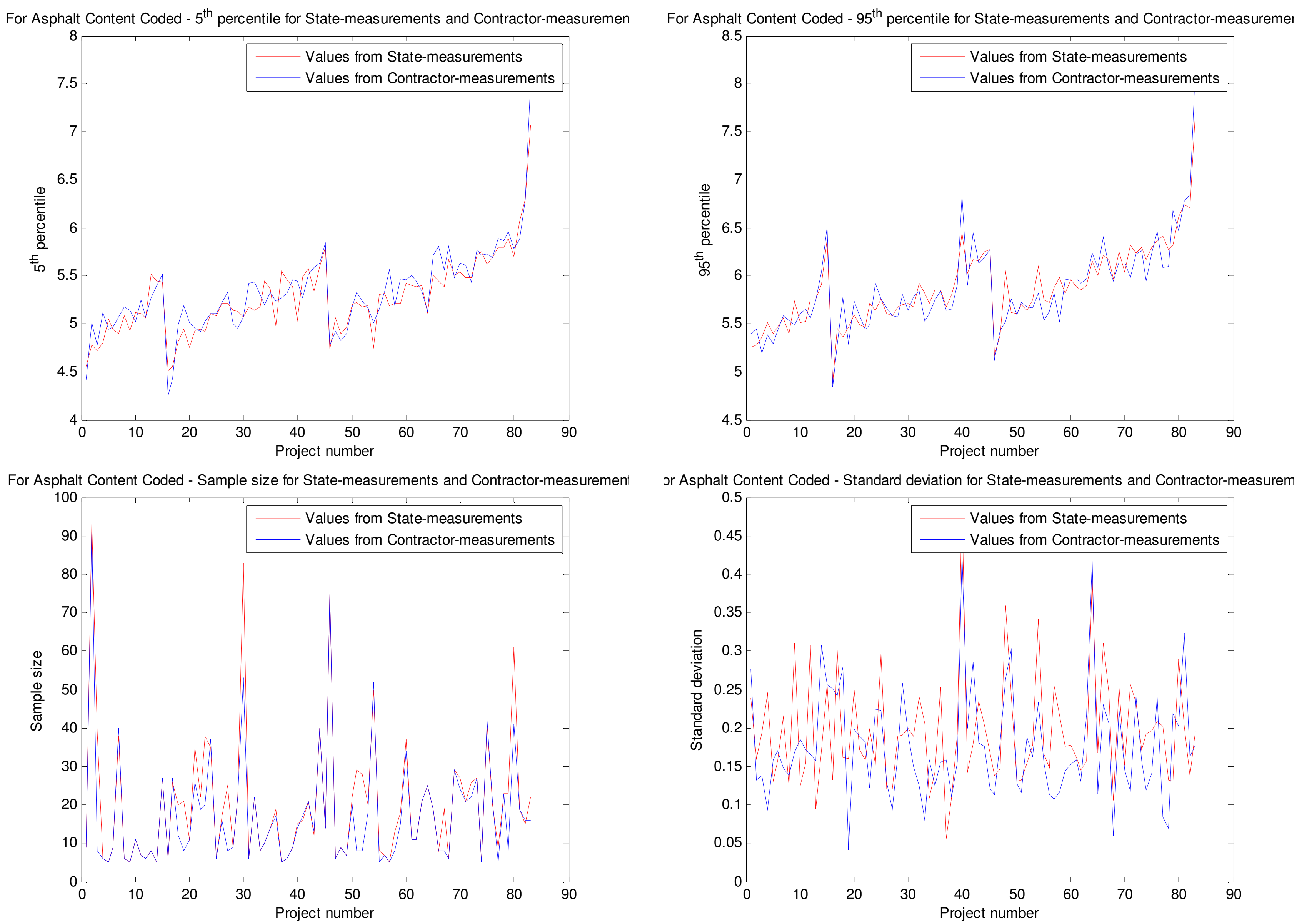


Figure 9.14 Comparison of state and contractor measured Asphalt Content for different mixes – standard deviation, 5th & 95th percentile and sample size

9.4 Effect of Sample Size on Feedback Process using Simulation

Chapter 8 of this report describes the development of Monte-Carlo based simulation to assess risk in payment to be made to the contractor in MDOT QA program. The same fundamental concept of simulation can also be employed to develop an optimal feedback process to design. Section 9.2 establishes the validity of synthetically generated data being similar to the actual field data collected from MDOT construction projects, both being normally distributed. The advantage of the synthetically generated data is that the error in the data is known a priori. Therefore, simulation using such synthetically generated data can be used to assess the extent to which data collected in the field represents true pavement quality compared to the design target. This section presents the details of this exercise.

One of the most important variables in the feedback process which needs to be optimized is the sample size. The feedback simulation developed in this project for flexible pavement construction was used to estimate the statistics enumerated below as a simultaneous function of sample size and mean of quality characteristic. Each scenario was simulated 10,000 times to identify the distribution of these statistics, allowing for a probabilistic study.

(1) Error in estimating the mean of a quality characteristic (density, air voids, asphalt content etc.) in a lot.

(2) Error in estimating the variability (standard deviation) in quality characteristics in a lot, and

(3) Risk in pay factor calculation for a lot.

All the above assessments were performed for a lot because MDOT QA program determines pay factor on a lot basis. Figures 9.15 through 9.17 show the above mentioned statistics as a function of sample size and mean quality characteristic (Q/C). The middle surface in Figure 9.15 represents the mean error in estimate of the mean quality characteristic. The surfaces that are above and below represent the 90% confidence interval for the error. The following observations can be made from this plot.

(1) The mean of the error is essentially equal to zero for all sample sizes and all values of mean Q/C

(2) It can be clearly seen that as the sample size becomes larger the confidence intervals tighten around the mean. The tighter the confidence interval the better the feedback

process would be. A tight confidence interval means that the estimate of the error lies within a small window, or in other words there is high probability that the error would be close to zero since the surface representing the mean of the error is essentially flat at zero level.

(3) The 90% confidence interval of error is tighter for higher mean Q/C, especially on the positive side of the mean. This happens because it would take much more compactive effort to reach 97% or higher density (density is being analyzed in this case).

The decision regarding optimal sample size for feedback will have to be made by MDOT. This is because defining the level of risk that MDOT is willing to take to save testing time (by not having a very large sample size) is a function of many considerations that only MDOT can weigh.

The plots presented in Figures 9.15 through 9.17 are helpful in understanding the trend in error, and therefore, how the optimal size should be selected. However, to be able to make this decision, MDOT would need a table or a plot showing a relationship between sample size and a metric tangible enough to make decisions (e.g., the width of the confidence interval).

Figure 9.16 shows the error in the estimate of variability (standard deviation) for different sample sizes and varying mean values of the quality characteristics. The overall behavior is similar to that observed in the case of the error in estimate of mean Q/C. The difference is quite noticeable when the sample size is small. For small sample size, the error is negative for all values of mean Q/C. In other words, a small sample size would lead to an underestimation of variability.

Figure 9.17 shows risk as a function of sample size and mean quality characteristics. The perspective view of this plot was chosen to be different from the preceding two plots because of the complex geometry of the surface. Therefore, the left horizontal axis which represented sample size in other plots has Mean Q/C and the right horizontal axis has Sample size instead of Mean Q/C. The effect of sample size on risk is similar to that for the estimate of mean and standard deviation.

Figures 9.18 through 9.20 were generated to get a better understanding of the magnitude of the effect of sample size on the three statistics being considered in this analysis.

Figures 9.18 and 9.19 have very similar trends and show that with increasing sample size the error in the estimate of mean and variability falls sharply in the beginning and then the reduction in error slows down. Therefore, MDOT will have to decide on the sample size beyond which the reduction in error is not worth the extra effort of having a larger sample size to increase gain.

Figure 9.20 shows the reduction in risk across the entire range of the quality characteristic with increasing sample size. It is noticeable that for certain mean values of the quality characteristic (around the lower specification limit) an increase in sample size leads to a small decrease in risk whereas the reduction in risk is appreciably higher for mean quality characteristics away from the lower specification limit. This is because of the sharp increase in the magnitude of risk around the specification limits that was observed in risk analysis presented earlier.

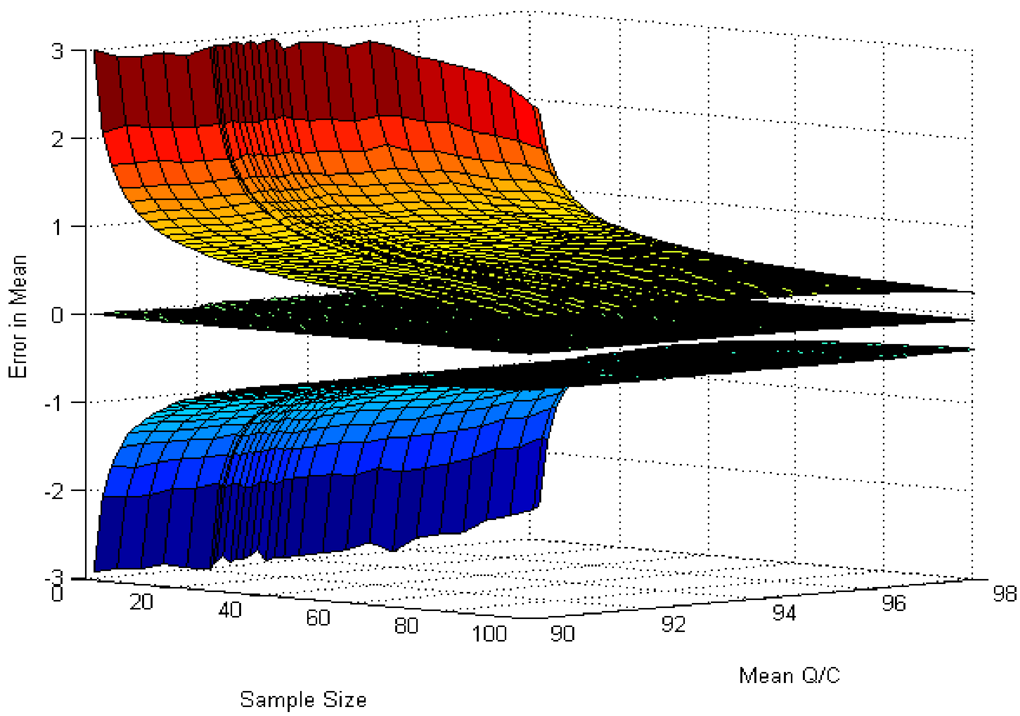


Figure 9.15 Error in estimate of mean quality characteristic with 90% confidence interval from feedback process

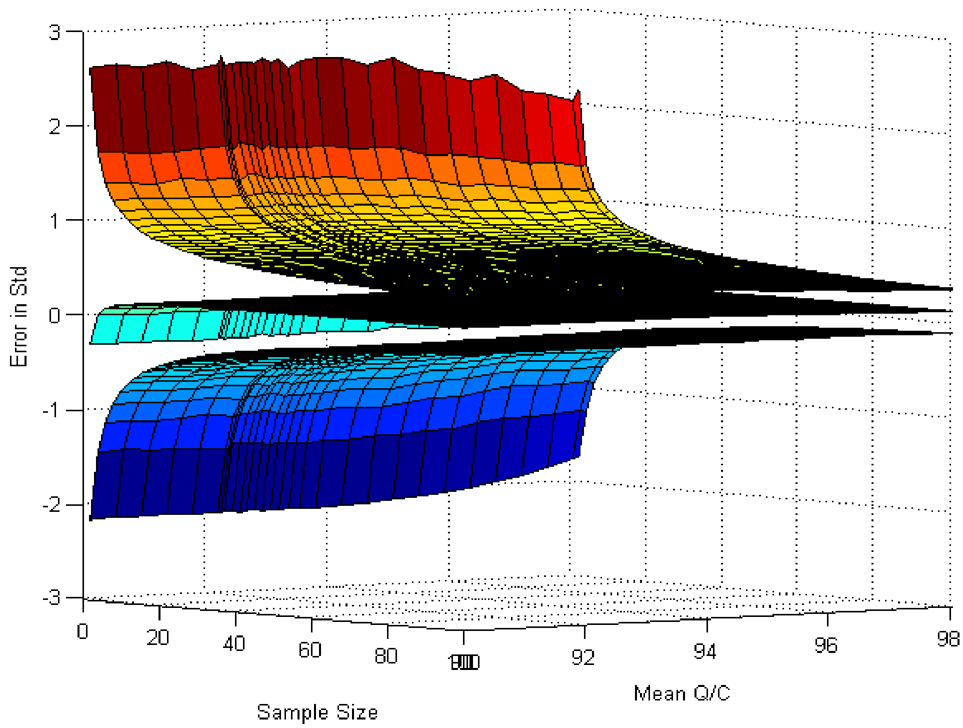


Figure 9.16 Error in estimate of variability in quality characteristic with 90% confidence interval from feedback process

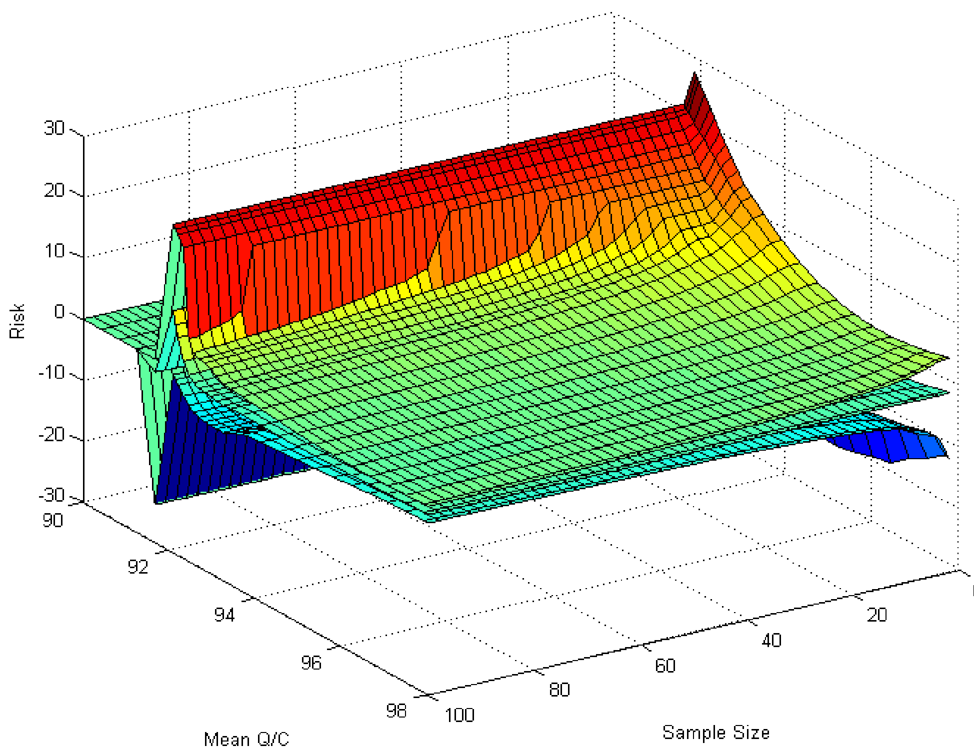


Figure 9.17 Error in estimate of payment risk with 90% confidence interval from feedback process

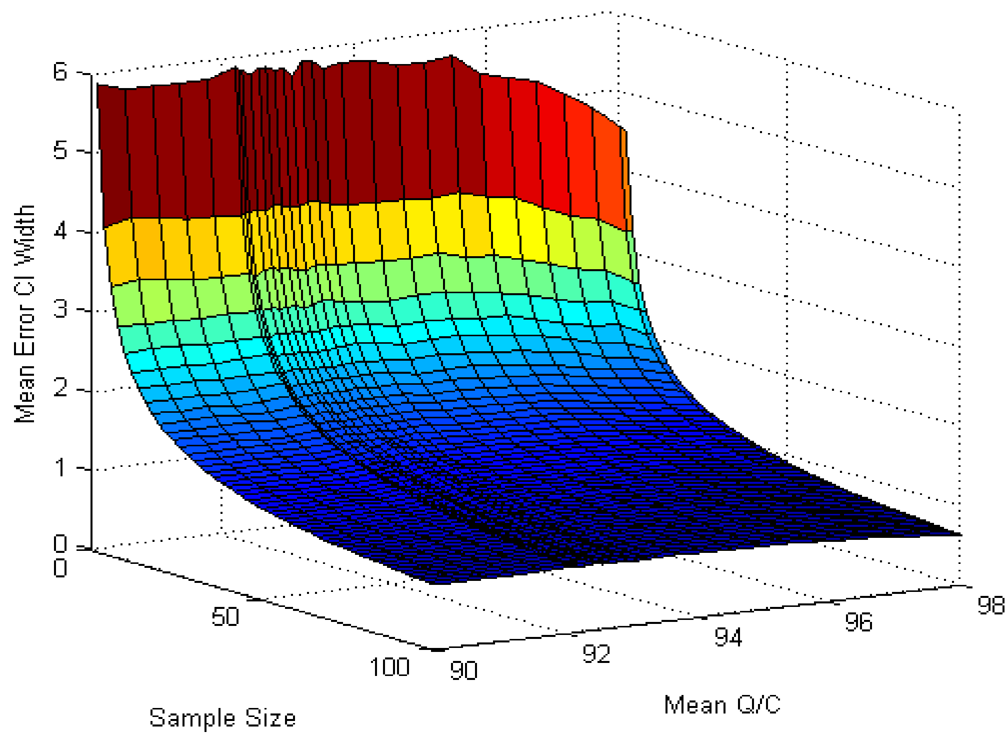


Figure 9.18 Width of 90% confidence interval in estimate of Q/C mean from feedback process

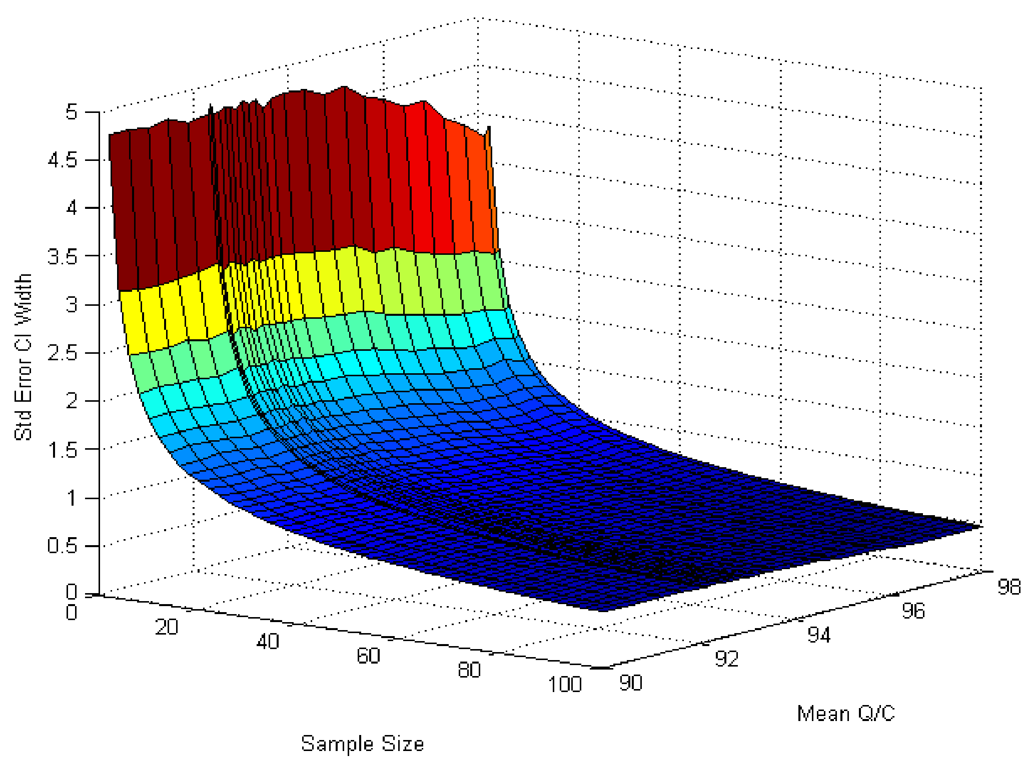


Figure 9.19 Width of 90% confidence interval in estimate of Q/C variability from feedback process

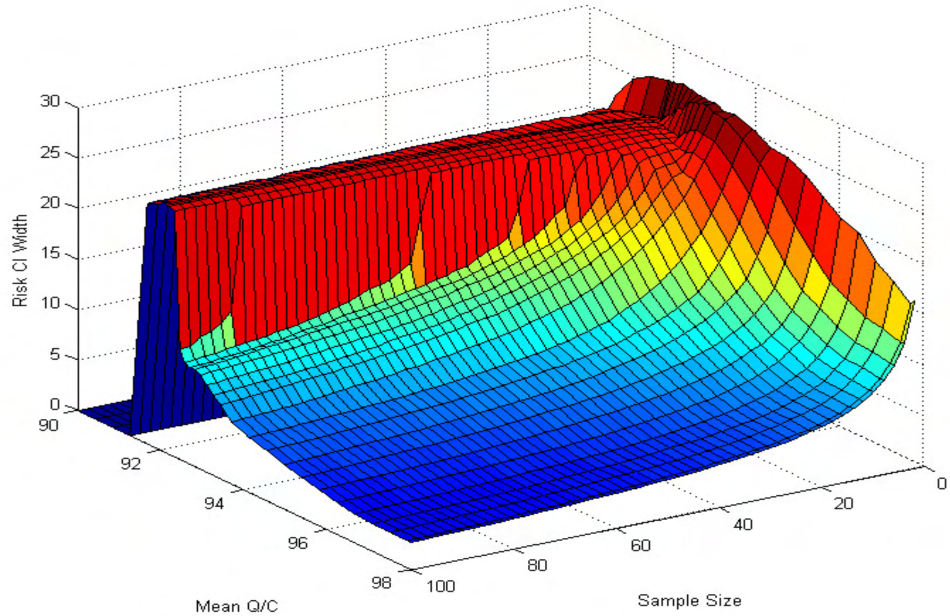


Figure 9.20 Width of 90% confidence interval in estimate of payment risk from feedback process

Tables 9.1 through 9.3 present the same information as the preceding three figures (Figures 9.18 through 9.20) but in tabular form to be able to see the magnitudes of confidence intervals, which would enable making decisions. One should consider the width of the confidence interval for the entire range of the quality characteristic for deciding on the sample size since different projects will have different values for the mean Q/C, although the sample size for the feedback process will probably have to be the same. It can be simplified one step further if we study the average width of confidence interval for the entire range of Q/C versus sample size. The last row in all these tables shows the average values. The sample sizes used in the analysis were varied from 2 to 100 with a step of 2. For the sake of brevity these tables present only a few selected values.

Figures 9.21 through 9.23 present the summarized form of the results obtained from this analysis. Figure 9.21 shows the maximum error in estimate of mean in 90% of the cases for different sample sizes. In other words, for example, if the sample size is 60 the maximum error in 90% of the cases will be lower than 1% (in-situ density in this case). However, if the sample size is only 8 the error in mean could be as high as 2.8%. If only two samples were collected the error in mean could be as high as 5.6% in ninety percent of the cases. This not only shows the benefit of having a larger sample size, but also quantifies the benefits in terms of reduction in error.

Table 9.1 Width of 90% confidence interval of error in estimate of mean quality characteristic for different sample sizes

Mean Q/C	Sample Size											
	2	**8**	**14**	**20**	**26**	**32**	**38**	**44**	**50**	**60**	**80**	**100**
90.0	5.8	2.9	2.2	1.8	1.6	1.5	1.3	1.2	1.1	1.0	0.9	0.8
90.4	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
90.8	5.8	3.0	2.2	1.9	1.6	1.4	1.3	1.2	1.2	1.1	0.9	0.8
91.3	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.2	1.1	0.9	0.8
91.7	5.8	2.9	2.2	1.8	1.6	1.5	1.3	1.3	1.2	1.1	0.9	0.8
92.1	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.2	1.1	0.9	0.8
92.2	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.2	1.1	0.9	0.8
92.2	5.8	2.9	2.2	1.8	1.6	1.5	1.3	1.2	1.1	1.1	0.9	0.8
92.3	5.8	2.9	2.2	1.9	1.6	1.5	1.3	1.2	1.2	1.1	0.9	0.8
92.4	5.8	2.9	2.2	1.8	1.6	1.5	1.3	1.2	1.2	1.0	0.9	0.8
92.5	5.9	2.9	2.2	1.8	1.6	1.5	1.3	1.2	1.1	1.1	0.9	0.8
92.6	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.2	1.1	0.9	0.8
92.7	5.7	2.9	2.2	1.9	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
92.9	5.9	2.9	2.2	1.8	1.6	1.5	1.3	1.2	1.2	1.0	0.9	0.8
93.0	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.2	1.1	0.9	0.8
93.2	5.8	3.0	2.2	1.9	1.6	1.5	1.3	1.2	1.2	1.1	0.9	0.8
93.3	5.9	2.9	2.2	1.9	1.6	1.5	1.3	1.3	1.2	1.1	0.9	0.8
93.5	5.9	2.9	2.2	1.8	1.6	1.5	1.3	1.2	1.2	1.1	0.9	0.8
93.9	5.8	2.9	2.2	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
94.3	5.8	2.9	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
94.7	5.7	2.8	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
95.2	5.5	2.8	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
95.6	5.6	2.8	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8
96.0	5.6	2.7	2.1	1.7	1.5	1.4	1.3	1.2	1.1	1.0	0.9	0.8
96.4	5.3	2.7	2.0	1.7	1.5	1.3	1.2	1.1	1.1	1.0	0.8	0.7
96.9	5.2	2.5	1.9	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8	0.7
97.3	5.0	2.5	1.9	1.6	1.4	1.2	1.1	1.1	1.0	0.9	0.8	0.7
97.7	4.6	2.3	1.8	1.5	1.3	1.2	1.1	1.0	0.9	0.9	0.7	0.7
98.0	4.5	2.3	1.7	1.4	1.3	1.1	1.0	1.0	0.9	0.8	0.7	0.6
Avg.	5.6	2.8	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8

Table 9.2 Width of 90% confidence interval of error in estimate of variability in quality characteristic for different sample sizes

Mean Q/C	Sample Size											
	2	8	14	20	26	32	38	44	50	60	80	100
90.0	4.8	2.1	1.6	1.3	1.2	1.0	0.9	0.9	0.8	0.7	0.6	0.6
90.4	4.6	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.8	0.6	0.6
90.8	4.8	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.8	0.6	0.6
91.3	4.7	2.1	1.6	1.3	1.2	1.0	0.9	0.9	0.8	0.7	0.6	0.6
91.7	4.8	2.2	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.1	4.8	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.2	4.7	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.2	4.9	2.2	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.3	4.7	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.4	4.8	2.2	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.5	4.8	2.2	1.5	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.6	4.9	2.2	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
92.7	4.7	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.8	0.7	0.6
92.9	4.7	2.2	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
93.0	4.7	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
93.2	4.7	2.1	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
93.3	4.7	2.1	1.5	1.3	1.1	1.0	0.9	0.8	0.8	0.7	0.6	0.6
93.5	4.7	2.1	1.5	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.6	0.6
93.9	4.8	2.0	1.5	1.2	1.1	1.0	0.9	0.8	0.8	0.7	0.6	0.5
94.3	4.6	2.0	1.5	1.2	1.1	1.0	0.9	0.8	0.8	0.7	0.6	0.5
94.7	4.6	2.0	1.5	1.2	1.1	0.9	0.9	0.8	0.7	0.7	0.6	0.5
95.2	4.5	1.9	1.4	1.2	1.0	0.9	0.8	0.8	0.7	0.7	0.6	0.5
95.6	4.4	1.9	1.4	1.2	1.0	0.9	0.8	0.8	0.7	0.7	0.6	0.5
96.0	4.4	1.9	1.4	1.1	1.0	0.9	0.8	0.8	0.7	0.6	0.5	0.5
96.4	4.3	1.8	1.3	1.1	1.0	0.9	0.8	0.7	0.7	0.6	0.5	0.5
96.9	4.1	1.8	1.3	1.1	0.9	0.8	0.8	0.7	0.7	0.6	0.5	0.5
97.3	4.0	1.8	1.3	1.0	0.9	0.8	0.8	0.7	0.7	0.6	0.5	0.5
97.7	3.9	1.7	1.3	1.1	0.9	0.8	0.8	0.7	0.7	0.6	0.5	0.5
98.0	3.8	1.7	1.3	1.1	0.9	0.8	0.8	0.7	0.7	0.6	0.5	0.5
Avg.	4.6	2.0	1.5	1.2	1.1	1.0	0.9	0.8	0.8	0.7	0.6	0.5

Table 9.3 Width of 90% confidence interval of error in estimate of risk in payment for the quality characteristic for different sample sizes

Mean Q/C	Sample Size											
	2	8	14	20	26	32	38	44	50	60	80	100
90.0	15.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
90.4	17.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
90.8	22.2	20.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
91.3	25.3	22.7	22.6	22.0	19.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
91.7	26.0	23.5	23.7	24.0	24.0	24.1	24.1	24.1	24.1	24.1	24.0	23.8
92.1	26.6	23.6	23.9	24.2	24.3	24.3	24.4	24.4	24.5	24.5	24.5	24.6
92.2	26.9	23.7	24.0	24.1	24.3	24.3	24.4	24.4	24.4	24.5	24.5	24.5
92.2	27.2	23.7	24.0	24.1	24.3	24.3	24.3	24.4	24.4	24.5	24.5	24.5
92.3	26.8	23.6	23.9	24.1	24.2	24.3	24.2	24.3	24.3	24.3	24.4	24.4
92.4	26.5	23.6	23.8	24.0	24.1	24.1	24.2	24.1	24.2	24.1	24.1	24.0
92.5	27.0	23.4	23.7	23.8	23.9	23.8	23.8	23.8	23.8	23.7	11.5	9.9
92.6	26.8	23.5	23.6	23.6	23.5	23.5	23.4	17.7	14.7	13.0	10.9	9.6
92.7	26.9	23.1	23.4	23.4	23.2	18.7	15.5	14.1	13.3	12.3	10.5	9.6
92.9	26.7	23.0	23.1	22.8	18.0	15.7	14.4	13.5	12.7	11.6	10.4	9.6
93.0	26.8	22.8	22.5	19.1	16.4	15.0	13.7	12.9	12.2	11.2	9.9	8.9
93.2	26.6	22.3	22.3	18.0	15.9	14.4	13.3	12.3	11.6	10.6	9.2	8.2
93.3	26.4	22.5	20.8	17.1	15.1	13.7	12.5	11.7	10.8	9.9	8.5	7.4
93.5	26.6	22.6	19.0	16.0	14.0	12.6	11.3	10.6	9.9	9.2	7.7	6.7
93.9	26.0	21.2	16.7	13.8	11.8	10.5	9.6	8.8	8.1	7.3	6.3	5.6
94.3	24.9	19.8	14.5	11.6	10.2	9.2	8.5	7.8	7.3	6.6	5.7	5.2
94.7	24.8	16.3	12.2	10.2	9.1	8.3	7.6	7.0	6.6	6.1	5.2	4.7
95.2	24.1	13.9	10.8	9.1	8.1	7.3	6.7	6.3	5.9	5.4	4.7	4.2
95.6	22.8	12.9	9.8	8.3	7.5	6.7	6.1	5.6	5.3	4.8	4.2	3.8
96.0	21.9	11.6	8.8	7.4	6.5	5.9	5.4	5.0	4.8	4.4	3.7	3.4
96.4	21.3	10.5	7.9	6.7	5.8	5.3	4.9	4.6	4.3	3.9	3.4	3.0
96.9	19.1	10.1	7.5	6.3	5.5	5.0	4.6	4.3	4.0	3.6	3.1	2.8
97.3	18.3	9.6	7.3	6.1	5.4	4.9	4.4	4.1	3.9	3.5	3.0	2.7
97.7	17.0	9.5	7.3	6.0	5.3	4.8	4.4	4.1	3.8	3.5	3.1	2.7
98.0	16.0	9.3	7.1	6.0	5.3	4.8	4.4	4.0	3.8	3.5	3.0	2.7
Avg.	23.8	17.7	15.7	14.5	13.6	12.2	11.7	11.2	10.8	10.3	9.3	8.8

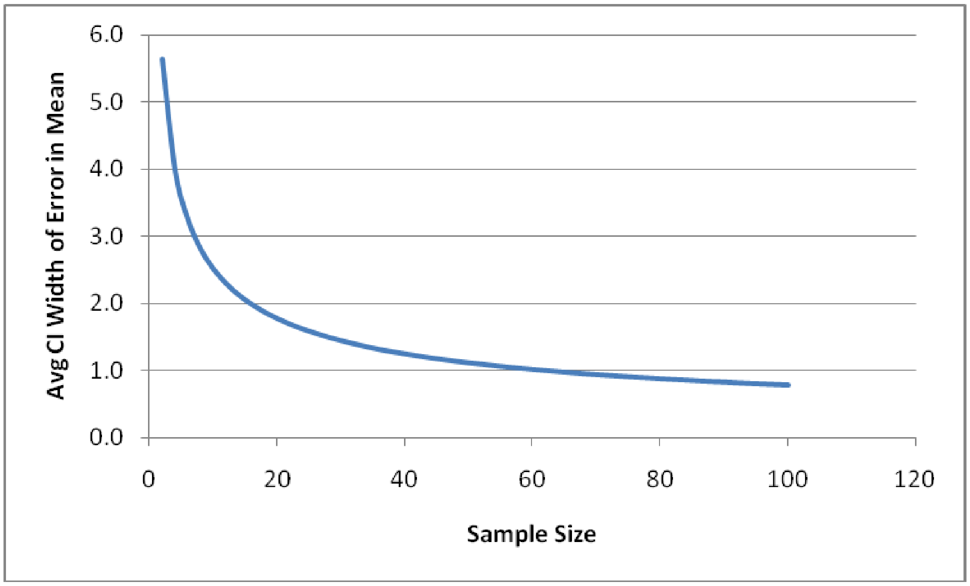


Figure 9.21 Average width of 90% confidence interval of error in estimate of mean

Figure 9.22 shows the maximum error in estimate of standard deviation in 90% of the cases for different sample sizes. Figure 9.23 is probably even more relevant because it shows how risk in payment to be made to the contractor goes down (in 90% of the cases) with increasing sample size for a lot. It is also more relevant because it includes the effects of errors in the estimation of the mean as well as variability. It should be noted that these errors are for each of the lots. A project will have several lots and the errors may cancel each other out, at least partially, when the payment is calculated for the entire project. However, a good quality assurance program should minimize risk in lot pay factors as well.

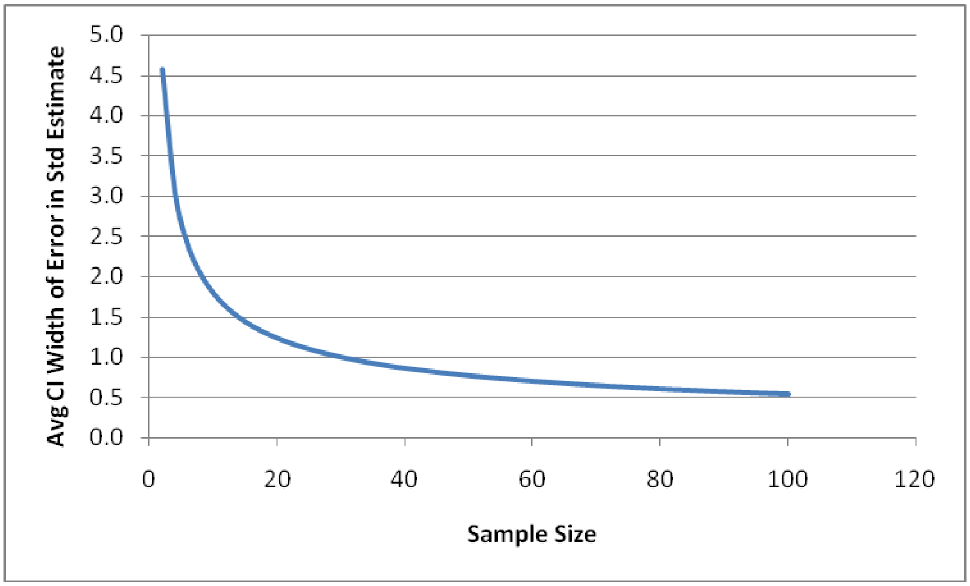


Figure 9.22 Average width of 90% confidence interval of error in estimate of variability

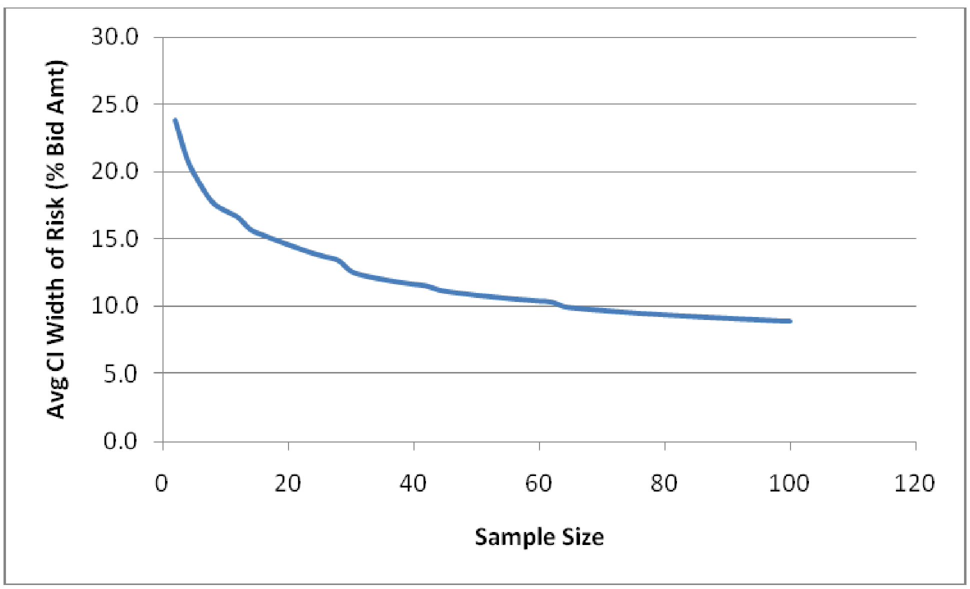


Figure 9.23 Average width of 90% confidence interval of risk

9.5 Conclusion

This chapter proposed a feedback process to design using QA and QC data. This chapter also presented the analysis that was performed using actual field data from projects constructed under MDOT QA program in the year 2008. This analysis helped in understanding the nature and magnitude of variability in MDOT projects and differences in these variabilities among projects depending on the strictness of quality control and other factors. This will help in making realistic assumptions in the design process. It was also established that most of the construction data follow a normal distribution to a large extent. This validated the assumption of normality that was made in developing the risk analysis simulation for MDOT QA program. Another simulation was developed to design optimal feedback process for design. The simulation helped in estimating the errors that can be expected depending on the sample size that is used in the feedback process and the associated probabilities. Finally, plots were developed to relate sample size to probabilistic error in estimation of the mean and standard deviation of the quality characteristic and payment risk. These plots can be used directly by MDOT to decide on the appropriate sample size (i.e., not too small to lead to higher errors and risk and not too large to be too costly or impossible to carry out).

Chapter 10: Conclusions and Recommendations for HMA QA Program

This chapter presents the overall strategy adopted in this research followed by the conclusions and recommendations that have been derived. A good quality assurance program should use the quality characteristics which can ensure pavement performance that meets or exceeds the design target. While there are several components to the QA program, identification of the suitable quality characteristics is the most important one.

10.1 Identification of Suitable Quality Characteristics for QA Program

Identification of suitable quality characteristics requires preparing an exhaustive list of potential characteristics which should be considered for inclusion in the QA program. The different sources for preparing this list are enumerated below.

(1) Quality characteristics being used in other states' QA programs: Different states use varying combinations of quality characteristics. Some of these quality characteristics are used in determining payment to be made to the contractor for any project, while others are used merely to provide feed back for proper construction.
(2) Quality characteristics being used in other states' QC programs: Any of the quality characteristics which are used in QC programs, i.e. is monitored by the contractor, but is not used in the QA program should also be considered.
(3) Quality characteristics used in the Mechanistic-Empirical Design Guide Software: The MEPDG software includes models to predict pavement performance from material, pavement structure, construction, traffic and environmental variables. The models used in the software are the result of studies carried out by many research teams after extensive testing and analysis to relate those variables to performance. Therefore, those variables or quality characteristics which are within the control of the contractor and which can be tested at the time of construction should also be included in the list.
(4) Quality characteristics studied in other research projects which have been shown to have impact on performance.

The second step is to shortlist those candidate QA variables which can be shown to affect pavement performance. These relationships can be established through one or more of the following options.

(1) Empirical data from Michigan: If empirical data can be obtained which establish that changes in levels of one quality characteristic leads to change in pavement performance, either individually or in conjunction with other quality characteristics, then it would be the most preferred way.
(2) Empirical data from other states: If Michigan data is not available or is not good enough to establish relationships mentioned in option 1 then empirical data from other states can be used. This is a slightly more indirect way of establishing whether a certain quality characteristic should be used in the Michigan QA program. This is because any other state may have climate, typical construction materials, construction practices and traffic different from those in Michigan. However, if some of these factors are matching with Michigan and/or if those parameters affect performance very significantly then they must be analyzed using such data.
(3) Analysis using MEPDG: MEPDG software puts together the best available response and performance models for flexible as well as rigid pavements. The models are generally mechanistic-empirical in nature. They have been developed and calibrated using empirical data. Therefore, analysis performed using these models simulates using empirical data but with more flexibility, although it also comes with prediction errors. Nonetheless, it should be recognized that simulation results are indicative rather than predictive. This means that these results should be used to guide decisions in general; but they cannot be used to provide direct predictions for a particular project. Therefore, relative differences in performance because of these quality characteristics are more relevant. MEPDG also allows for studying the effect of variability in these quality characteristics on pavement performance.
(4) Other research studies: Other research studies firmly establishing relationship of the candidate QA variables to performance can also be used to verify the findings from the above three options. In the case of variables for which none of the above options can be feasibly used for analysis this may be the only option.

The third and last step is to identify those variables which should be incorporated into the Michigan DOT QA program. The criteria for including those variables are that:

(1) They affect pavement performance either directly or in conjunction with other variables.
(2) They need to be tested individually and cannot be estimated or calculated from other significant QA variables already being used in the QA program.
(3) It is feasible to test for them within a reasonable amount of time during the construction.
(4) The testing for these candidate QA variables does not require very specialized or costly equipment.

It is possible for the contractor to control these variables through sound construction practices and tight quality control.

10.1.1 Comparison of MDOT QA programs with others in the US

The MDOT HMA QA program is similar to those being used by the states who have adopted end result specifications for their QA program. This similarity exists in (1) the tests that are used for verifying the quality of the constructed pavement, (2) the specifications limits that are used and (3) the quality characteristics, the statistical method and the pay formula used for calculating the payment to be made to the contractor. In other words there is nothing alarmingly different in the QA program being used by MDOT as compared to other "ERS" states.

10.1.2 QA Parameters Indentified by Other Studies

Based on literature review of studies that looked at the effect of pavement design and construction variables on performance, the following variables were identified as key QA parameters:

1. Asphalt Content
2. Air Void Content
3. Mixture Density
4. Aggregate Size and Gradation
5. Pavement Thickness
6. Binder Performance Grade

All the above variables can be input explicitly in the MEPDG software to study theoretical effects on performance.

10.1.3 Summary of Results from Empirical Analysis

The analysis using empirical data from the LTPP database showed that such data generally cannot be used to develop a performance model that explicitly relates specific QA variables to performance. Such analysis can at best show some trends as to how a particular quality characteristic can affect certain aspects of pavement performance.

The set of data extracted from LTPP database in this analysis showed that pavements with low (< 4%) or high (>8%) in-situ air voids have higher probability of rutting than those in which air voids fall in the range of 4 to 8%. This indicates that in-situ air voids immediately after construction is an important test for a quality assurance program. Contrary to intuition the asphalt content data does not show any clear trend in its effect on rutting. This is most likely because of not having data points corresponding to very low or very high asphalt content compared to the corresponding design values. The trends observed in the case of fatigue cracking and longitudinal cracking were not very clear. This may most likely be because of the fact that several other factors are also affecting the fatigue performance and these cannot be controlled in the analysis with such data.

An argument was presented in chapter 6 as to how percent-within-limits for a particular quality characteristic should be related to pavement performance to rationally justify its use in the pay factor equation. The analysis did not show a clear relationship between PWL for in-situ density and fatigue cracking, longitudinal cracking and transverse cracking. Based on the plots showing PWL (plant air voids) versus cracking performance it can be said that while lower PWL does not necessarily mean poor performance, the probability of better performance certainly goes up with higher PWL. Further plots show that as PWL (plant air voids) become higher, i.e. air voids are within the specification limits to a greater extent, there is better longitudinal and transverse cracking performance. No observable relationship between PWL (plant air voids) and rutting was found in this analysis.

The conclusions derived from the empirical data analysis can at best be considered as mixed and not clear. This happens because of several factors discussed in chapter 6. These conclusions, therefore, indicate that a thorough mechanistic-empirical analysis needs to be performed to derive firm conclusions required for assessing quality assurance programs like the one being used by the state of Michigan.

10.1.4 Summary of Results from Mechanistic-Empirical Analysis

Based on the limited number of verifications, MEPDG seems to give reasonable predictions for different pavement distresses. Therefore, MEPDG can be used to explore all the aspects of mix and construction process which influence pavement performance. Pending verification through other methods, the other aspects of mix production or lay down that are not being accounted for in the current QA program can be identified though the strategy presented in this report. The proposed strategy can also be directly used to determine the targets and allowable windows for each of the quality characteristics being used in the QA program.

MEPDG can be used to simulate actual construction and resulting pavement performance. A preliminary example of that has been presented in this report. When extended with possible life cycle cost analysis, such a simulation process can lead to a rational pay factor formula/procedure.

Effect of varying air void distribution on average project performance

The results of MEPDG analyses corresponding to cases of varying in-situ air voids to achieve various PWL ranges, while fixing the asphalt content (at -0.4% of optimum), showed that on average, the maximum difference in fatigue was only about 2% between the worst PWL and the acceptable PWL (greater than 90%); a negligible difference. It was also noticeable that for very similar PWL values, different projects showed different amounts of fatigue cracking. This is because fatigue cracking in each project is estimated by sampling. As long as the range, mean and standard deviation of two samples are the same they would have the same PWL. But the average fatigue cracking can vary depending where each of the samples falls within the specifications. For example, if a project has all densities equal to 93%, PWL would be 100%. If the density of all the samples were 98%, PWL would still be 100%. However, fatigue performance for the 98% density project would be much better than that for the project with mean density of 93%.

Rutting performance estimated through the simulation showed that rutting performance improves as PWL approaches 100%. However, this improvement is very small. In other words rutting performance is not so sensitive to PWL (in-situ air voids) according to MEPDG results. The analysis also showed that IRI does not seem to be much affected by PWL (in-situ air voids).

It must be mentioned that all these results represent average project performance as opposed to a more realistic sublot by sublot performance, and they were only for one type of mix. It is important to study different types of mixes to assess how PWL values affect the performance for each one of them. It is quite possible that this effect may be much more pronounced in other types of mixes.

Effect of varying air void distribution and asphalt content on project sublot performance

The observed effects of PWL on average performance for a given project were not as pronounced as one would expect intuitively. The main reason for this is the attenuation of distress levels because of the averaging of performance across sublots. Therefore, it is more realistic to show results in terms of performance by individual sublots. Results in terms of fatigue performance for various scenarios (three air void and five asphalt content distributions) showed that:

1. Fatigue performance of different sublots within a project can vary significantly;
2. Within a given asphalt content range, there are more underperforming sublots when the air void distribution moves towards the upper specification limit ;
3. As the asphalt content range moves dry-of-optimum, more sublots are underperforming in fatigue, even at relatively high combined PWL.
4. As the asphalt content range moves wet-of-optimum, more sublots perform better in fatigue, even at relatively low combined PWL. This may however lead to more rutting within the HMA layer.

The analysis also showed that cases where the combined percent-within-limits (PWL) values were very similar (combined PWL is generally used directly to calculate payment for the contractor), the average fatigue cracking at 30 years varied from 0.7% to 6.8%. This much of variation is very significant because they would be categorized as being in very good and nearly poor condition, respectively. The 75th percentile value for the same cases varies from 0.8% to 8.7% and the 95th percentile varies from 1.1% to 9.9%. This clearly shows that the candidate QA variable causing this variation, asphalt content in this case, has significant influence on fatigue performance.

Variations in fatigue performance as a result of change in in-situ air voids in these cases are also appreciable, although they are not as large as those because of asphalt content. When the air voids was increased from 3.5% to 8%, the 75th percentile for fatigue cracking increased from 13.9% to 17.9%. An advantage of percentile distresses is that it can possibly identify those cases where a few sublots perform very poorly while the rest of the sublots may have acceptable performance. Such a scenario should not be acceptable because the distressed sublots may force early repair work for the entire project.

For rutting performance, the results showed that the range of rutting values corresponding to the average, 75th and 90th percentiles for all the cases was rather small. It should be noted that in these runs the asphalt content was allowed to vary only up to 0.5% more than the optimal. If higher asphalt contents were allowed the increase in rutting would have been probably more significant. However, the selected criteria, namely the average rutting at 30 years, the 75th and 90th percentile rut values are still good indicators of the effect of QA variables on performance.

The results also showed that even though different projects may have very similar combined PWL values, they may perform differently in rutting. However, this does not necessarily mean that PWL methodology does not work. Different factors may have relatively different influence on rutting performance and they may affect each others' effect as well. It is the goal for a good QA program to factor this in, to determine how combined PWL is calculated for the entire project.

Effect of varying HMA thickness on project performance by sublot

Based on the analysis and results for the effect of AC thickness variation on project performance, the following conclusions were drawn:

1. The performance of different sublots within a project can vary significantly.
2. HMA mat thickness variation has a significant influence on fatigue performance. The mean fatigue ranges from 25.3% to 40.6% for HMA mat thickness ranging from 7.1 in. to 8.1 in., respectively. Thus, the loss in fatigue performance can be significant with as little as 0.5-in. reduction in HMA mat thickness.
3. There seems to be almost 10% increase in fatigue distress for every 0.5-in. decrease in HMA mat thickness from 8.1 in. to 7.1 in.
4. The mean IRI ranges from 143 in./mile to 158 in./mile for thicknesses ranging from 8.0 in. to 7.1 in., respectively. It should be noted that in MEPDG, the IRI values are influenced by rutting and fatigue distresses.

MEPDG simulations provided valuable information to help us understand how performance varies within a project within such narrow ranges and variations in inputs, which is not possible to gain any other way.

10.2 Summary of Results from ERS Risk Analysis

A Monte-Carlo simulation was developed to assess the current QA program of MDOT. The analysis of the simulation showed that production variability, measurement variability, sample size and bias have a significant influence on the payment risk to be made to the contractor. This knowledge leads to identification of ways to reduce payment risk. The simulation can be used to analyze all other variables of a QA program and thereby improve it to achieve lower risk of overpayment or underpayment. The analysis also showed that, even though measurement variability is very low, high production variability and mean production being in the middle of the specification window still creates risk. Accordingly, the contractor should (1) produce right around the target, and (2) maintain low variability in production quality. This is significant from the point of view of pavement performance.

Generally the test methods and instruments are standardized and calibrated in the beginning of the construction project. For longer projects the instruments may develop bias with continued use over several days. Bias has a very significant effect on payment risk. Such situations can lead to disputes and even law suits. Therefore, bias must be avoided through suitable inspection of the test equipments.

10.3 Feedback Process for Design

A feedback process to design using QA and QC data was proposed. In addition, an analysis was performed to (1) to understand the nature and level of variability in MDOT projects, and (2) to isolate the effect of strict quality control and other factors on the variability among projects. This will help in making realistic assumptions in the design process. The analysis was performed using actual field data from projects constructed under MDOT QA program in 2008. It was also established that most of the construction data follow normal distribution to a large extent. Consequently, the assumption of normality that was made in developing the risk analysis simulation for MDOT QA program was validated.

Another simulation was developed to design optimal feedback process for design. The simulation helped in estimating the errors that can be caused by the sample size in the feedback process and the associated probabilities. Finally plots were developed to relate sample size to the probabilistic error in estimating the mean and standard deviation of the quality characteristic and payment risk. These plots can be used directly by MDOT to decide about the appropriate sample size (i.e., not too small to lead to higher errors and risk and not too large to be too costly or impossible to carry out).

10.4 Use of Non-destructive Tests in QA Program

A detailed review of non-destructive tests is provided in Appendix B. The following are some of the relevant tests for use in QA programs of flexible pavements:

- Ground Penetration Radar (GPR) testing for thickness measurement
- Falling Weight Deflectometer (FWD) testing for modulus estimation
- Dynamic Cone Penetrometer (DCP) and/or lightweight FWD for modulus measurement of unbound layers
- Infra-red thermal imaging of HMA mats for checking uniformity and detection of temperature segregation

10.5 Overall Conclusion and Recommendations

Based on the review of Michigan and other DOT QA programs, it is concluded that the MDOT QA program is on par with ERS based QA programs used by the majority of the states. In other words there is nothing alarmingly different in the QA program being used by MDOT as compared to other "ERS" states. The results presented in this report confirmed the importance of HMA asphalt content, air void content and HMA thickness on long-term performance. It is recommended that HMA thickness be considered as an additional candidate QA parameter and that the interactions between QA characteristics (e.g., air voids and asphalt content) be incorporated in the pay formulae.

Because empirical analyses linking key characteristics to long-term performance were inconclusive (not enough data from the MDOT construction database and inconclusive results from the LTPP database), it is recommended that the mechanistic-empirical approach be adopted for this purpose. With the future possible adoption of the MEPDG by MDOT and other DOT's, it is suggested that the MEPDG be adopted for this purpose. The analyses conducted as part of this research study can serve as examples for such future efforts. The advantage of mechanistic-empirical approach is its ability to quantify the relative effects of deviations from the target on long-term performance and to include interactive effects between different QA characteristics. This allows for modifying/refining the pay formulae based on rational arguments.

Therefore, potential improvements to the QA program should focus on fine tuning the specification limits used and refining the pay formulae to minimize the risk associated with construction variability. In addition, accounting for interactions between certain QA construction quality characteristics (e.g., asphalt content and air voids) in these formulae should help in preventing extreme combinations that have drastic negative effects on pavement performance. Ideally, these refinements should be made based on mechanistic analyses. The pay formulae should be based on the final pavement performance such as pavement life, which can be predicted based on the MEPDG.

The QA data and the pavement surface distress data obtained by MDOT's PMS are the two most relevant data for evaluating the effectiveness of the current QA processes. Unfortunately, MDOT's QA data are either incomplete or missing. A good database system for storing QA data should therefore be developed.

The complexity of the QA processes increases as the number of characteristics is increased. If we rely on probability/statistics methods to investigate the impacts of acceptance sampling rules on the risks of accepting poor quality level of products and rejecting good quality level of products, it may suggest that there is a need to investigate how to reduce the number of characteristics for QA processes without affecting product quality level. However, if we use simulations based on mechanistic modeling, we can account for multiple QA characteristics and their interactions without the need for complex analyses.

Finally it is recommended that the QA data be used as part of the feedback process for design, as described in chapter 9 of this report. Results from probabilistic analyses like those described in chapter 9 can be used for the selection of optimal sample size for QA testing in order to minimize the error in estimating the mean and standard deviation of the quality characteristic and estimated pavement life.

The use of non-destructive testing to quantify as-constructed material properties should be a systematic part of the QA program. For AC pavements, GPR and FWD testing should be conducted as they offer complementary information on the pavement structure and material properties/parameters. In addition, DCP and/or lightweight FWD testing for unbound materials should be conducted in reconstruction projects or in areas where pavement coring is done. Thermal imaging of the HMA mat should also be made part of the QA program in addition to the current density measurement requirements.

Part II : Rigid Pavements

CHAPTER 11: MDOT Quality Assurance Data for Rigid Pavements

Certain projects were identified to get a sample of construction and materials test data collected historically on highway construction projects. Although this project was meant to evaluate quality *assurance* procedure, data from quality *control* procedures were also collected as part of the data collection effort. The rationale behind this decision is that different quality assurance procedures were followed at different times in the history of highway construction in Michigan. However, if suitable data were collected under the quality control procedures, they could be used with the current quality assurance procedure, or a slightly modified version of it (like a shadow specification), to assess how the construction would be rated according to those specifications. This can then be related to the actual performance of those pavements. In other words, any material test data, whether under QC or QA, can be evaluated for their possible relationship to pavement performance. If any of such test results show strong correlation to performance, then they can be considered for quality assurance.

11.1 Selection of Sample Projects

Two sources of data were identified:

(1) A list of projects for which performance data had already been processed by MDOT was used to identify the first set of sample projects for data collection.

(2) Actual construction documents from those projects stored in boxes in the record center of Michigan Department of Transportation and microfilms stored in C & T office of MDOT, Lansing.

Table 11.1 lists the projects which were selected as sample projects. All the data available at MDOT related to these projects were collected. This was done in order to understand the types of data that would be available for analysis in these projects.

Table 11. 1 Sample rigid pavement projects selected for data mining.

REGION	ROUTE	CS	JN	BMP	EMP	LET	LOCATION
University	I-96 EB	47065	28215	5.671	9.200	6/7/1996	Chilson Road to Dorr Road
University	I-96 WB	47065	28215	5.623	9.223	6/7/1996	Chilson Road to Dorr Road
Southwest	I-94 EB	11017	32516	0.905	5.603	5/12/1995	E of I-196/US-31 Interchange to W of M-140
Southwest	I-94 WB	11017	32516	0.888	5.886	5/12/1995	E of I-196/US-31 Interchange to W of M-140
Metro	WB/I-27:	63191	36003	0.000	2.230	5/12/1995	Ramp from WB I-96/NB I-275 to WB I-96
		61131	3036				

Key: CS: Control Section
JN: Job Number
BMP: Beginning Mile Point
EMP: Ending Mile Point

11.2 Sample Data Gathering

All the boxes and microfilms (if available) were searched for data related to the sample projects listed in Table 11.1. Table 11.2 summarizes the number of concrete pavement projects with test data available. Appendix A gives a more detailed list of the types of data available.

Table 11. 2 Summary of test results collected from sampled concrete pavement projects.

Material Property	Cores	Stockpile	Job Site
Compressive Strength	2		
Gradation		2	2

It can be seen that, for concrete pavements, there is very little test data available in the project construction files. There is a possibility that this data had been collected, but was either misplaced or is being used by some third party. Other quality characteristics used in contemporary QA programs are also very scarce.

Considering the fact that there is scarcity of data required for determining the influence of various quality characteristics on pavement performance, it was deemed necessary to consider other alternatives rather than relying on Michigan data. The alternatives were to collect relevant data from the Long Term Pavement Performance (LTPP) database and to conduct mechanistic-empirical analysis using MEPDG and other performance models.

CHAPTER 12: Review of MDOT QA Program and Design Process for PCC Pavements

12.1 MDOT PCC Pavement Quality Assurance Program

Portland cement concrete work is investigated during all phases of production through placement for adequate and acceptable quality. Sampling, testing, and inspection should meet or exceed the minimum rates specified by the Michigan Department of Transportation. The following are the current concrete Quality Assurance (QA) sampling and testing requirements:

- Concrete Temperature
- Concrete Slump
- Concrete Air Content
- Concrete Yield
- Concrete Strength
- Concrete Pavement Thickness

Random sampling and testing methods, which assure all material being produced have an equal chance of being selected for testing, is specified by the engineer. The materials quality assurance manual describes such acceptable methods.

- **Concrete Temperature**: It is required that, at the time of placement, concrete temperatures be between 45° F and 90° F. Steam or hot water coils, live steam, and indirect hot air is usable for heating concrete ingredients, e.g. water and/or aggregates, if necessary, to meet the minimum placement temperature. Also, if the mean daily air temperature is expected to remain below 45° F during the curing period, the engineer may allow or require the use of concrete accelerators, e.g. additional cement or an admixture, to accelerate the rate of strength gain.

- **Concrete Slump**: Slump reading should not exceed 3.0 inches or the contractor's approved mix design. Samples for slump measurements are taken and tested in accordance with ASTM C 143, Standard Test Method for Slump of Hydraulic-Cement Concrete.

- **Concrete Air Content**: At the time of placement, existence of 6.50 ± 1.50 percent entrained air is required. Samples for determination of air content for freshly mixed concrete are taken in accordance with MTM 207 and tested by ASTM C 231, Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method, or by ASTM C 173, Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method.

If freshly mixed concrete contains slag or other highly absorptive coarse aggregate, air content of the mixture is determined according to ASTM C 173, Standard Test Method for air content of freshly mixed concrete by the volumetric method.

- **Concrete Yield**: As concreting operation starts and specified slump and air content is attained, unit weight of each mix design should be immediately determined. Average unit weight from three different batches is used for determination of actual mixture yield. That is, in the progress of concreting, if yield based of a single unit weight determination differs from theoretical value (adjusted for differences in air content) more than ±2 percent, two additional unit weight determination needs to be made.

- **Concrete Strength**: At least once every 200 cubic yards for a specific mix design of one day's production, concrete strength against flexure and compression should be determined for two cylinder or beam samples. Specified procedures for concrete strength sampling, curing, and testing are ASTM C 31, ASTM C 39, ASTM C 78, and ASTM C 293, respectively.

- **Concrete Pavement Thicknes**s: Final acceptance of concrete pavements is based on thickness and, if required, depth of reinforcement below the concrete pavement surface. In accordance with MTM 201, cores are taken and determination of pavement thickness is conducted. If thickness or location of steel reinforcement exceeds acceptable tolerances, samples are classified by their deficiency and the contract unit price will be adjusted accordingly. These adjustments will be applied cumulatively to the pavement unit being evaluated.

Figure 12.1 shows the flowchart for acceptance testing procedure for PCC pavements, with corresponding tables 12.1 through 12.3. Table 12.4 show the sampling rates for the concrete QA program.

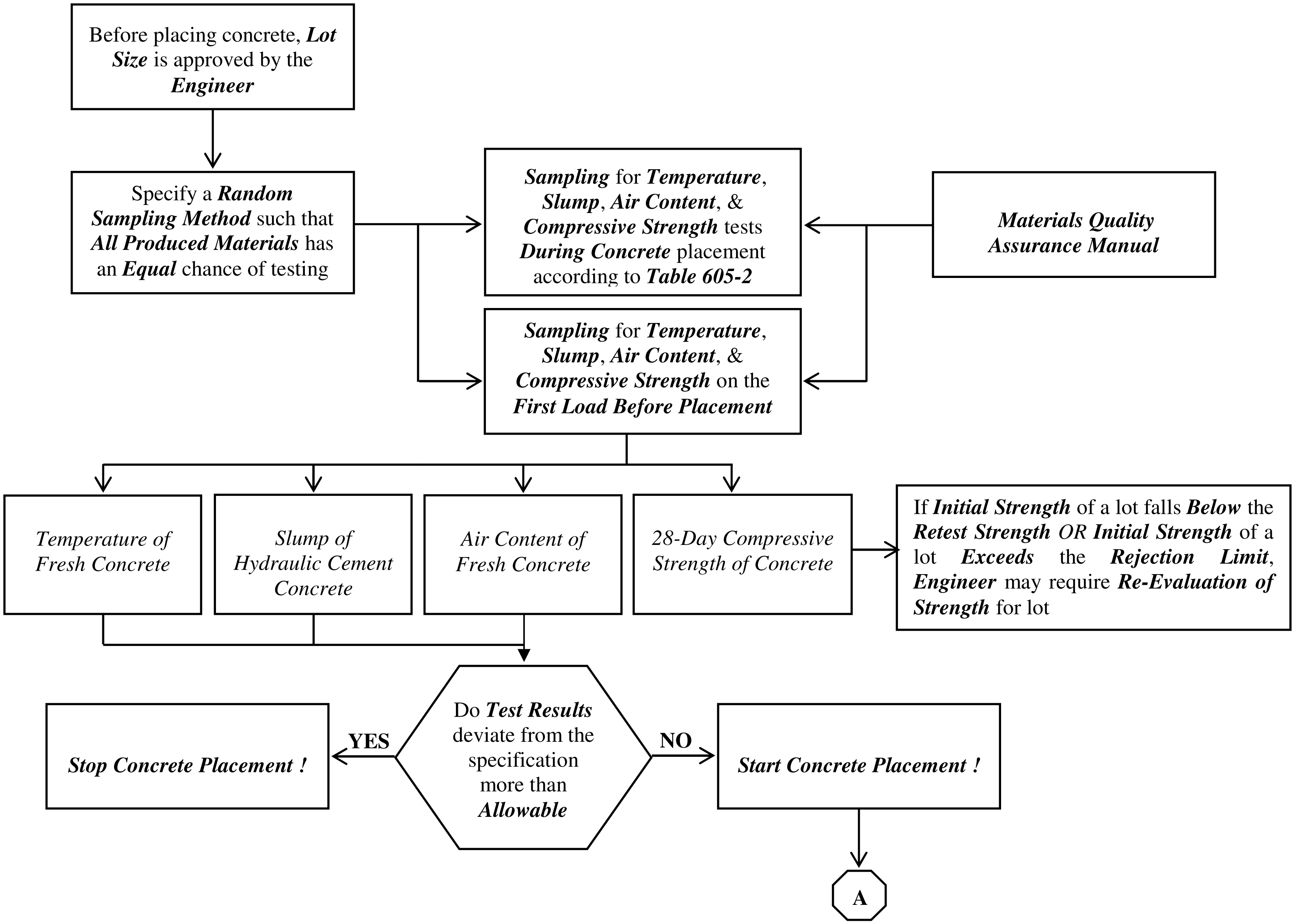


Figure 12.1a Flowchart for Acceptance Testing Procedure for PCC Pavements

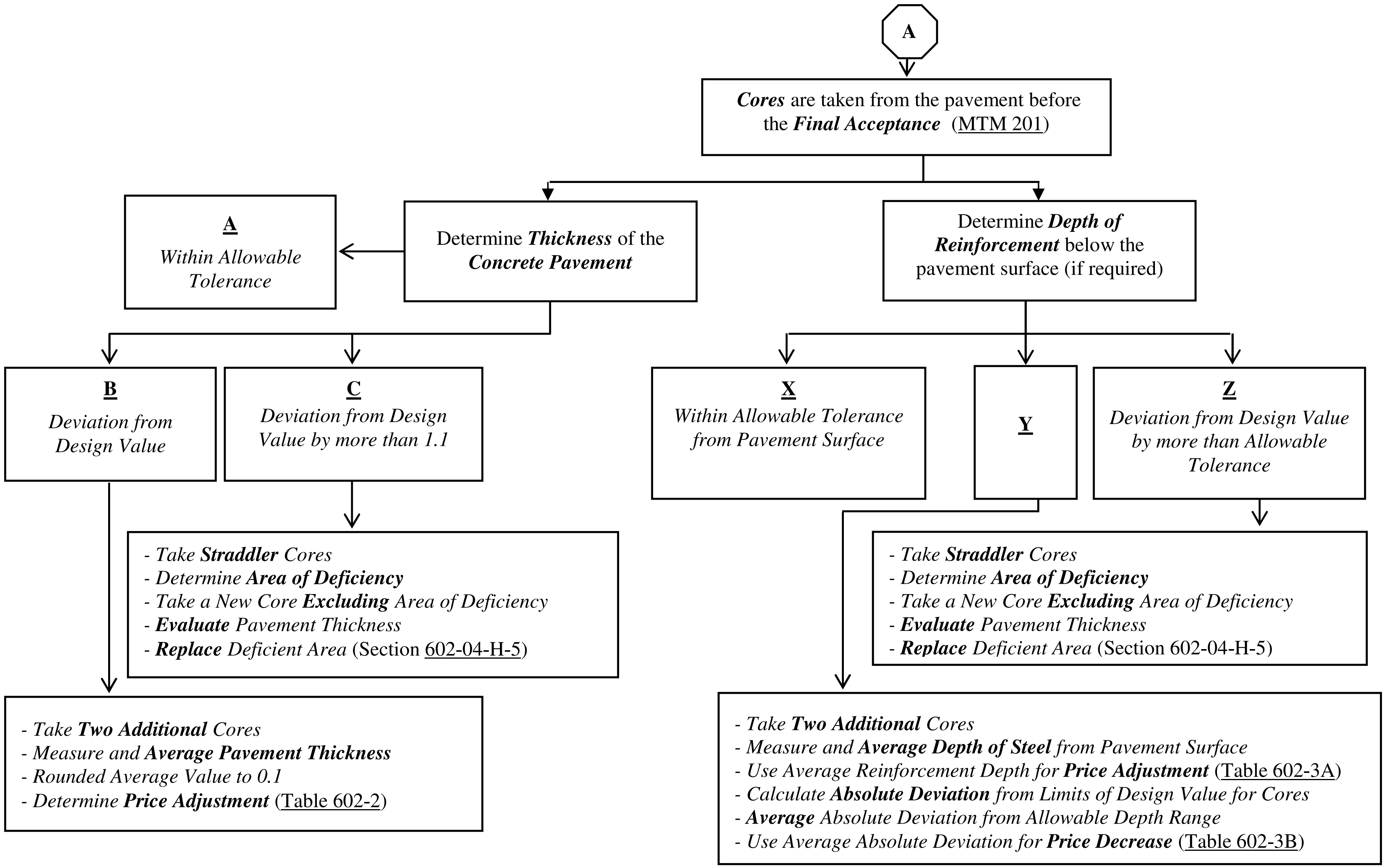


Figure 12.2b Flowchart for Acceptance Testing Procedure for PCC Pavements

Table 12.1 Price Adjustment for Concrete Thickness Deficiency (MDOT Table 602-2)

Initial Core Type	Deficiency in Thickness (Inch)	Price Adjustment (Percent)
A	0.20 or Less	0
B	0.30	-5.0
B	0.40	-15.0
B	0.50	-25.0
B	0.60 To 1.0	-50.0
C	1.10 and Over	-100 [a]

a. *Corrective Action* up to and including *Remove* and *Replace* pavement

Table 12.2 Price Adjustment for Depth of Steel from Pavement Surface (MDOT Table 602-3A)

Initial Core Type	Tolerance on Depth of Reinforcement (Inch) [a, e] For Uniform Plan Thickness (Inch) [c]					Price Adjustment (Percent)
	7.75 - 8.50	8.75 - 9.50	9.75 - 10.50	10.75 - 11.50	Shoulder	
Z	0.0 – 0.9	0.0 – 0.9	0.0 – 0.9	0.0 – 0.9	0.0 – 0.9	-100 [d]
Y	1.0 – 1.9	1.0 – 1.9	1.0 – 1.9	1.0 – 2.4	1.0 – 2.4	-25 [d]
X [b]	2.0 – 4.0	2.0 – 4.5	2.0 – 5.0	2.5 – 5.5	2.0 – 4.0	0
Y [b]	4.1 – 4.8	4.6 – 5.4	5.1 – 6.0	5.6 – 6.6	4.1 – 5.0	-25
Y [b]	4.9 – 6.4	5.5 – 7.2	6.1 – 8.0	6.7 – 8.8	5.1 & Over	-50
Z [b]	6.5 & Over	7.3 & Over	8.1 & Over	8.9 & Over	-	-100 [d]

a. When a pavement is specified to reinforce with two layers of reinforcement, only the top layer of steel will be measured for the proper depth.
b. When a core length measures 0.20 inches or more over the plan thickness, the maximum depth range will be increased by one-half the excess core length over the plan thickness. For each core, the increase will be rounded off to the nearest tenth of an inch according to AASHTO R 11 and then added to the range shown.
c. Pavement base course
d. Corrective action up to and including remove and replace pavement
e. Use the same depth range used for pavement thickness that the shoulder is tied to. Use average shoulder thickness, if tapered.

Table 12.3 Adjustment for Deviation of Depth of Steel from Design Range ((MDOT Table 602-3B)

Initial Core Type	Allowable Average Absolute Deviation from Design Depth of For Uniform Plan Thickness (Inch) [c]					Price Adjustment (Percent)
	6.50 - 7.50	7.75 - 8.50	8.75 - 9.50	9.75 - 10.50	11.0 - 13.0	
X [d]	0.0 – 0.5	0.0 – 0.5	0.0 – 0.5	0.0 – 0.5	0.0 – 0.5	o
Y [d]	0.5 – 1.0	0.5 – 1.0	0.5 – 1.0	0.5 – 1.0	0.5 – 1.0	-10
Y [d]	1.0 & Over	1.0 & Over	1.0 & Over	1.0 & Over	1.0 & Over	-25
Design Range	2.0 – 4.0	2.0 – 4.0	2.0 – 4.0	2.0 – 4.0	2.0 – 4.0	

a. When a pavement is specified to be reinforced with two layers of reinforcement, only the top layer of steel will be measured for the proper depth.
b. Use same depth range used for pavement thickness that the shoulder is tied to. Use average shoulder thickness, it tapered.
c. Pavement or base course.
d. When a core length measures 0.20 inches or more over the plan thickness, the maximum depth range will be increased by one-half of the excess core length over the plan thickness. For each core, the increase will be rounded off to the nearest tenth of an inch according to AASHTO R 11 and then added to the range shown.

Table 12.4 Sampling Rates, Re-Sampling Rates, & Rejection Limits for Concrete QA

	Grade of Concrete				
	D	S1	T	S2 / P1	S3 / P2
Critical Concrete QA Items					
Initial Sampling Rate (per lot)					
Lot Size 0 – 100 Cubic Yards	3	3	3	3	3
Lot Size Over 100 Cubic Yards	6	5	4	5	4
Retest Strength (PSI)					
Lot Size 0 – 100 Cubic Yards	4,500	4,000	3,500	3,500	3,000
Lot Size Over 100 Cubic Yards	4,000	3,500	3,000	3,000	2,500
Non-Critical Concrete QA Items					
Initial Sampling Rate (per lot)	3	3	3	3	3
Retest Strength (PSI)	4,500	4,000	3,500	3,000	2,500
All Concrete QA Items					
Rejection Limit (Percent)	10	10	10	10	10
Re-Sampling Rate (per lot)	6	6	6	6	6

12.2 MDOT PCC Pavement Design Process

An effective pavement design is highly dependent upon performing an adequate investigation of the existing pavement structure. Therefore, prior to construction/reconstruction of a pavement some investigations such as:

- reviewing as-built plans
- reviewing and analyzing existing pavement distress condition
- determining causes of pavement surface distresses
- evaluating pavement ride quality
- reviewing pavement remaining service life
- evaluating drainage system
- evaluating subgrade

should be conducted. A comprehensive investigation of the pavement structure not only ensures that the Engineer employs the proper reconstruction or rehabilitation strategies, but also aids the Designer in the selection of appropriate input values for pavement design.

MDOT uses the pavement design methodology recommended by the American Association of State Highway and Transportation Officials, AASHTO:

- 1993 AASHTO Guide for Design of Pavement Structures
- AASHTO Pavement Design Software DARWin

The following summarizes typical values recommended for design inputs :

- New / Reconstruction Design Life …………..…… 20
- Accumulated ESAL's Years ………………………. 20
- Initial Serviceability ………………………………. 4.5
- Terminal Serviceability …………………………… 2.5
- Reliability Level …………………………………... 0.95
- Overall Standard Deviation ……………………….. 0.39

- 28-Day Mean PCC Modulus of Rupture …………. 670 psi
- 28-Day Mean Elastic Modulus of Slab …………... 4,200,000 psi
- Load Transfer Coefficient (J) …………………….. 2.7 (Tied Shoulder/Widened Lane)
  ……………………... 3.2 (Untied Shoulders)

- Mean Effective k-Value: 50 – 200 psi/in (Typical Range)

Use AASHTO's chart for estimating composite modulus of subgrade reaction and for the correction of effective modulus of subgrade reaction for potential loss of subbase support (Figures 3.3 and 3.6 in AASHTO's 1993 Guide for Design of Pavement Structures).

- Effective Existing Pavement Thickness (Condition Survey Method): Pavement management condition data are used as an aid but a site review of the existing pavement and the planned amount of joint work to be done prior to the concrete overlay must be obtained.

- Overall Drainage Coefficient ………………………1 to 1.05

CHAPTER 13: Survey of QA Programs for Rigid Pavements in the United States

Performance-related specifications for PCCP are not used by as many agencies as for the flexible pavement; however their use is increasing more rapidly than the HMA pavements. The vast majority of the states use the same tests for QA as for QC.

Table 13.1 gives the number of agencies using particular attributes for concrete QC and QA. Forty agencies had responded to the survey from which this data has been extracted (Hughes, 2005).

Table 13.1 Most commonly used quality characteristics for QC and QA on PCC pavements (Hughes, 2005)

Quality Characteristic	No. of Agencies	
	QC	QA
Air content	25	38
Thickness	14	36
Slump	24	33
Cylinder strength	18	31
Gradation	25	26
Beam strength	14	18
Water-cement ratio	12	16
Ride quality	1	15
Aggregate fractured faces	7	6
Sand equivalence	0	3
Permeability	0	3
Core strength	0	2

Table 13.1 and figure 13.1 show the attributes that are most often used for acceptance of PCC pavements: Air content, used by 38 agencies; thickness used by 36; and slump used by 33. Thirty one agencies accept PCC structures based on cylinder strength, and 26 accept gradation. Michigan also uses the first four of these quality characteristics in its QA program. The lesser-used acceptance attributes are aggregate fractured faces, sand equivalence, permeability and core strength.

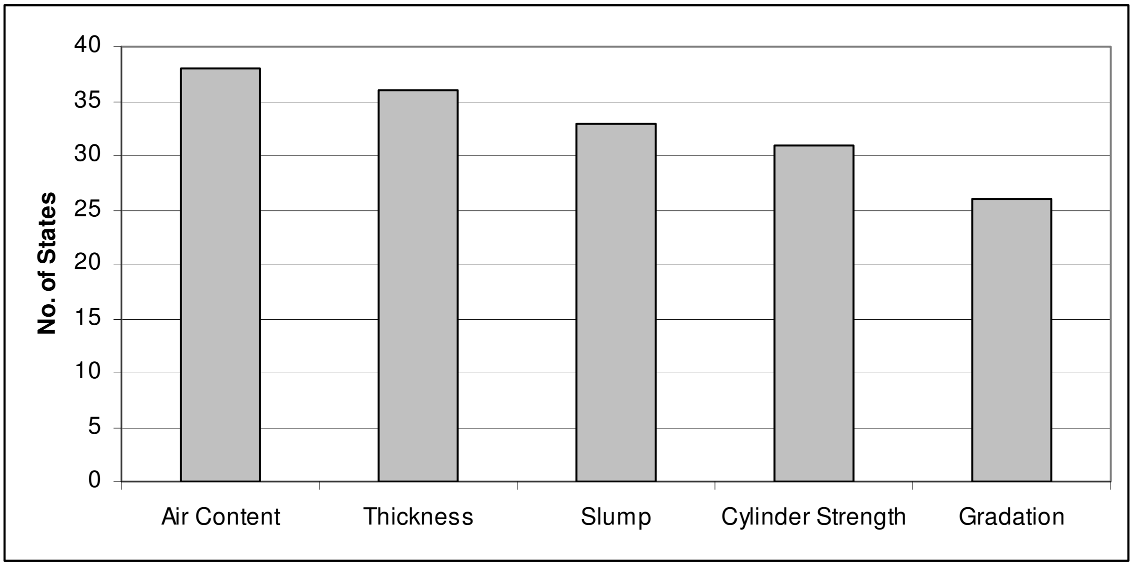


Figure 13.1 Attributes most often used for QA of PCC (Hughes, 2005)

13.1 Regional Level Analysis of the QA Practices

We have listed the QA practices in various states on the basis of their regional location in US.

The quality assurance parameters and the corresponding criteria for rigid pavements in southern states are given in table 13.2 and compared to those used in Michigan. We can observe that there is lot of variability in the criterion followed across the states. For instance, the sample and testing procedure for compressive strength is AASHTO T 22 in Arkansas, but in Georgia, GDT-22 is used for the same quality characteristic. In Texas, the tolerance for deviation from the QA standards for compressive strength is 20% of the mean while Arkansas requires a minimum compressive strength of 4000 psi.

The QA program for rigid pavements in Michigan is done on characteristics such as concrete temperature, slump, air content, yield, strength and pavements thickness. It is required that at the time of placement, the concrete temperature should be in the interval 45° F to 90° F. The concrete slump metric should not exceed 3.0 inches. The sample for slump measurements are taken in accordance with "Department Methods" and tested by ASTM C 143.

At the time of placement, 6.50 ± 1.50 percent entrained air is required. Samples for determination of air content for freshly mixed concrete are taken in accordance with MTM 207 and tested by ASTM C 231 or C173. The final acceptance of concrete pavement is based on thickness. In accordance with MTM 201 cores are taken and determination of pavement thickness is conducted.

Other QA procedures for concrete pavements are summarized in Table 13.3 (Western states) and 13.4 (other states).

Table 13.2 Quality Assurance Practices in Southern States versus Michigan

	Arkansas	Georgia	Texas	Michigan
Sample & Test				
Gradation	AASHTO T 27			
Compressive strength	AASHTO T 22	GDT-22	Tex-448-A Tex-418-A	ASTM C 31
Air content	AASHTO T 152	GDT-26	Tex-414-A	MTM 207, ASTM C 231
Slump	AASHTO T 119	GDT-27	Tex-415-A	ASTM C 143
Sample location				
Compressive strength		Paver		
Air content		During Pouring Operation at Roadway		
Slump		During Pouring Operation at Roadway		
Frequency				
Gradation	One test per 750 Cubic meters of mix	Two per One Day of Production		
Compressive strength	Test specimens for compressive strength determined by cores will be obtained according to AASHTO T 24	Two per One Day of Production		3 to 6 per lot
Air content	Sampled after placement on grade, but before consolidation by paver or vibrators	Two per One Day of Production		3 to 6 per lot
Slump	Sampled after placement on grade, but before consolidation by paver or vibrators	Two per One Day of Production		3 to 6 per lot
Tolerance				
Gradation	±7.0			
Compressive strength	4000 psi (min)		20% of the mean	2,500 to 4,500 for diff grades
Air content	4.0%-8.0%		± 1%	6.50 ± 1.50
Slump			± 1%	± 3 inches

Table 13.3 Quality Assurance Practices in Western States versus Michigan

	Colorado	Utah	Michigan
Sample & Test			
Thickness	CP 68	AASHTO T 24	ASTM C 31
Compressive Strength		AASHTO T 22, 24	MTM 207, ASTM C 231
Air Content	T 141, 152		
Sample Location			
Thickness	Finished Concrete	Finished Pavement	
Compressive Strength		Grade	
Air Content	Mixer Discharge		
Frequency			
Thickness	In accordance with Subsection 412.21.	1 per 12,000 Square Feet	
Compressive Strength		Smaller of One Strength Test Each 700 Square Yards or one Day's Placement	
Air Content	1 per 2500 sq. yards. Or one per day if less than 2500 sq. yards. are placed in a day tests.		
Tolerance			
Thickness		0 - ⅛ ⅛ - ¼ ¼ - ½ ½ - ¾ >¾	
Compressive Strength		(Below 4,000 psi) 1 - 100 101 - 200 201 - 300 301 - 400	

Table 13.4 QA Practices in Some of the States for PCC Pavements

(a) Alabama

Quality Characteristic	Sampling Method	Sample Size	Sampling Location	Testing Frequency
Compressive strength	AASHTO T 23 & AASHTO T 141	Set of 2 Cylinders	ALDOT 210	1 per Pavement Testing unit
Air Entraining	AASHTO T 141	Minimum of one	ALDOT 210	Minimum 1 per Pavement Testing unit
Slump	AASHTO T 141	Minimum of one	ALDOT 210	Minimum 1 per Pavement Testing unit
Thickness	AASHTO T 24	One core	ALDOT 210	1 per Pavement Testing unit

(b) Indiana

Quality Characteristic	Sampling Method	Test Standard	Frequency	Tolerance
Air Content	ITM 802	AASHTO T 152 or ASTM C 173	One per sub lot	– 0.8% to +2.4% of the 6.5% Target
Flexural Strength		AASHTO T 97	Two beams per sub lot	
Thickness of PCCP		ITM 404	Two per sublot	
Unit Weight and Relative Yield		AASHTO T 121	One per sublot	± 3.0%
Water-Cement Ratio		ITM 403	Once per week	± 0.030

(c) Maine

Quality Characteristic	Sample & Test Standard	Sample Location	Tolerance
Gradation	AASHTO T30	Paver Hopper	
PGAB Content	AASHTO T308	Paver Hopper	Target +/-0.4%
%TMD (Surface)	AASHTO T269	Mat behind all Rollers	
%TMD (Base or Binder)	AASHTO T269	Mat behind all Rollers	
Air Voids at N_d	AASHTO T 312	Paver Hopper	4.0% +/-1.5%
%VMA at N_d	AASHTO T 312	Paver Hopper	
Fines to Effective Binder	AASHTO T 312	Paver Hopper	0.6 to 1.2
%VFB	AASHTO T 312	Paver Hopper	

CHAPTER 14: Literature Review – Rigid Pavements

Several research projects were reviewed to verify the significance of candidate QA variables identified in this project for pavement performance. These variables are listed in the following sections.

14.1 Slab Thickness

PCC slab thickness is obviously the basic parameter in the design of rigid pavements. In support of such a statement, it has been experimented that higher tensile strength is not necessarily required for excellent long-term performance in any climate because slab thickness is an effective parameter in controlling the fatigue resistance (Hansen et al., 2001). However, quality of subgrade and compressive strength of the concrete affect the PCC slab thickness design.

In general, variation in thickness affects pavement performance such as IRI, faulting, and cracking. For instance, predicted 20-year IRI decreases as concrete slab thickness increases (Khanum et al., 2005). Among pavements built on fine-grained soils, those with 8-inch PCC slab have higher changes in IRI than those with 11-inch PCC slab (Chatti et al., 2005). Faulting performance improves in pavements with thicker slabs, (Khanum et al., 2005).

In addition, it has been reported that increasing slab thickness from 8 to 12 inches decreased pavement cracking from 90 percent to 0.3 percent.(Khanum et al., 2005). The study of LTPP data has also proved that occurrence of longitudinal and transverse cracking among pavements with thinner PCC slabs (8 inches) is higher than among those with an 11-inch thick PCC slab (Chatti et al., 2005).

14.2 Water-Cement Ratio

The water-cement ratio is defined as the mass of mix water divided by the combined mass of the cement and any additional cementitious admixtures. This ratio is important in determining the overall strength of the mix as well as other mechanical properties including creep and shrinkage. As an example, studying five JPCP projects along with LTPP data proved that decreasing water-cement ratio decreases shrinkage performance of concrete mixtures (Hansen et al., 2001) because total water content is directly related to volume shrinkage (ACPA, 2002). Consequently, the potential for uncontrolled cracking is directly related to water demand.

In general, increasing the water-cement ratio will decrease the strength of the mix (McCullough and Rasmussen, 1998) like compressive strength and tensile strength as well as modulus of elasticity (Huseyin, 2007). On the other hand, it has been observed that a decrease of 4 percent of water-cement ratio (from 48 percent to 44 percent) in mixes of four different types aggregate results in increase of flexural strength in a range of 11.2 to 13.4 percent (Darter et al., 1993). It has to be mentioned though that increasing the cement content instead of reducing the water-cement ratio has a detrimental effect on pavement performance, like early transverse cracking (Hansen et al., 2001).

Water-cement ratio is also one of the main factors which have variability in production as well as in the field that greatly affect freeze-thaw durability of concrete (Hodgson, 2000). Therefore, it should be selected so that it is as low as possible while still maintaining a workable mix.

14.3 Coefficient of Thermal Expansion (CTE)

The Coefficient of Thermal Expansion (CTE) of concrete is more greatly influenced by the CTE of aggregate particles than of cement paste, since 70 to 75 percent of total solids volume of a concrete mixture is aggregate particles. However, it has been cautioned that if the CTE of the aggregate differs too much from the cement paste, a large change in the temperature may induce a break in their bond (Al-Ostaz, 2007) and affect performance of concrete.

Temperature sensitivity of concrete products is greatly influenced by the coarse aggregate portion of mixtures. Therefore, concrete, which is more temperature sensitive, will expand or contract more with temperature change and there will be an increase in potential for uncontrolled cracking (ACPA, 2002).

It has also been observed that pavements constructed with aggregates of low CTE, while all other factors being equal in the mixtures, will generally perform better than those constructed with aggregates of a higher CTE (McCullough and Rasmussen, 1998). In other words, lowering the drying shrinkage and CTE of concrete mixture could minimize the risk of cracking and problems related to exposed cracks (Hansen et al., 2001).

14.4 Aggregate Gradation

Aggregates constitute the largest portion of portland cement concrete in terms of both volume and mass. Therefore, their proportions and properties will dominate the overall properties of the mix. For instance, lack of fines in aggregate creates an open void structure, allowing water to percolate from the surface down through the interconnected voids and affects the performance of the mix.

It has been reported that the compressive strength of a concrete mix and its effective air void content are dependents of the size and gradation of the aggregates (Ghafoori and Dutta 1995; Crouch et al., 2007). As the aggregate size decreases, the number of particles per unit of volume increases; and as the amount of particles increases, the binding area increases, resulting in improved strengths.

Adjusting aggregate proportions, such as gradation, shape, and amount, in a mix is one of the factors that control the air void content of the mixture. Optimized effective void content plays a major role in the properties of hardened concrete like strength and permeability (Crouch et al., 2007). It has been observed that tough, large-size coarse aggregate particles improve fracture behavior of PCC (Hansen et al., 2001).

Besides gradation, the effects of aggregate source on concrete pavement performance have been studied (Stark and Klieger, 1973). It has been reported that susceptibility of PCC pavements to distresses like D-Cracking may be governed by the source of coarse aggregates. Field observations in this study have indicated that reducing the maximum particle size from one-and-half inch to one inch and also half an inch can greatly reduce the rate of development of D-Cracking or possibly eliminate it due to control of expansions.

14.5 Tensile Strength

Concrete mixes are proportioned on the basis of achieving the desired compressive strength at the specified age. However, it is believed that the material characteristics, in general, affect the tensile properties in a similar manner as the compressive strength (Hansen et al., 2001). Limited literature exists, though, on the effect on flexural and tensile strength as the compressive strength increases (ACI Committee 363, 1984; and Tachibana et al., 1981)

Tensile strengths often play a vital role in concrete performance. It represents one of the most important mechanical properties of concrete as it relates to PCC's resistance to crack initiation. It has been reported that fatigue transverse cracking and corner breaks are distresses directly related to the PCC tensile properties (e.g. flexural strength) whereas spalling is primarily a durability-associated distress and to a lesser extent related to tensile strength (Hansen et al., 2001).

14.6 Cement Content

Cement content in concrete mixtures affects strength of the product. An investigation by Huseyin et al. (2007) has suggested that reducing cement dosage in a mixture provides insufficient paste volume to surround the aggregates. Hence there will be a clear decrease in concrete strength. Moreover, the same study has reported a slight increase in strength for cement content dosage of more than optimum (Huseyin et al., 2007). In general, it can be believed that increasing the cement content and lowering the water-cement ratio of the mixture helps in producing a denser and more durable mixture with higher early strength (ACPA, 2002).

On the other hand, although any increase in cement content will contribute to a higher potential for uncontrolled cracking, it may result in smaller aggregate proportion. If this happens, the modulus of elasticity of the mixture generally rises. This effect can be explained through improvement in compressive strength due to higher cement content (Huseyin et al., 2007).

Mixtures with higher quantities of cement require more mixing water and consequently shrink more. Even if the water-cement ratio is minimized, the actual volume of water increases with higher cement content (ACPA, 2002).

The investigation of 15 JPCP projects in the LTPP database has shown that for a 40-percent increase in cement content will cause about a 40-percent increase in CTE of the mixture (Hansen et al., 2001).

14.7 Construction Temperature

It has long been recognized that PCC pavements develop “Built-In” curling depending upon the climatic conditions at the time of paving. Studying effects of temperature on responses of concrete pavements has shown that controlling the time of paving can considerably alleviate slab curling in the long term, which will improve fatigue performance of the pavement (Rao, 2001).

High concrete temperatures increase the rate of hydration, thermal stresses, the tendency for drying shrinkage cracking and permeability. Therefore, long-term concrete strengths decrease and, as a result, cracking occurs and concrete durability is lost (Schlinder, 2002).

14.8 Other Factors

Kurtis and Monteiro (1999) examined the damage caused to concrete pavements through deleterious reactions such as sulfate attack, aggregate reactions, corrosion, and freeze-thaw action. It was reported that:

- Low permeability concrete produced from sulfate-resistant Portland cements, Portland-pozzolan blends, calcium aluminate cements and blends have an improved performance and alkaliaggregate reaction resistance in sulfate-rich environments, while the resistance of calcium sulfoaluminate cements is similar to that of Portland cement.
- Portland-pozzolan blends and fly ash-based cements have an improved resistance to oxidation reactions because of decreased permeability to water. The oxidation of sulfide and sulfate minerals in aggregate may cause concrete cracking and aggregate pop-outs.
- Portland cement, Portlandpozzolan blends, and calcium sulfoaluminate cements provide resistance to corrosion.

Voigt (2002) provides a summary of the reasons and recommendations for minimizing cracking. It provides not only a comprehensive review of the factors that contribute to uncontrolled cracking, including proper concrete mixture design and jointing techniques that can minimize risk of early uncontrolled cracking, but also a summary of industry standard practice for uncontrolled crack repair. This was also published as an official ACPA bulletin.

14.9 Review of Different Non-Destructive Tests

A detailed review of non-destructive tests is provided in Appendix B. The following sections give a brief overview of some of the relevant tests for rigid pavements.

14.9.1 Thickness

Pavement layer thickness is an important factor in determining the quality of newly constructed pavements and overlays, since deficiencies in thickness reduce the life of the pavement. In order to implement pavement thickness as a measure of quality assurance, it is necessary to have an accurate and reliable method for making the thickness measurement. Cores are accurate, but they are time consuming, they damage the pavement, and they represent a very small sample of the actual pavement. Therefore, it is desirable to have a thickness measuring method which is quick, non-destructive, and which can generate an accurate and representative population of pavement thickness data points. Some of the non destructive test methods available for thickness measurements are listed below with their key features from the point of view of their suitability of use in a quality assurance (QA) program.

14.9.1.1 Ground-penetrating radar (GPR)

Ground-penetrating radar (GPR) is a high resolution geophysical technique that utilizes electromagnetic radar waves to scan shallow subsurface. It can provide information on pavement layer thickness or locate targets (Daniels, 1990; Hasted, 1973; Ulriksen and Peter, 1982; Harris, 1998). Frequency of GPR antenna affects depth of penetration. Lower frequency antennas penetrate further, but higher frequency antennas yield higher resolution. To successfully provide pavement thickness information or scan an interface, the following conditions have to be present:

- Physical properties of the pavement layers must allow for penetration of the radar wave.

- Interface between pavement layers must reflect the radar wave with sufficient energy to be recorded.
- Difference in physical properties between layers separated by interfaces must be significant.

Physical (electrical) properties of pavement layers, thickness of pavement layers, and magnitude of difference between electrical properties of successive pavement layers impact the ability to detect thickness information using GPR. Conductive losses occur when electromagnetic energy is transformed into thermal energy to provide for transport of charge carriers through a specific medium. Presence of moisture or clay content in a pavement layer will cause significant conductive losses and hence will increase the dielectric permittivity and decrease the depth of penetration. The GPR wave attenuates more rapidly in concrete, especially new concrete, than it does in asphalt (Ulriksen, 1993). This is due to the free moisture and conductive salts that are present in the concrete mix. Also, the dielectric constant between concrete and base is much smaller than it is between asphalt and base. Therefore, air-coupled GPR is not a feasible technology for thickness measurement on new concrete. Ground-coupled GPR, on the other hand, provides more energy input into the pavement, and can overcome some of the penetration limitations of the horn antenna. However, ground-coupled GPR requires slower survey speeds than does air-coupled GPR.

14.9.1.2 Mechanical wave methods for concrete thickness evaluation

Mechanical wave methods are very similar in concept to electromagnetic wave methods. With mechanical wave methods, a pulse of mechanical energy is transmitted into the pavement, and a transducer receives the reflected waves from the pavement layers. Analysis of these reflected-return signals yields information on the pavement layer thickness and mechanical material properties.

Mechanical wave techniques (impact-echo and others) work much more effectively than GPR in concrete. Mechanical waves travel well in concrete, and there is usually a strong mechanical contrast between the concrete and the base material. Data collection is considerably slower (because it is point specific) than with GPR, but certainly faster and less expensive than coring.

14.9.1.2.1 Impact-Echo

Impact-echo (IE) is a technique developed for thickness measurement and delamination location in concrete. Several different sources of commercial equipment are available. Recent studies show that impact-echo technique can be used for concrete early-strength gain estimation and evaluation of micro-cracking and chemical attacks in concrete structures (IAEA, 2005). A number of concrete pavement thickness accuracy studies have been carried out over the past several years. A summary of the results of these studies is shown in Table 14.1.

The differences shown between the impact-echo and core data in Table 14.1 are generally small. However, discussions with experienced practitioners have indicated that the small differences shown in Table 14.1 are not typical of field practice.

Table 14.1 Summary of Previous Impact Echo Concrete Pavement Thickness Studies

Location/reference	subsite	Core(mm)		Impact Eco(mm)		Difference of Mean
		mean	ST Dev	mean	ST Dev	
Indiana	n.a.	361	9	364	15	-4
Nebraska	n.a.	256	4	253	4	3
Virginia	Route 460	242	9	242	9	0
	Route 64	208	6	209	8	-1
Arizona	200-LCB	205		203		2
	200-ASPB	209		212		-3
	200-DGAB-1	197		195		2
	200-DGAB-2	212		209		3
	300-LCB	294		291		3
	300-ASPB	294		300		-6
	300-DGAB-1	288		279		9
	300-DGAB-2	287		279		8

14.9.1.2.1.1 Advantages and disadvantages of impact-echo

Advantages:

- Equipment is commercially available,
- Capable of locating a variety of defects,
- Does not require coupling material,
- Access to only one face is required
- Light weight, portable

- Locate flaws as well as accurately determine at what depth the flaws are occurring
- Results are achieved very correctly and quickly (<10s) through the use of a portable computer

Limitations:

- Experienced operator is required,
- Current instrumentation limited to testing members less than 2 meters thick

14.9.1.2.2 MIT SCAN-2

MIT SCAN-2 was developed for verification of dowel bar positions. It uses methods of the Electro-Magnetic Tomography (Yu and Khazanovich, 2005). Dowel bar depths can be determined up to +/- 2mm, misalignments up to +/-4, and side shifts upto +/- 8mm. (MIT on-line brochure). A sister device termed **MIT SCAN-T2** can be used for measurement of asphalt and concrete pavement thickness. Some of the pertinent characteristics of this device include (CalTrans, 2007):

- Provides immediate measurement
- Can measure from 0 inch to 20 inches
- Commercially available reflectors can be used
- No on-site calibration is required
- Can measure thickness of fresh concrete
- Can measure thickness of milled surface also
- Accuracy is +/- 0.5% of measurement value + 1mm
- Resolution is 0.04 inch
- No disturbances by wet road covers or magnetic aggregates
- Check of dimensions & conditions of reflectors

14.9.2 Density

14.9.2.1 Thickness and Density (Radioisotope Gauges)

The use of radioisotopes for the non-destructive testing of concrete is based on directing the gamma radiation from a radioisotope against or through the fresh or hardened concrete. When the reflected pulses are counted, the resulting count or count rate is a measure of the dimensions or physical characteristics, e.g. density of the concrete. Although this radiometry method has not been commonly used on concrete, the increasing use of radioisotopes to measure the compaction of asphalt or bituminous concrete and the soil-aggregate mixtures used in road construction indicates that the method may be more commonly used in the future.

For typical, commercially available backscatter density gauges, the top 25 mm of concrete sample yields 50 to 70% of the density reading, the top 50 mm yield 80 to 95%, and there is almost no contribution from below 75 mm.

14.9.2.1.1 Applications of thickness and density gauges

Currently no procedures are in standard use to measure the in-place quality of concrete immediately after placement; that quality is not assessed until measurements such as strength, penetration resistance, and/or smoothness can be made after the concrete has hardened.
Gamma radiometry is also being used extensively for monitoring the density of roller compacted concrete. Densification is critical to strength development in these mixtures of cement (and pozzolans), aggregates and a minimal amount of water. After placement, the concrete is compacted by rollers, much the same as asphalt concrete pavements.

A short lived but interesting application of gamma radiometry is in pavement thickness determinations. Researchers placed thumbtack-shaped 46Sc sources on a pavement sub-base before a PCC pavement was placed. The sources were difficult to locate after the concrete was placed, however, and the technique was abandoned albeit with a recommendation that it deserved further research.

14.9.2.1.2 Advantages and limitations of thickness and density gauges

Gamma radiometry offers engineers a means for rapidly assessing the density and, therefore, the potential quality of concrete immediately after placement. Direct transmission gamma radiometry has been used for density measurements on hardened concrete, but its speed, accuracy, and need for internal access make it most suitable for quality control measurements before newly placed concrete undergoes setting.

Backscatter gamma radiometry is limited by its inability to respond to portions of the concrete much below the surface, but it can be used over both fresh and hardened concrete and can be used, in non-contact devices, to continuously monitor density over large areas. Gamma radiometry techniques have gained some acceptance in density monitoring of bridge deck concrete and fairly widespread acceptance for density monitoring of roller-compacted concrete pavement and structures.

Summary of the advantages and limitations of backscatter and direct transmission gamma radiometry techniques is given in Table 14.2.

Table 14.2 Advantages and Limitations Various Gamma Radiometry Techniques

Technique	Advantages	Limitations
Gamma radiometry for Density	Technology well developed; rapid, simple, rugged and portable equipment; moderate initial cost; minimal operator skill	Requires license to operate; requires radiation safety program
Backscatter mode	Suitable for fresh or hardened concrete; can scan large volumes of concrete continuously	Limited depth sensitivity; sensitive to concrete's chemical composition and surface roughness
Direct transmission mode	Very accurate; suitable primarily for fresh concrete; low chemical sensitivity	Requires access to inside or opposite side of concrete

14.9.2.2 Moisture Gauges

Moisture gauges consist of a source of neutron radiation, which irradiates the material under test. As a result of radiation, gamma rays are created and detected. The result is a series of counts, which are a measure of the composition of the concrete. It can be used to measure moisture content of concrete, soil and bituminous materials and to map moisture migration patterns in masonry walls. Their application to concrete testing is very recent and still in the exploratory stage.

Advantages:

- Instrument is portable
- Moisture measurements can be made rapidly

Limitations:

- A minimum thickness of surface layer is required for backscatter to be measured,
- It measures only the moisture content of surface layer (50 mm),
- It emits radiation,
- Results are inaccurate because hydrogen atoms of building materials are measured in addition to those of water,
- Its use in concrete is limited and requires calibration in order to calculate density or moisture content

14.9.2.3 Air Void Analyzer

The principal test method for measuring air entrainment in hardened concrete is ASTM C 457, "Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete". However this procedure is highly labor intensive and is rarely used in QA programs. Some people have praised the value of using AVA for QC/QA (AASHTO Technology Implementation Group) whereas some others such as Caltrans found that it may not be practical to use this instrument on their bridge construction. They found that AVA process requires a very stable base to allow the finite air bubbles to be measured. At the same time Caltrans has developed a draft California test method to use AVA in freeze thaw conditions. Missouri Department of Transportation has also used AVA successfully.

14.9.3 Uniformity test

14.9.3.1 Pulse velocity test

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete from the transducer using a liquid coupling material such as grease or cellulose paste, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. The equipment consists essentially of an electrical pulse generator, a pair of transducers, an amplifier and an electronic timing device for measuring the time interval between the initiation of a pulse generated at the transmitting transducer and its arrival at the receiving transducer.

Measurement of the velocity of ultrasonic pulses of longitudinal vibrations passing through concrete may be used for the following applications (IAEA, 2002):

- determination of the uniformity of concrete in and between members
- measurement of changes occurring with time in the properties of concrete
- correlation of pulse velocity and strength as a measure of concrete quality
- determination of the modulus of elasticity and dynamic Poisson's ratio of the concrete

Pulse velocity measurements made on concrete structures may be used for quality control purposes. In comparison with mechanical tests on control samples such as cubes or cylinders, pulse velocity measurements have the advantage that they relate directly to the concrete in the structure rather than to samples, which may not be always truly representative of the concrete in situ.

The number of individual test points depends upon the size of the structure, accuracy required and variability of the concrete. In a large unit of fairly uniform concrete, testing on a 1m grid is usually adequate but, on small units or variable concrete, a finer grid may be necessary.
The use of the ultrasonic pulse velocity technique to detect and define the extent of internal defects should be restricted to well-qualified personnel with previous experience in the interpretation of survey results.

Pulse velocity measurements are particularly useful to follow the hardening process, especially during the first 36 hours. Here, rapid changes in pulse velocity are associated with physiochemical changes in the cement paste structure, and it is necessary to make measurements at intervals of 1 hour or 2 hours if these changes are to be followed closely.

The relationship between the elastic constants and the velocity of an ultrasonic pulse traveling in an isotropic elastic medium allows one to determine dynamic modulus and Poisson's ratio of concrete using this technique.

14.9.3.1.1 Advantages and disadvantages of pulse-velocity test

The pulse velocity method is an excellent means for investigating the uniformity of concrete.
Advantages:

- rapidly survey large areas and thick members,
- simple method with readily available equipment,
- portable and as easy to use on the construction site as in the laboratory
- Testing procedures standardized by ASTM and other organizations. A large number of variables can affect the relation between the strength properties of concrete and its pulse velocity; therefore, it is important that a correlation between pulse velocity and compressive strength be developed for project mixes prior to any measurements in-situ (Crawford, 1997).

Disadvantages:

- requires proper surface preparation,
- time consuming as it takes only point measurements,
- skill is required in the analysis of results as moisture variations and presence of metal reinforcement can affect results,
- The interpretation of ultrasonic test results based on published graphs and tables can be misleading. It is therefore necessary that correlation with the concrete to be inspected is carried out. It works on single homogenous materials.

14.9.3.2 Surface Hardness Test

The rebound hammer has been used to estimate the in-situ compressive strength of concrete. It has also been used to assess the overall uniformity of concrete prior to undertaking more extensive destructive tests, such as coring. The rebound hammer is easy to use and provides a large number of readings in a short time. However, extreme care should be taken in evaluating the results. Frequent calibration of the hammer is also required to ensure the greatest accuracy.

14.9.3.2.1 Advantages and disadvantages of surface hardness test

Advantages:

- Provides a quick and inexpensive means of assessing the general quality of concrete and for locating areas of poor quality
- Takes large number of readings rapidly, so scan large exposed areas in few hours

Disadvantages:

- Because the test only measures the rebound of a given mass on the concrete surface, the results reflect only the quality of the surface, not the entire depth,
- The results of the test are affected by the smoothness of the test surface, type of coarse aggregate, age of concrete being tested, moisture content, type of cement, and surface carbonation.
- The rate of gain of surface hardness of concrete is rapid for the first 7 days, after which there is little or no gain in surface hardness. However, for properly cured concrete, there is a significant strength gain beyond 7 days, because cement continues to hydrate within the concrete and gain strength. When concrete over 28 days is to be tested, direct correlations need to be developed between the rebound numbers taken on the concrete and the compressive strength of cores taken from the concrete.
- Caution should also be exercised when testing concrete less than 3 days old or concrete with expected compressive strengths less than 7 Mpa (1000 psi). The reason for this is that the rebound numbers will be too low for an accurate reading, and the rebound hammer will leave blemishes on the concrete surface when impacted.
- The presence of surface moisture and the overall moisture content of the concrete have a profound effect on the results of the rebound hammer test.

- The type of cement can have a significant effect on the rebound number. Concrete containing type 3 high-early-strength cement can have higher rebound numbers at an early age than concrete made with type 1 cement.

- The rebound numbers for carbonated concrete can be up to 50 percent higher than those obtained on a non-carbonated concrete surface.

- For equal compressive strengths, concrete made with crushed limestone shows rebound numbers approximately 7 points higher than those for concretes made with gravel, representing approximately 7 Mpa (1000 psi) difference in compressive strength. The same type of coarse aggregate obtained from different sources can yield different concrete strength estimations. Correlation testing of materials is necessary. A general correlation exists between the compressive strength of concrete and the hammer rebound number. However, there is a big disagreement among researchers concerning the accuracy of the hammer for estimating the compressive strength of concrete. These large deviations can be reduced by developing a proper correlation curve for the hammer that takes into account the variables discussed earlier, instead of relying on the correlation curves provided by the manufacturer of the rebound hammer.

- For a properly calibrated hammer, the accuracy is between 15 and 20 percent for test specimens cast, cured, and tested under lab conditions. However, the accuracy of the rebound hammer for estimating in-situ compressive strength is approximately 30 to 40 percent.

14.9.3.2.3 Summary

The Schmidt hammer should not be regarded as a substitute for standard compression tests but as a method for determining the uniformity of concrete in structures, and comparing one concrete against another. Estimation of the strength of concrete by the rebound hammer within an accuracy of ± 15 to 20 percent may be possible only for specimens cast, cured, and tested under similar conditions as those from which the correlation curves are established.

14.9.4 Strength of Concrete

14.9.4.1 Penetration Resistance or Windsor Probe Test

The Windsor probe, like the rebound hammer, is a hardness tester, and its inventors' claim that the penetration of the probe reflects the precise compressive strength in a localized area, is not strictly true. However, the probe penetration does relate to some property of the concrete below the surface, and, within limits, it has been possible to develop empirical correlations between strength properties and the penetration of the probe (IAEA, 2002). It is, therefore, imperative for each user of the probe to correlate probe test results with the type of concrete being used.

The Windsor probe test has been used to estimate the early age strength of concrete in order to determine when formwork can be removed. The simplicity of the test is its greatest attraction. The depth of penetration of the probe, based on previously established criteria, allows a decision to be made on the time when the formwork can be stripped. If the standard cylinder compression tests do not reach the specified values or the quality of the concrete is being questioned because of inadequate placing methods or curing problems, it may be necessary to establish the in situ compressive strength of the concrete. This need may also arise if an older structure is being investigated and an estimate of the compressive strength is required. In all those situations the usual option is to take a drill core sample since the specification will generally require a compressive strength to be achieved. It is claimed, however, that the Windsor probe test is superior to taking a core. With a core test, if ASTM C42 –87 is applied, the area from which the cores are taken needs to be soaked for 40 h before the sample is drilled. Also, the sample often has to be transported to a testing laboratory which may be some distance from the structure being tested and can result in an appreciable delay before the test result is known. Swamy and Al-Hamed report that the Windsor probe estimated the wet cube strength to be better than small diameter cores for ages up to 28 days. For older concrete, the cores estimated the strength better than the probe.

14.9.4.1.1 Advantages and Limitations of Windsor probe test

Advantages:

- The test is relatively quick and the result is achieved immediately provided an appropriate correlation curve ,
- The probe is simple to operate, requires little maintenance except cleaning the barrel and is not sensitive to operator technique,
- Access is only needed to one surface,
- The correlation with concrete strength is affected by a relatively small number of variables,
- The equipment is easy to use and does not require surface preparation prior to testing,
- It is good for determining in situ quality of concrete,

- The results are not subject to surface conditions, moisture content or ambient temperature.
- The test result is likely to represent the concrete at a depth of from 25 mm to 75 mm from the surface rather than just the property of the surface layer as in the Schmidt rebound test.

Limitations:

- The minimum acceptable distance from a test location to any edges of the concrete member or between two test locations is of the order of 150 mm to 200 mm,
- The minimum thickness of the member, which can be tested, is about three times the expected depth of probe penetration,
- The distance from reinforcement can also have an effect on the depth of probe penetration especially when the distance is less than about 100 mm,
- The test is limited to <40 Mpa and if two different powder levels are used in an investigation to accommodate a larger range of concrete strengths, the correlation procedure becomes complicated,
- The test leaves an 8 mm hole in the concrete where the probe penetrated and, in older concrete, the area around the point of penetration is heavily fractured,
- On an exposed face the probes have to be removed and the damaged area repaired,
- Calibration by manufacturers does not give precise prediction of strength for concrete older than 5 years and where surface is affected by carbonation or cracking.
- Calibration based on cover is necessary for improved evaluation.

14.9.4.2 Pullout Test

A Pullout test, by using a dynamometer and a reaction bearing ring, measures the force required to pullout from concrete a specially shaped insert whose enlarged end has been cast into the concrete. The pullout test has been adopted as a standard test method in many parts of the world, including North America, and has been used successfully on numerous large construction projects. Primary use of the system has been in either controlling formwork removal or the time of post–tensioning, or determining the minimum amount of curing needed in cold weather concrete placement.

14.9.4.2.1 Advantages and Disadvantages of Pullout Test

Advantages:

- It provides a direct measure of the in situ strength of concrete.
- The method is relatively simple and testing can be done in the field in a matter of minute.

Disadvantages:

- Minor damage to the concrete surface must be repaired,
- The standard pullout tests have to be planned in advance, and unlike other in situ tests, cannot be performed at random after the concrete has hardened.

14.9.4.3 Break-Off Test

Out of the many currently available NDT methods, only the Break-Off test and the Pullout tests measure a direct strength parameter (Naik, 1991). The Break-Off test consists of breaking off an in-place cylindrical concrete specimen at a failure plane parallel to the finished surface of the concrete. Break-Off test is not very widely used in North America. The primary factor in limiting the widespread use of this method is the lack of necessary technical data and experience in North America. Initial work at the Canada center for Minerals and Energy Technology (CANMET) in the early 1980s indicated inability to reproduce results of this test method (Naik, 1991).

The Break-Off method can be used both as quality control and quality assurance tools. The most practical use of the Break-Off test equipment is for determining the time for safe form removal and the release time for transferring the force in prestressed or post-tensioned members.

14.9.4.3.1 Advantages and Limitations of Break-off Test

Advantages:

- Ability to measure in-place compressive strength
- Safe, simple to use
- Test is quickly performed , requires only one exposed surface
- Reproducible to an acceptable degree of accuracy and correlates well with the compressive strength of concrete.

Disadvantages:

- The damage to the concrete member that requires patching

14.9.4.4 Maturity Test

The maturity concept is a useful technique for estimating the strength gain of concrete at early ages, generally less than 14 days old. The method accounts for the combined effects of temperature and time on concrete strength development. An increase in the curing temperature can speed up the hydration process which will increase the strength development. Maturity is a function of the product of curing time and internal concrete temperature. It is then assumed that a given mix at equal maturities will have the same strength, independent of the curing time and temperature histories (Carino, 1991).

The maturity method has numerous applications in concrete construction:

- It has been used successfully to estimate in-place strength of concrete to assure critical construction operations. Such as form removal or the application of prestressing or post-tensioning force.
- To determine when vehicles can be turned on to new pavement construction or the opportune time to saw joints in concrete pavement,
- Some of the more advanced maturity techniques, such as the Computer Interactive Maturity System (CIMS) can be used for quality control and concrete mix verification.

14.9.4.4.1 Advantages and Disadvantages of Maturity Test

Advantages:

- Useful, easily implemented, accurate means of estimating *in-situ* concrete strength.
- Quality assurance costs can be reduced because the number of test cylinders is reduced by using the maturity concept.

Disadvantages:

- There is a need for correlation with laboratory work.

14.9.5 Hidden Flaws

14.9.5.1 Infrared Thermography

The thermograms taken with an infrared camera measure the temperature distribution at the surface of the object at the time of the test. Naturally any interior 'structure' has an effect on the temperature distribution on the surface. All the information revealed by the infrared system relies on the principle that heat cannot be stopped from flowing from warmer to cooler areas, it can only be slowed down by the insulating effects of the material through which it is flowing.

Thermographic testing techniques for determining concrete subsurface voids, delaminations, and other anomalies have advantages over destructive tests like coring and other NDT techniques such as radioactive/nuclear, electrical/magnetic, and acoustic and radar techniques.

14.9.5.1.1 Advantages and Disadvantages of Infrared Thermography

Advantages:

- Major concrete areas need not be destroyed during testing.

- Only small calibration corings are used.
- Major savings in time, labor, equipment, traffic control, and scheduling problems.
- When aesthetics are important, no disfiguring occurs on the concrete to be tested.
- Rapid set up and take down, when vandalism is possible.
- No concrete dust and debris are generated that could cause environmental problems.
- Infrared thermographic equipment is safe as it emits no radiation.
- It only records thermal radiation, which is naturally emitted from the concrete, as well as from all other objects. It is similar in function to an ordinary thermometer, only much more efficient.
- It is an area testing technique, while the other NDT methods are mostly either point or line testing methods.
- Infrared thermography is capable of forming a two dimensional image of the test surface showing the extent of subsurface anomalies.
- Portable and permanent records can be made.
- Testing can be done without direct access to surface and large areas can be rapidly inspected using infrared cameras.

Disadvantages:

- The depth or thickness of a void cannot be determined, although its outer dimensions are evident. It cannot be determined if a subsurface void is near the surface or farther down at the level of the reinforcing bars.
- Equipments are expensive and require highly skillful and experienced operator.
- It is very sensitive to thermal interference from other heat sources. Moisture on the surfaces can also mask temperature differences.

14.9.5.2 Betatron PXB - 7.5 MeV (Force technology, 1999)

The Portable X-ray Betatron (PXB) produces X-ray beams with an energy level of 7.5 MeV. With such high energy, the X-rays can penetrate thick concrete and steel, and reveal flaws inside the concrete structure by high quality X-ray images. The radiation levels outside the main beam are low. It is suitable for both in-lab and in-situ operations.

14.9.5.2.1 Applications

The Betatron is typical being used for:

- Mapping of the reinforcement (size, depth, position, configuration and condition)
- Studying the homogeneity of the concrete (voids)

14.9.5.2.2. Performance and advantages

- It is possible to fulfill the Nuclear Energy Agency requirements: x-ray detect ability of 20 mm porosity in 1000 mm thick concrete.

- It is possible to detect from 5% - 20% loss of thickness in cables and reinforcement depending on the direction of exposure.
- The depth placement of reinforcing bars can be determined by means of image processing if the nominal diameter of the bar is known.
- It is possible to determine the approximate depth of a void by calculating a void density factor.

14.9.6 Modulus of Pavement Layers

Making accurate assessments of the structural condition of roads during construction helps in locating weak areas prone to localized failure and correcting them prior to completion of the pavement. NDT tests that are specific to characterizing unbound materials were discussed in section 5.7.3 under flexible pavements. These include:

- Humboldt Stiffness Gauge
- Portable light weight FWD
- Dynamic Cone Penetrometer (DCP)

14.9.6.1 Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is a nondestructive testing device widely used for assessing the structural condition of a pavement. When complete deflection basins are available, deflection testing can provide key properties for the existing pavement structure through backcalculation of the measured pavement responses, Specifically, for portland cement concrete (PCC) pavements, the elastic modulus (E) of the slab and the modulus of subgrade reaction (k or k-value) can be determined. In addition, deflection testing conducted on PCC pavements can be used to estimate the load transfer efficiency (LTE) across joints or cracks as well as for the identification of loss of support at slab corners. Also, FWD data can be used for determining the presence of built-in curling. The advantages and disadvantages of the FWD were listed in section 5.7.3.2.

14.9.7 Seismic Pavement Analyzer

The Seismic Pavement Analyzer (SPA) has been discussed in section 5.7.4 under flexible pavements. A study concluded that testing rigid pavements at ambient temperatures in excess of 35°C is not feasible (Nazarian et al., 1993). Also, to minimize the effects of fluctuation in the moisture level due to precipitation, the equipment should not be used until one day after significant precipitation.

CHAPTER 15: Empirical Data analysis – Rigid Pavements

As reported in Chapter 11, an attempt was made to collect data from Michigan rigid pavement construction projects. However, the data search led to the finding that most of the construction records were either lost or unaccounted for. Therefore, alternative sources of data needed to be explored to determine how quality characteristics used in QA programs affect pavement performance. A preliminary analysis was first performed to study the relationship of acceptance parameters (e.g., thickness and strength) to performance (e.g., cracking and faulting) using data from Long Term Pavement Performance (LTPP) projects. In this analysis, data from several states was used. These states geographically lie in different climatic zones. The LTPP database contains performance data (cracking, faulting, IRI etc.) and design and construction data (including physical inventory data, material properties from in-situ and laboratory tests). For the preliminary analysis, all the data were derived from the Specific Pavement Studies – 2 (SPS - 2) experiment. This analysis was followed by alternative analysis with data from General Pavement Studies (GPS) experiments.

Table 15.1 lists categories of data that were extracted from the LTPP database. The data were collected for multiple states. There were very few data points available for the state of Michigan.

15.1 Analysis Using Percent-Within-Limits Concept

Similar to the data for flexible pavements, LTPP documents a variety of data for rigid pavements. Compressive strength is the only critical quality characteristic in the Michigan QA program because, although a check is done on slump and entrained air voids in concrete, payment to the contractor is determined solely on the compressive strength test results. The LTPP database does have compressive strength test results from projects all across the US. In LTPP surveys for rigid pavements, testing is performed at different locations on the same section of the pavement, and results corresponding to all these locations are registered in the database, unlike for the case of flexible pavements.

Table 15. 1 Data extracted from LTTP database.

Constructions number	Cement Type	Flexural Strength
Traffic opening date	Entrained Air	Type
Type of transverse construction joint	Mean	Age
Cement type	Minimum	Mean
Compressive strength age	maximum	Minimum
Compressive strength	Slump	Maximum
Age	Mean	Standard deviation
Mean	Minimum	Number of samples
Minimum	Maximum	Tensile Strength
Maximum	Standard deviation	Age
Standard deviation	Number of samples	Mean
Number of samples	Bulk Specific Gravity	Minimum
Joint Spacing	Fine Aggregate	Maximum
Transverse Joint type	Coarse Aggregate	Standard deviation
Dowel	Concrete Curing Method	Number of samples
Distance	Elastic Modulus	
Length	Mean	
Coating	Minimum	
Mix design	Maximum	
Fine Aggregate	Standard deviation	
Coarse Aggregate	Number of samples	
Cement	Method	
Water		

For some of these projects, the standard deviation of compressive strength is also documented. Therefore, PWL values were calculated for those projects. Although there were more than 18,000 compressive strength tests reported, only about 3,000 of them had standard deviation values. Also, documentation for two cement types were available. Figures 15.1 and 15.2 show plots of faulting versus PWL (compressive strength at 28 days) and age of the pavement. In both of these plots, data points have been represented by circles floating in the 3-D space rather than columns as was used in the case of flexible pavements. This is because with such a large number of points and several points from one project having the same age, pavements are faulting at different locations within the same project; columns would hide each other and most of the data points will not be distinguishable from each other. The shading of the circles in the plots is proportional to the magnitude of faulting. It is noticeable that many points also tend to fall in line for the same PWL. This is because the compressive strength and its standard deviation for different locations within the same project are reported to be the same, although with varying amounts of faulting. The plots do not show any clear relationship between PWL (compressive strength) and faulting performance in either case.

Figures 15.3 and 15.4 show plots of longitudinal and transverse cracking respectively against PWL (compressive strength) and age. Each of these plots has almost 22,000 data points. However, since distress surveys give cracking values at many different locations along the project and because there is almost no longitudinal and transverse cracking in most of the cases, the points fall on top of each other. Therefore, these plots also do not show any trend.

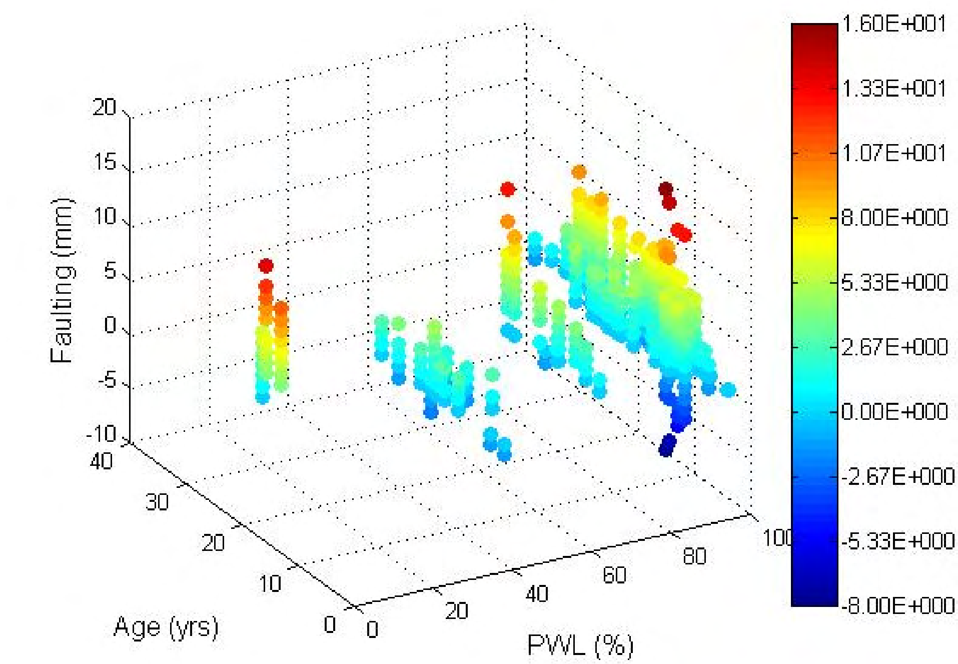


Figure 15. 1 PWL (compressive strength) Vs faulting – cement type 41.

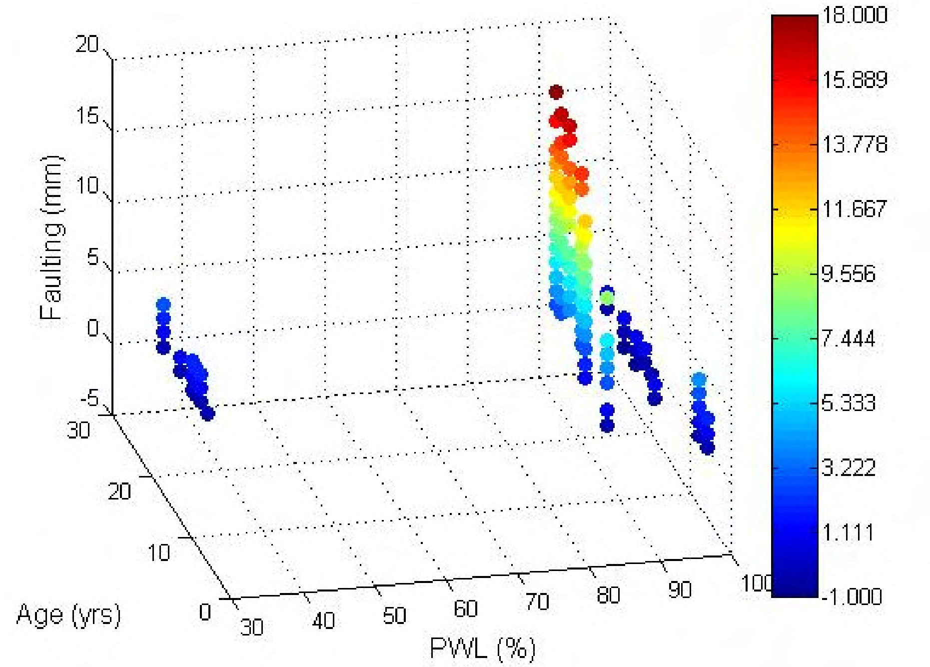


Figure 15. 2 PWL (compressive strength) Vs faulting – cement type 42

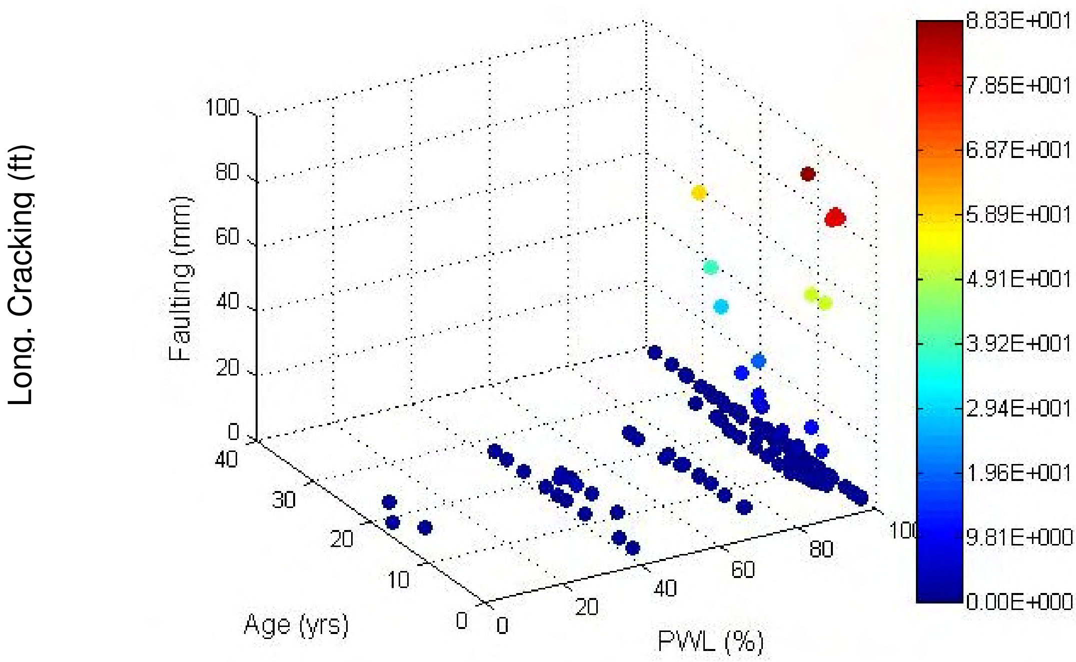


Figure 15. 3 PWL (compressive strength) Vs longitudinal cracking – cement type 41

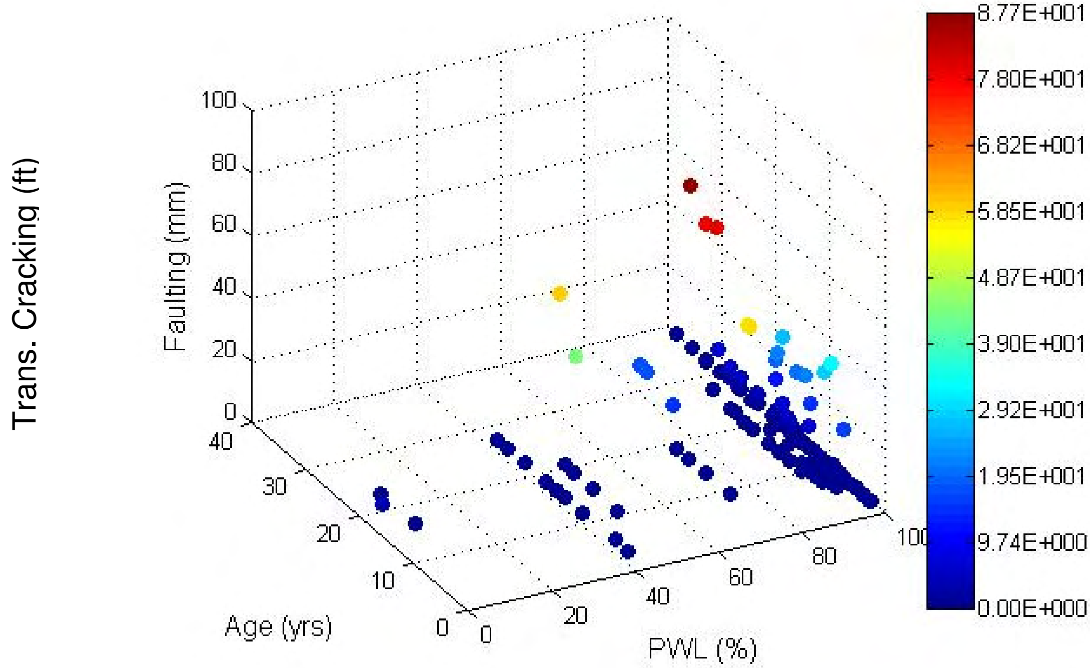


Figure 15. 4 PWL (compressive strength) Vs transverse cracking – cement type 41

15.2 Summary of Findings from LTPP Data Analysis

Since percent-within-limits takes into account mean as well as standard deviation of the quality characteristic it is a good measure of quality control exercised during the construction. However, it is important that PWL be related to actual pavement performance. An effort was made to find out if this holds true for pavements for which construction and performance data are available in LTPP database. In the case of flexible pavements, it was found that PWL for plant air voids seems to be affecting fatigue, longitudinal and transverse cracking but not rutting. Also, no clear trend was observed between PWL for in-situ density and cracking or rutting performance. These findings however, may be so because the performance of the pavement is affected by many factors and if PWL (in-situ density) does have influence on performance, it may be getting confounded because of those other factors. The solution to such a situation would be to separate the effects of different variables. But that is not easy either as far as LTPP data is concerned. The alternative would be to use models where only the desired variables can be studied while keeping all other factors as controlled factors.

In the case of rigid pavements also, no clear trend was observed between PWL for compressive strength and faulting and cracking performance. One of the reasons for this is that, despite the large number of data points available in the database, the variability (range) of independent variables (e.g., strength) was much smaller compared to performance. The argument regarding many other factors affecting pavement performance and confounding the findings on individual effects holds true in the case of rigid pavements also.

CHAPTER 16: Mechanistic-Empirical Analysis for Rigid Pavements

16.1 Analysis using MEPDG

MEPDG was extensively used to analyze the candidate QA variables for flexible pavements earlier in this project. In the case of rigid pavements, MEPDG software accepts inputs mainly corresponding to design of the pavement, e.g. amount of cementitious material, water to cement ratio etc., and fewer inputs with respect to construction, like temperature of fresh concrete before pouring, time of the day when the concrete was poured etc. However, two of the expectedly most significant variables, namely slab thickness and 28-day compressive strength of concrete can be studied using MEPDG.

In line with the analysis performed so far for flexible pavements, a set of 49 runs (Table 16.1) were designed corresponding to all possible combinations of 7 levels of slab thicknesses and 28-day concrete compressive strength values. The ranges for these variables were determined through study of actual project data in the LTPP database.

Performance predicted by MEPDG for the above mentioned 49 runs were gathered and response surfaces were generated in MATLAB to run actual simulations. The MATLAB code simulates actual projects with 50 sublots each and having varying mean and standard deviations for compressive strengths and thicknesses.

The simulation runs were the result of combining three different mean compressive strengths and 5 levels of mean slab thicknesses. Table 16.2 presents the 15 cases which were run. The mean values of thickness or strength do vary slightly even when they were meant to be fixed. This is because the mean values noted here are mean of the artificially generated thicknesses and strength in the simulation. The simulation also takes into account the fact that in reality when the contractor is producing concrete with strength near the lower specification limit, the distribution of sample strengths would be skewed inwards towards the allowable window than being symmetrically distributed. Table 16.2 also lists the resulting percent-within-limits (PWL) values for strength and thicknesses and average and percentile values of cracking. The results clearly show that when the PWL values are lower for both strength and thickness percent cracking is high. For example, in the case of run 1, one-fourth of the sublots would have more than 52% slabs cracked at the end of 30 years and one-tenth of sublots would have more than 90% of slabs cracked. However, in the case of run 2, when PWL for strength goes up to 81.9% one-fourth of the sublots have only 33.9% or more slabs cracked (compared to 52% in the case of run 1) and one-tenth of the sublots have 52% or more slabs cracked (compared to 90% in the case of run 1).

Table 16. 1 Compressive strength and slab thicknesses for MEPDG runs for rigid pavements

Run Number	Compressive Stregth (psi)	Slab Thickness (in)	Run Number	Compressive Stregth (psi)	Slab Thickness (in)
1	3000	6	26	4875	10
2	3000	7	27	4875	12
3	3000	8	28	4875	14
4	3000	9	29	5500	6
5	3000	10	30	5500	7
6	3000	12	31	5500	8
7	3000	14	32	5500	9
8	3625	6	33	5500	10
9	3625	7	34	5500	12
10	3625	8	35	5500	14
11	3625	9	36	6750	6
12	3625	10	37	6750	7
13	3625	12	38	6750	8
14	3625	14	39	6750	9
15	4250	6	40	6750	10
16	4250	7	41	6750	12
17	4250	8	42	6750	14
18	4250	9	43	8000	6
19	4250	10	44	8000	7
20	4250	12	45	8000	8
21	4250	14	46	8000	9
22	4875	6	47	8000	10
23	4875	7	48	8000	12
24	4875	8	49	8000	14
25	4875	9			

Table 16. 2 Details of cracking simulation runs with summary cracking results

Mean Comp. Strength (psi)	Mean Thickness	Run No.	PWL (Strength)	PWL (Thickness)	Avg. Cracking (%)	Cracking (in)75th Percentile	Cracking (in) 90th Percentile
3569.10	8.77	1	56.6	42.5	32.88	52.40	90.16
3886.00	8.78	2	81.9	40.0	20.66	33.86	52.84
4975.50	8.92	3	99.8	51.4	5.47	1.18	23.94
3579.20	9.52	4	56.5	84.3	8.97	7.98	32.30
3852.80	9.53	5	76.8	85.2	4.36	3.13	26.47
5016.10	9.53	6	100.0	84.9	1.54	1.04	1.40
3533.60	9.86	7	52.9	94.3	2.06	2.49	4.23
3954.60	10.00	8	79.7	95.4	1.12	1.53	3.22
5034.90	9.96	9	99.9	95.9	0.31	0.73	1.41
3498.60	10.48	10	49.9	100.0	1.42	3.26	3.74
3892.30	10.40	11	77.4	99.7	1.58	2.69	3.74
5036.90	10.53	12	99.9	99.8	0.00	0.30	1.17
3529.20	11.10	13	53.1	100.0	1.41	2.99	3.73
3999.20	11.33	14	88.4	100.0	1.55	2.29	3.41
5118.10	11.34	15	100.0	100.0	0.00	0.12	0.57

Figures 16.1 and 16.2 show percent of slabs cracked for the 50 sublots in each project pictorially for the first twelvve of these runs. Comparing plots 1 and 2, the overall improvement in performance of sublots is very clear. In the case of run 3 (refer to plot 3 in Figure 16. 1) even fewer sublots show a high amount of cracking. Run 3 represents the case when PWL for strength is close to 100%. Comparing the set of plots 1, 2 and 3 with the next set of plots 4, 5 and 6, we can see the effect of higher PWL value for thickness. Therefore, an increase in thickness by approximately 0.75 inches seems to improve cracking performance more than a strength increase of about 300 psi. However, when strength is increased by 1400 psi (refer to run numbers 1 and 3), 75th percentile cracking decreases from 52.4% to 1.18 %. It should be noted that even then, the 90th percentile for cracking is close to 24 percent or almost one fourth of all slabs. Figure 16.2 represents the cases which have better PWL values for thicknesses than those shown in Figure 16.1. The performance of these cases, with higher PWL for thickness, is far better with only about 2% of the slabs cracking on an average in the worst case (run number 7).

The above clearly shows that the effect of deviations from the target compressive strength and slab thickness is drastic. The simulations help us see how different sublots would perform over time rather than knowing just the average percent of slabs cracked.

Figures 16.3 and 16.4 show similar plots for IRI for the same twelve runs of the simulations. IRI also shows similar trend as slab cracking. Plots from runs simulating faulting are shown in Figure 16.5 and 16.6. Faulting does not seem to be appreciably affected by strength and thickness levels.

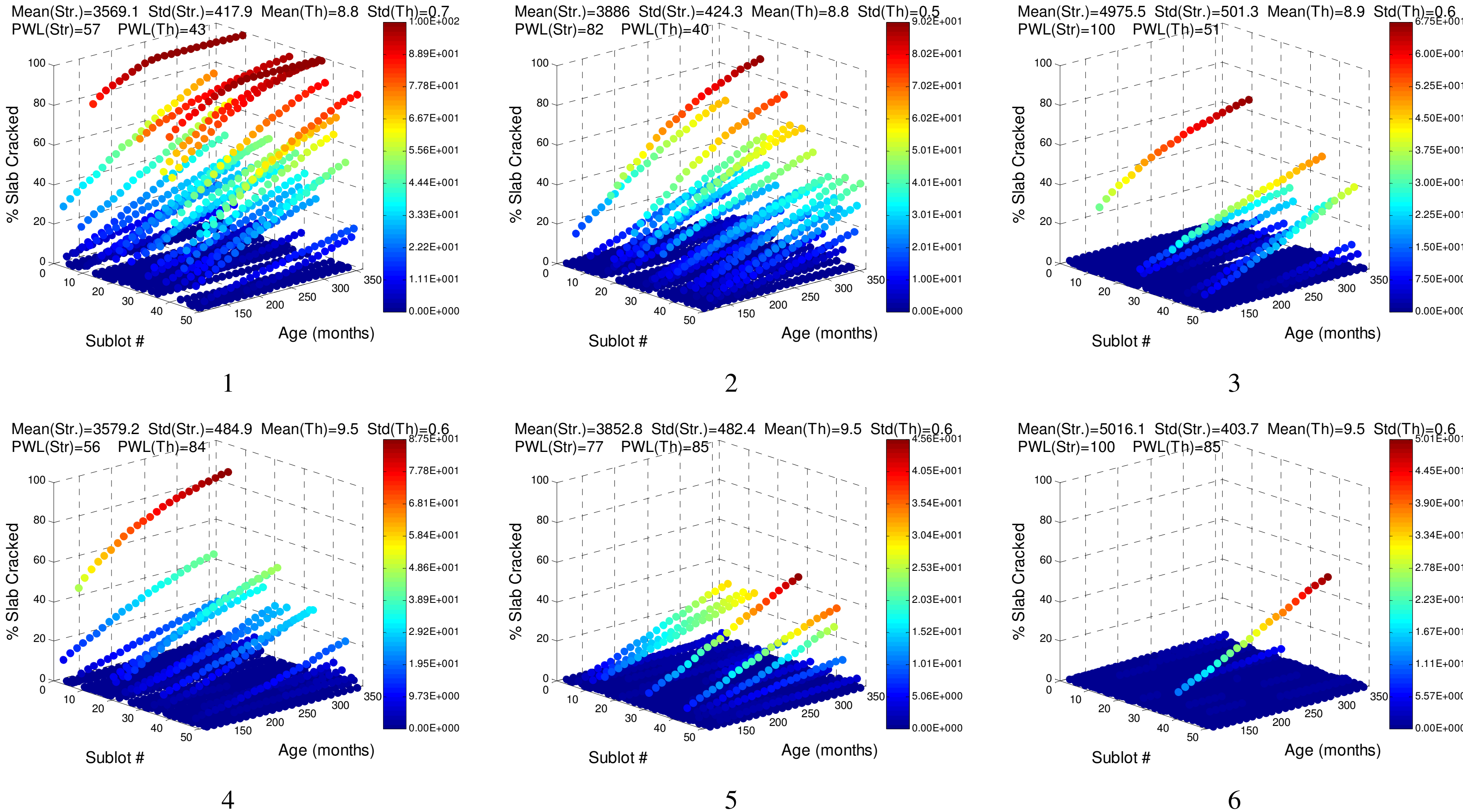


Figure 16. 1 Percent slab cracked for 50 sublots with varying compressive strength and slab thicknesses- Cases 1 through 6

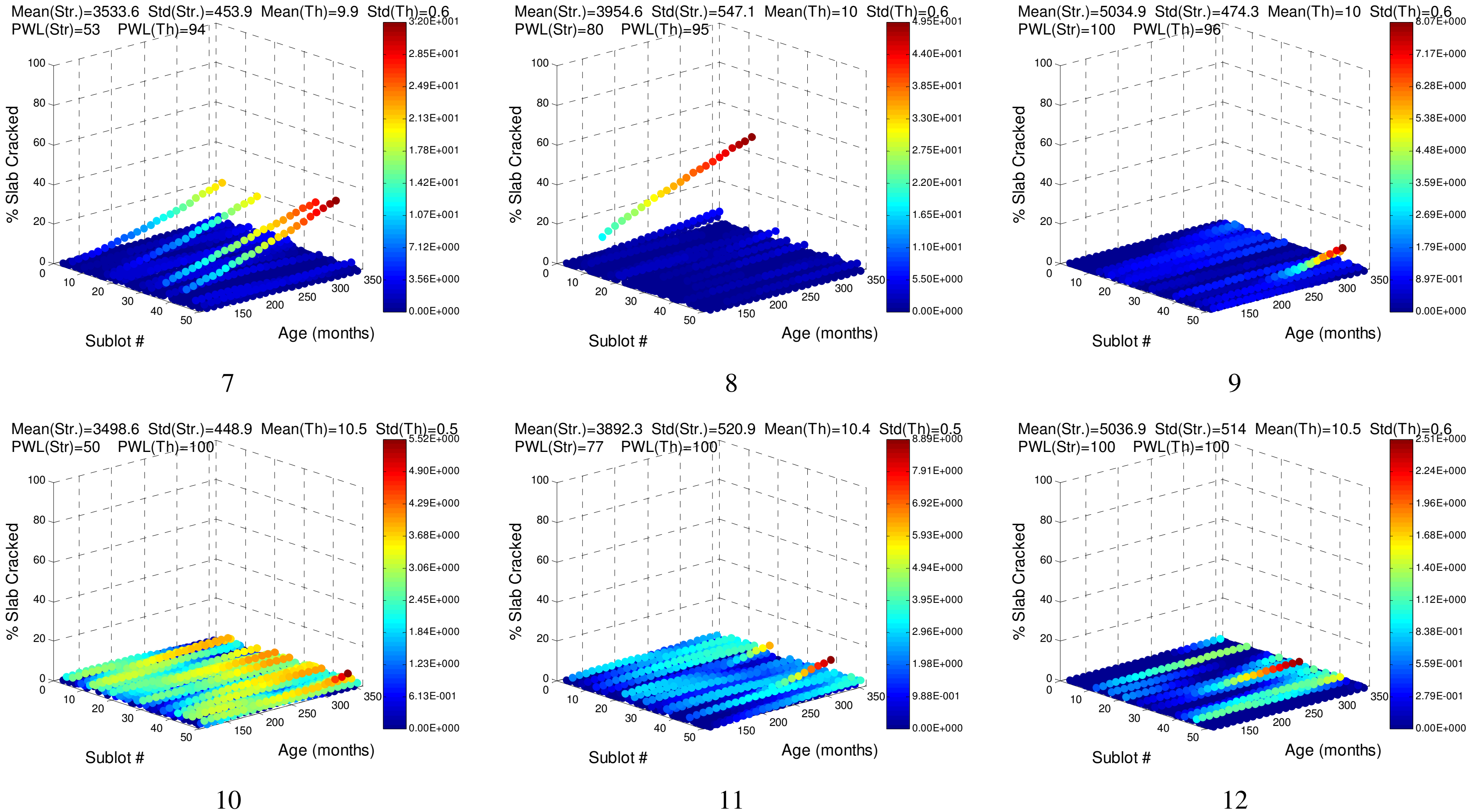


Figure 16. 2 Percent slab cracked for 50 sublots with varying compressive strength and slab thicknesses- Cases 7 through 12

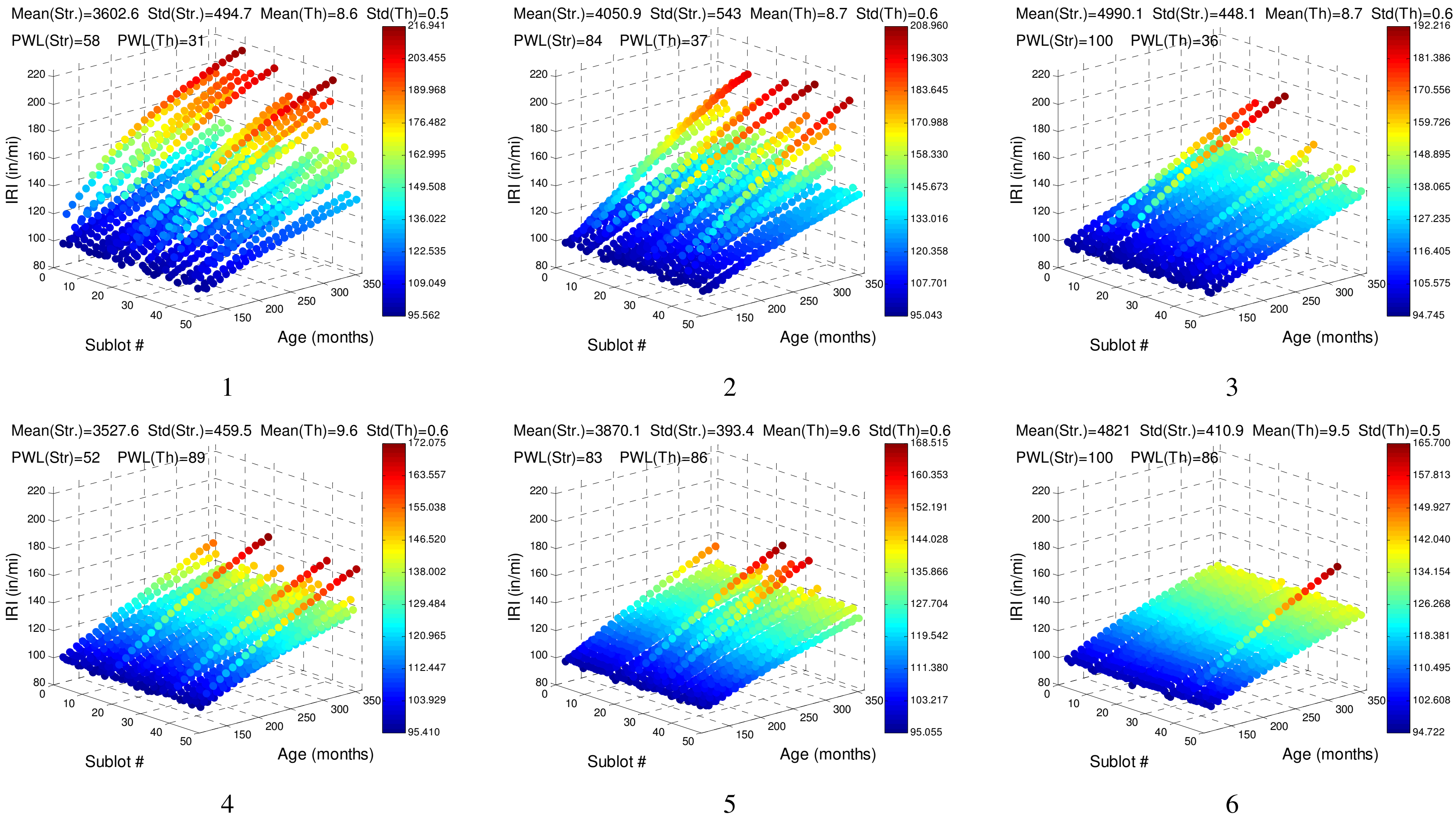


Figure 16. 3 IRI for 50 sublots with varying compressive strength and slab thicknesses- Cases 1 though 6

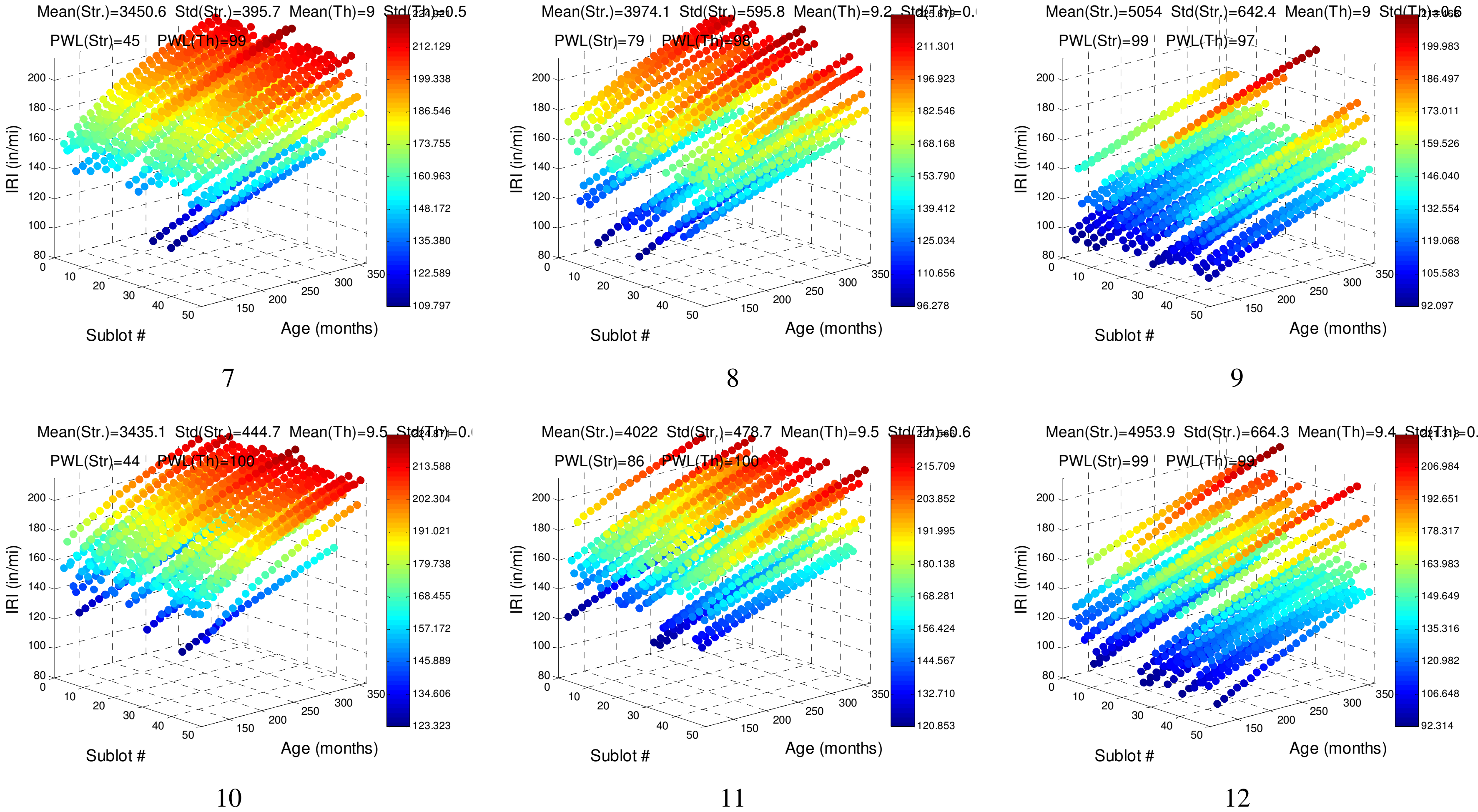


Figure 16. 4 IRI for 50 sublots with varying compressive strength and slab thicknesses- cases 7 through 12

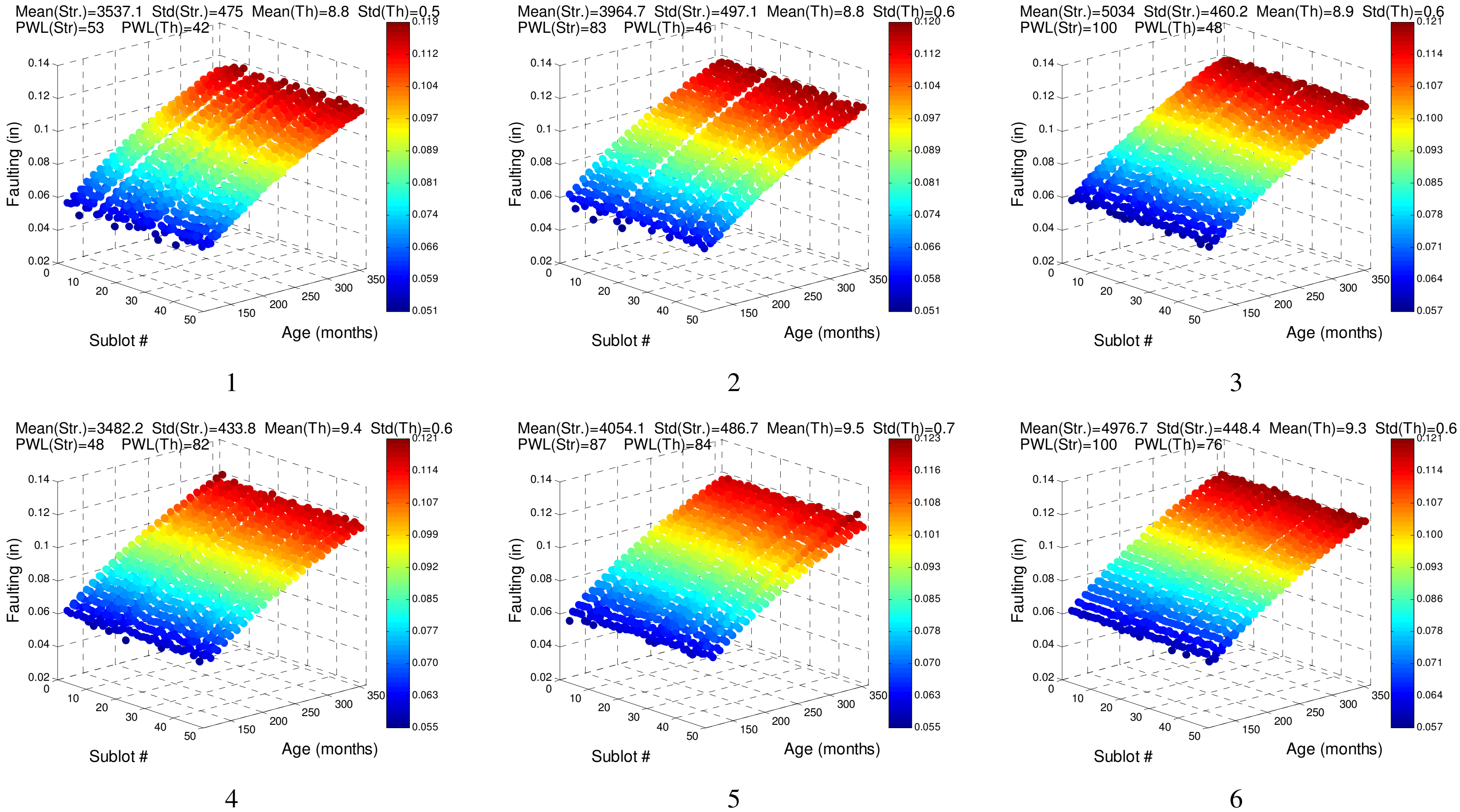


Figure 16. 5 Faulting for 50 sublots with varying compressive strength and slab thicknesses- Cases 1 through 6

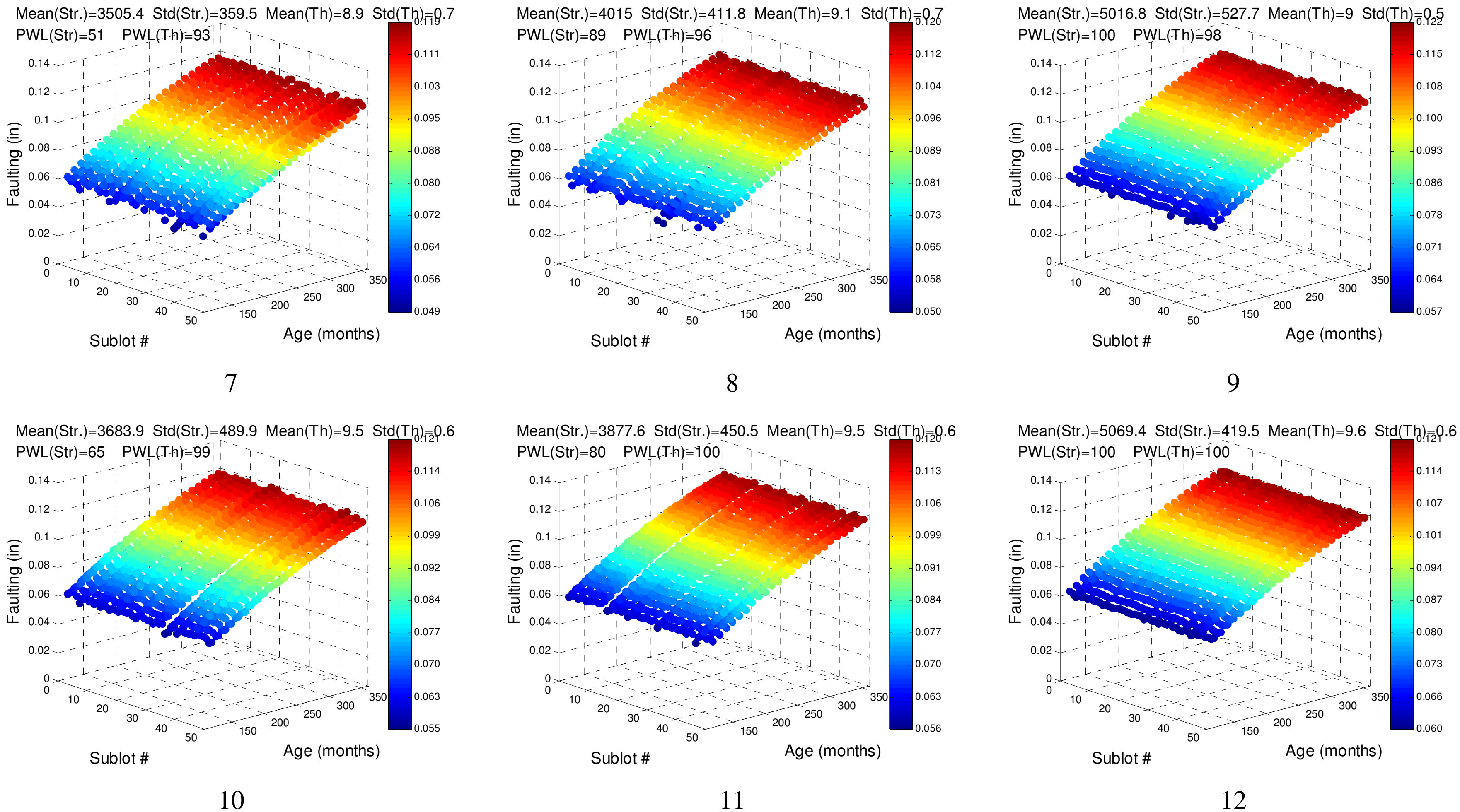


Figure 16. 6 Faulting for 50 sublots with varying compressive strength and slab thicknesses-cases 7 through 12

16.2 Analysis using HIPERPAV II

HIPERPAV II was developed as a tool for predicting early age behavior and its influence on long-term pavement performance for JPCP and CRCP pavements. It takes into account the effect of construction related variables. This section presents the results from analysis performed for three variables, namely (a) time of the day when concrete is poured, (b) month of construction and (c) temperature of the fresh concrete at the time of pouring.

It may be argued that month or time of construction are not QA variables and therefore, need not be studied in this project. However, the analysis presented here shows that they can influence pavement performance significantly. Therefore, suitable QA tests must be incorporated in the QA program to check for their effects.

In order to study the above mentioned variables a real MDOT project was used as the example case. It is a concrete pavement constructed in Novi, Michigan in November 1995 and is identified with the section ID 36003E. It is one of the five rigid pavement projects, details of which was provided by Mr. Michael Eacker to the MSU research team in the project entitled "Evaluation of the 1-37A Design Process for New and Rehabilitated JPCP and HMA Pavements". This pavement has a 12 inch thick slab above 4 inch crushed gravel base and a 10 inch sand subbase. HIPERPAV uses its internal environmental database to account for the effect of climate. Therefore, in this study climate similar to Novi, Michigan was used.

Figure 16.7 shows critical stresses and strength gain during the first 72 hours in concrete pavement when they are constructed at different times of the day. The larger monotonically rising curve represents the strength gain since the time of construction. The lower family of curves shows critical stresses in the pavement because of curing and environmental effects. The x-axis shows time of the day and not the time since construction. Also, the curve in the front represents construction at 1:00 pm followed by 2:00 pm behind it and so on for the entire day. If at any time the stress developed in the concrete slab becomes equal to or greater than its strength then the slab could develop premature cracking. The stress curve goes in cycles, each cycle spanning one day. Since strength during the first day is the smallest, when the stress curve for the first cycle gets closer to the strength gain curve, the possibility of getting premature cracking is higher.

From the plot, it is clear that, as noon time approaches, the critical stress curve becomes higher and gets closer to the strength gain curve. At noon, they are closest to each other. Even if these two don't cross each other, which means less likelihood of premature cracking, the stresses may get locked in the concrete slab and may lead to built-in curling. Built-in curling is known to lead to cracking within a few years of construction even if they are well-designed and properly constructed concrete pavements. Therefore, taking into account the time of construction and the resultant stress development in the slab is important in attempts to reduce the possibility of built-in curling.

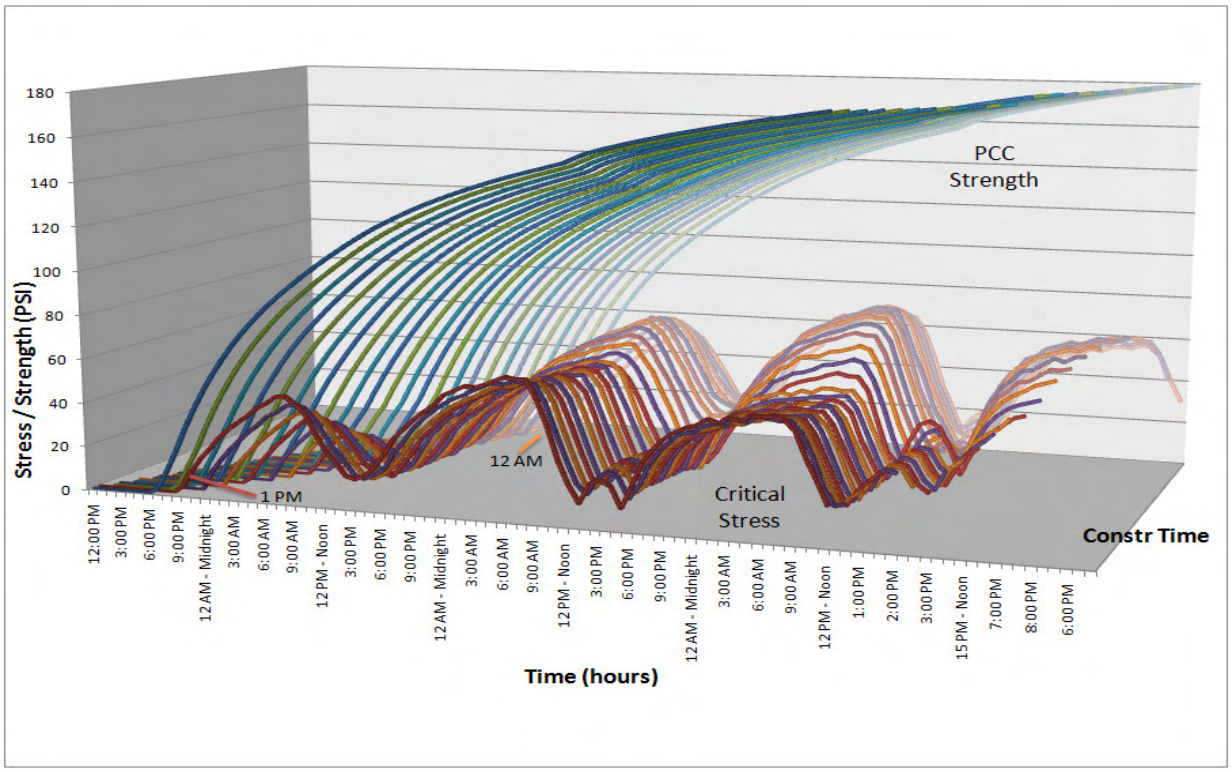


Figure 16. 7 Difference in critical stresses and strength gain in concrete pavement for construction at different times of the day

Figure 16.8 shows the critical stresses and strength gain curves for construction done at different times of the year. Because of the very different shapes of the critical stress curves for different months, they have been plotted with solid areas rather than just line curves.

Figure 16.9 shows another view of the same plot to show the curves which are not visible in Figure 16.8. From the point of view of premature cracking and built-in curling, it appears that colder months are better than hotter months. In the case analyzed here, the month of May was found to have the maximum possibility of premature cracking and built-in curling (based on visual inspection of the curves). All the curves presented here correspond to noon as the time of construction because this was found to be the most critical time from the stress development point of view.

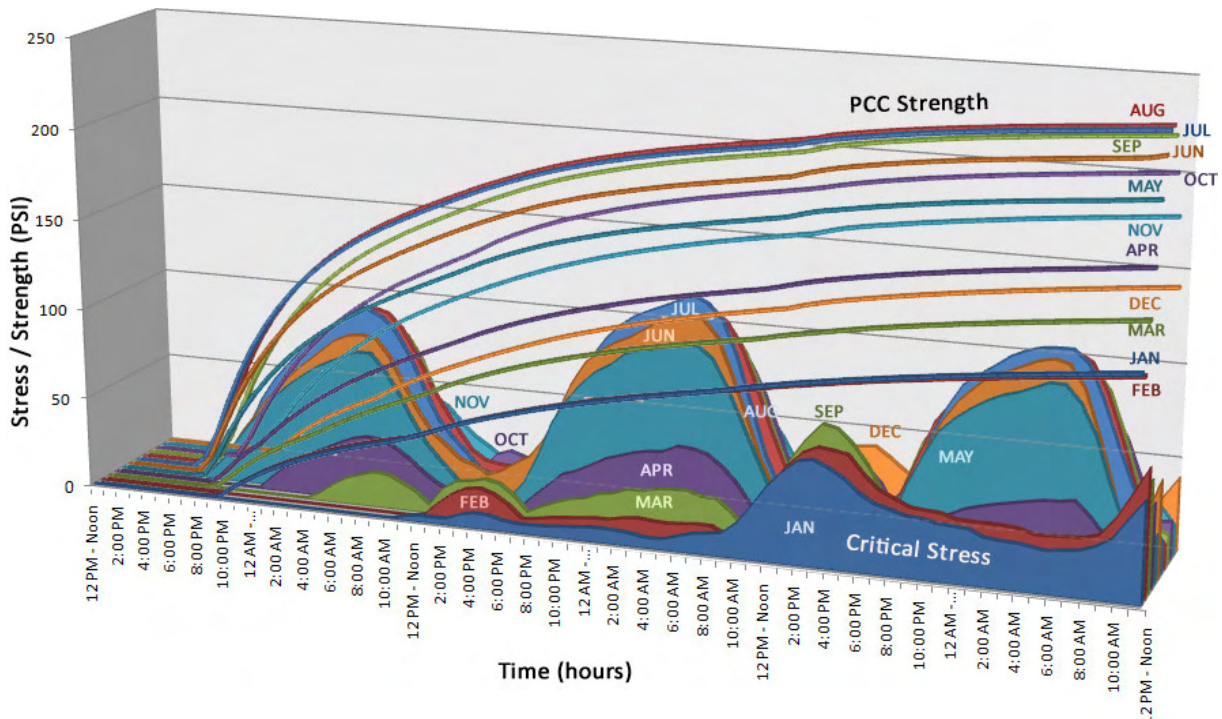


Figure 16. 8 Difference in critical stresses and strength gain in concrete pavement for construction at different times of the year

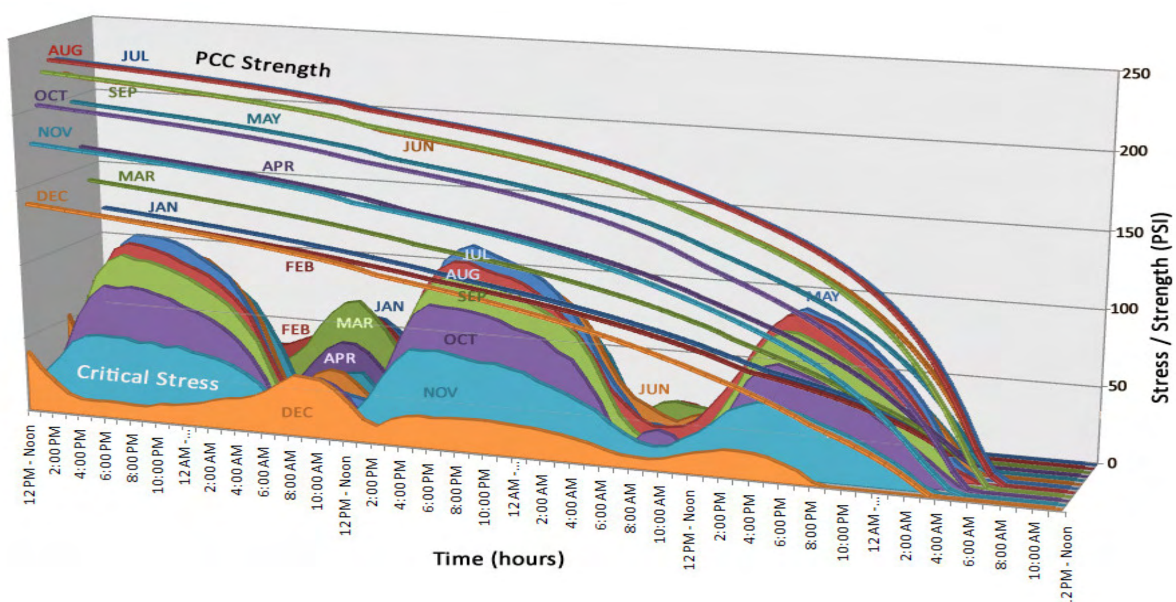


Figure 16. 9 Difference in critical stresses and strength gain in concrete pavement for construction at different times of the year

The third variable studied using HYPERPAV was temperature of fresh concrete at the time of construction. Figure 16.10 shows the critical stresses and strength gain corresponding to the different temperatures of fresh concrete at the time of construction. To study the effect of this variable, all other variables were kept constant and the most critical time of construction (noon) and the most critical month for construction (May) was used. All curves for all the months show that the stress and strength curves are very close to each other. But in the case of higher concrete temperatures, significantly higher stresses are developed along with relatively quick gain in strength. Quick gain in strength means that the stresses developed in the slab would not be relaxed and would accumulate over time, thus possibly leading to built-in curling. Therefore, temperature of concrete is an important variable which can potentially influence pavement performance.

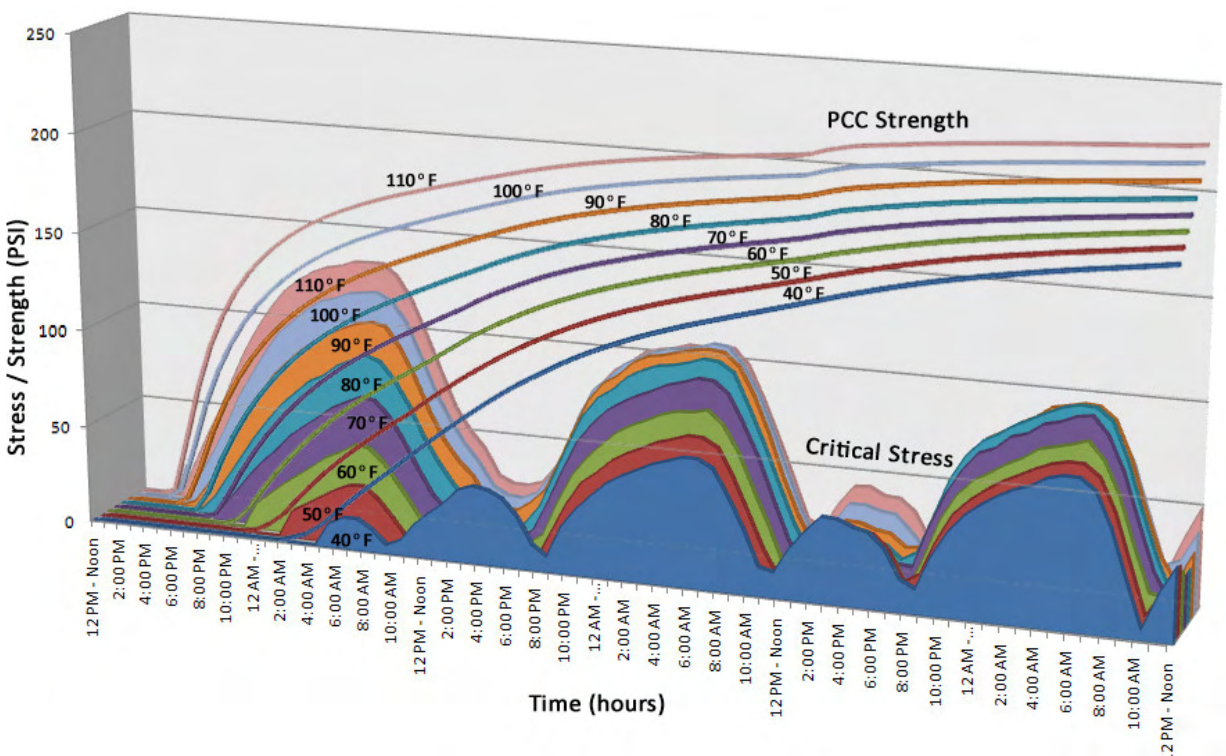


Figure 16. 10 Difference in critical stresses and strength gain in concrete pavement for different temperature of concrete at the time of construction.

16.3 SUMMARY AND CONCLUSIONS

The MEPDG software accepts inputs mainly corresponding to design of the pavement, e.g. amount of cementious material, water to cement ratio etc., and fewer inputs with respect to construction, like temperature of fresh concrete before pouring, time of the day when the concrete was poured etc. However, two of the expectedly most significant variables, namely slab thickness and 28-day compressive strength of concrete can be studied using MEPDG. The analysis showed that when the PWL values are lower for both strength and thickness, percent cracking is high. The results also show that the effect of deviations from the target compressive strength and slab thickness is drastic. The analysis for IRI shows similar trend as slab cracking. It was also observed that faulting does not seem to be affected by strength and thickness levels.

The HYPERPAVII analysis showed that all the three factors analyzed can have significant influence on pavement performance. However, the first two factors, namely time of the day and time of the year of construction are not QA variables. They can possibly be used to provide guidelines to the contractor for better construction. Similarly, the third factor, i.e. temperature of fresh concrete, is not a “performance-related” or even “end-result” variable. But the effect of these factors should be checked. Built-in curling could be checked after 24 to 48 hours of construction using either a dip-stick or falling weight deflectometer. Premature cracking because of these factors would also appear within first few days of construction. This is checked by the state department of transportation as part of the QA program.

CHAPTER 17: ERS Risk Analysis for Rigid Pavements Using Simulation

17.1 INTRODUCTION AND BACKGROUND

Over the years many highway agencies in North America have made a commitment to End Results Specifications (ERS). As a direct result, it is believed that the quality of our roadways has improved (Smith, 1998, Benson, 1999). A quality assurance (QA) program, which involves material testing, plays an important role in measuring this quality and is an integral ERS component. The results from the material testing are used to determine payment to be made to the contractor.

Aurilio et al. (2002) considered effect of differences between laboratory test results on payment for the contractor. Further analysis of actual ERS project data indicated that in addition to test bias several other factors, like measurement variability, production variability etc. can have significant effect on payment. It has also been demonstrated that the concept of simulation program can be used to take into account all the parameters that could be identified to be affecting payment calculation (Aurilio et al. 2002, Manik and Buttlar, 2006). This chapter reports on the development of a Monte-Carlo based simulation program for assessing the Michigan Department of Transportation's QA program and estimate the errors or risk involved with payment made to the contractor according to the provisions in the QA program. The chapter presents the details of the program, analysis and some of the conclusions that can be derived from those. Such study would provide valuable insight into how QA programs can be formulated to reduce risk to the contractor as well as the agency and also balance risk.

17.2 END-RESULT SPECIFICATIONS

End-result specifications place full responsibility of producing a pavement of a certain specified quality on the contractor. The contractor has full freedom to choose methodologies for construction process and take strategic decisions. He conducts quality control tests at a specified frequency to monitor the quality of the pavement being constructed. The responsibility of the state highway authority (SHA) is to check from their own side that the quality is acceptable, through quality assurance tests (AASHTO, 1996). The SHA can decide, based on criterion laid out in the specification, whether the quality is acceptable or rejectable, or that the pavement be accepted but with penalty to the contractor in terms of reduced pay. Adjustment in pay is one of the most significant aspects of ERS in present day practices. Rather than setting pass and fail criteria, a percentage of the material produced is judged to be within acceptable limits and payment is determined accordingly. This calls for use of statistical methods (Box and Wilson, 1951).

The quality characteristics (defined as the characteristic of a unit or product that is actually measured to determine conformance with a given requirement) that are being used to determine "quality" of the pavement are generally air content, slab thikness, slump, cylinder strength, gradation, etc. These quality characteristics are believed to be related to performance but the exact relationships are not yet firmly quantitatively established for all of them. Therefore, the pay adjustments are based on the values of the quality characteristics themselves and not on expected performance of the constructed pavement (Smith, 1998).

17.3 ESTIMATING RISK

In the past, researchers have attempted to develop statistical or simulation tools to help understand and balance risks in construction specifications. A computer simulation program called OCPLOT, developed in FHWA Demonstration Project 89 by Weed (Weed, 1996) is available for generating Operating Characteristics (OC) curves. OCPLOT was found to be user-friendly and very useful for initial assessment of relative risks, allowing the user to vary the following factors: sample size, pay factor equation, specification limits, and retest provisions. The program allows the user to assess the probability of acceptable material being rejected (defined as contractor risk) and the probability of rejectable quality material being accepted (defined as agency risk) over the long run (e.g., when considering the characteristics of the specification a long period of time). However, a number of the factors that appear to be related to risk, including measurement variability and testing bias are not considered in OCPLOT. In addition, it can be argued that the most tangible measurement of risk should be linked to the financial impact on the project, i.e., how *risk* affects *what is actually paid versus what should have been paid*.

One of the necessary steps in the assessment of payment risk is to clearly define the risk metric. A very straight-forward and yet very effective way of defining risk could be as shown in equation 1, where baseline pay represents the ideal or correct payment.

$$\text{Payment Risk} = \text{Payment made to the contractor} - \text{Base Line pay} \tag{1}$$

Ideally, tests performed by different parties on the same material should give very similar results. However, in practice even split samples will show different results when the tests are carried out by two different agencies or in two different labs. Because of these uncertainties there is a risk of accepting rejectable quality and vice-versa. In the ERS approach, a percentage of acceptable quality (Percent Within Limits-PWL) is determined rather than pass/fail criteria used in typical QC/QA. Then, payment is made based on this percent within limits value (Patel, 1996). Because of the uncertainties involved with the test results the payment made also may be more or less than what it would be if the actual quality of the construction would have been exactly determined (Weed, 1996; Willenbrock, 1976; Bowery and Hudson, 1976; Barros et al. 1983; Puangchit et al., 1983; Afferton et al., 1992; AASHTO, 1995). Overpayment of the contractor is often referred to as 'agency risk' while underpayment is often termed as 'contractor risk'. Throughout this report, *positive values of risk refer to the instance where the agency paid more than required (agency risk) and negative values of risk indicate that the agency paid less than what the contractor deserved (contractor risk).*

Buttlar and his coworkers at the University of Illinois at Urbana-Champaign have developed a series of risk simulation models that provide the user a virtual environment to quickly generate and analyze thousands of realistic ERS data sets. The first simulation model developed was ILLISIM (Buttlar and Hausman, 2000). This was followed by PaySim and BiasSim (Aurillio et al. 2002) which used different models and catered to different aspects of risk analysis and simulation. The latest model developed for the Illinois Department of Transportation is called Simulate Risk Analysis, or SRA, which combines the capabilities of all earlier programs into a single program, with added features to simplify the process of conducting sensitivity analyses (Buttlar and Manik, 2007). Using the same principles, a new simulation model called AMSim has been developed to analyze MDOT QA program by the authors. This chapter presents the details of this simulation model along with the analysis performed and conclusions derived from the analysis.

17.4 MAJOR FACTORS AFFECTING RISK

The data analysis of actual construction projects corresponding to various quality characteristics performed by Hall and Williams (2002) showed that such data are generally normally distributed. Therefore, it was assumed that the quality characteristics data for concrete project are normally distributed. Generally, the target values for these quality characteristics are fixed. This indicates that, as it would be expected in the real world, there are certain factors involved in the construction and quality characteristic measurement procedures which are not completely controllable, or even predictable. They tend to induce variability (Benson, 1995) in the quality around the targeted quality level.

Variability observed in the field, however, has at least two components, namely production variability and measurement variability. Production variability includes all variability introduced due to workability of concrete, variability in the quality and physical characteristics of source materials, changes in the relative proportions of ingredients in the mix, changes in plant operational characteristics, changes in equipment operators, changes in ambient temperature etc. Measurement variability is the variability which is introduced by the measuring devices, test procedures, and operator techniques and human error. In addition to variability around the actual value, a measurement bias may be introduced as well. Bias refers to a consistent shift in data and can be introduced by device calibration errors, human error, or by the intentional biasing of measurements and/or recorded data (Aurilio et al. 2002).

Every choice made in the development of an ERS comes with an associated risk. Risks are undertaken by both the contractor and the agency. The introduction or manipulation of certain specification attributes can shift the risk from the contractor to the agency and vice-versa. Other specification attributes can widen or narrow the range of risk. In summary, the key contributors to risk in ERS are:

- Contractor data versus agency data
- Frequency of testing and/or number of samples
- Variability and/or bias of test device and/or test procedure
- Specification parameters, including:
    - Specification limits
    - Pay factor equation

- Pay “caps”
- Acceptance test logic and frequency and acceptance tolerance
- Third party testing provisions

In the procedure used for determining pay factor in ERS a sample of data with finite measurements is used to estimate the quality of a population which this sample belongs to. Therefore, mean and standard deviation of any quality characteristic of the sample is considered as equal to the mean and standard deviation of the entire material in the lot or pavement produced in that project. However, the finite sample being used may not have exactly the same mean and standard deviation as it could have been if a much larger number of samples were collected. Theoretically, actual quality of the material can be determined only if the sample collected is infinite. Such an infinite size sample, or rather population, would give payment called as “ideal payment”. Therefore, finite size sample would lead to a deviation from the ideal pay. In addition, the use of imperfect measuring devices would also lead to error in measurements. The error in turn would lead to deviation from the ideal Pay. Therefore, to be able to determine ideal pay, thousands of data with similar characteristics would need to be simulated, where each simulation represents an actual individual project. Pay calculated for each individual project coupled with the ideal or base-line pay for the entire population would provide distribution of risk on a project with those characteristics.

In order to simulate variability in concrete pavements material properties, one must be able to sequentially simulate, in this order: 1) production or construction variability; 2) results of random samples taken from that variable material; 3) the effects of measurement variability on the estimated properties; and finally; 4) the effects of bias on the final reported test measurement values. In order to estimate risk in terms of effects on pay, the software must also simulate the formulas and decision tree logic contained in the construction specification.

17.5 COMPOSITE RISK INDEX

A simulation tool like AMSim or SRA helps estimate and analyze risk in payment that can be expected in different scenarios using a certain set of end-result specifications. The main advantage of such a tool is that it can provide invaluable information in what-if scenarios without the need of a demonstration project or shadow specification. This can greatly help in determining the effect of different aspects or values in the specification used in end-result projects.

The main format in which AMSim would provide information would be risk plots. A risk plot presents the expected mean risk and associated confidence interval for the entire range of quality characteristic possible on a project. This means that a risk plot can give a very good understanding of how "well" a set of specifications would do for that quality characteristic.

A wealth of information can be gained from the risk plots generated by SRA. However, the interpretation of the risk plot could be subjective. This may make it difficult to compare risk scenarios arising because of two different specifications or any combination of other parameters affecting risk. In addition to this, if an algorithm needs to be developed for comparing risk plots for the purpose of comparing specifications etc. various quantitative characteristics of the plot would have to be used. Manik (2006) developed a composite risk index (CRI) to quantitatively characterize the risk plots. The concept of CRI was tested on a wide range of risk plots and was found to be very objective and promising in its purpose. The analysis presented in this chapter also uses CRI.

17.6 RISK ANALYSIS

One of the earlier sections in this chapter identified several factors associated with a QA program that affect risk involved in payment made to the contractor through that program. It is very important to assess how exactly these factors affect payment risk. In addition to that, it is also important to determine how they influence contribution of other factors to payment risk. This section presents risk analysis performed for MDOT QA program with the following four factors in focus.

(1) Production Variability
(2) Measurement Variability
(3) Sample size and
(4) Bias

Three levels were identified for each of these four quality characteristics and a full factorial run matrix was constructed as shown in Table 17.1.

AMSim simulates the entire MDOT QA program including the specification limits, sampling scheme and decision logic ending with pay factor calculations. MDOT QA program has a separate set of specifications for concrete pavement thickness and strength. In this chapter, thickness specifications were used for analysis. Table 17.2 shows the pay schedule used by MDOT for pay factor calculation in thickness QA specifications. In the past, several states followed the practice of using similar pay schedules. However, with the increasing use of statistical methods in ERS pay schedules have replaced by percent-within-limits concept and pay formula (Buttlar and Harrell, 1998).

Table 17. 1 Run matrix for Risk Analysis of Rigid Pavements

Sample Size	Bias	Prod Var** = 0.1			Prod Var = 0.3			Prod Var = 0.5		
		Meas var*=0.1	Meas var=0.3	Meas var=0.5	Meas var=0.1	Meas var=0.3	Meas var=0.5	Meas var=0.1	Meas var=0.3	Meas var=0.5
10	0.1	1	10	19	28	37	46	55	64	73
	0.3	2	11	20	29	38	47	56	65	74
	0.5	3	12	21	30	39	48	57	66	75
40	0.1	4	13	22	31	40	49	58	67	76
	0.3	5	14	23	32	41	50	59	68	77
	0.5	6	15	24	33	42	51	60	69	78
70	0.1	7	16	25	34	43	52	61	70	79
	0.3	8	17	26	35	44	53	62	71	80
	0.5	9	18	27	36	45	54	63	72	81

* Measurement Variability (in inches)

** Production Variability (in inches)

Table 17. 2 Price Adjustment for Concrete Thickness Deficiency

Initial Core Type	Deficiency in Thickness (Inch)	Price Adjustment (Percent)
A	0.20 or Less	0
B	0.30	-5.0
B	0.40	-15.0
B	0.50	-25.0
B	0.60 To 1.0	-50.0
C	1.10 and Over	-100 [a]

AMSim was run for all the 81 cases identified in the run matrix (Table 17.1). In this case the target thickness of the PCC slab was 9 inch. The maximum deficiency of 1.1 inch in thickness is allowed beyond which the contractor must abide by "remove and replace" specifications at no cost to the state/highway agency.

Figures 17.1 through 17.9 show sample results from the 81 runs performed with AMSim. It would be important to describe the concept of risk plot first. The x-axis in the risk plot has mean of the quality characteristic in the QA program which is being analyzed. The three risk curves shown in each of the figures correspond to the mean and upper and lower 90% confidence interval. Any point on the mean risk plot shows the magnitude of payment risk with 50% likelihood if the mean of the quality characteristic achieved in a specific project is equal to the corresponding slab thickness value. Depending on the value of other parameters such as production and measurement variability, and number of samples the shape of the risk plot can change considerably. This change can be easily observed visually. The maximum risk is represented by the peaks in the plot and gives the maximum amount of risk for the given set of parameter values within specification limits. Positive risk represents the risk for the agency (overpayment) while negative risk represents the risk for the contractor (underpayment).

Figures 17.1 through 17.4 were carefully selected to demonstrate some of the salient conclusions that can be derived through this analysis as listed below. This will be followed by analysis of variance and corresponding conclusions for the entire run matrix.

(1) The analysis presented in this chapter shows the effect of using pay schedule instead of pay formula. Almost all the plots in Figures 17.1 through 17.4 show waviness in the risk plot curves, both the mean curve and confidence limit curves. The waviness in the curve indicates that the agency or the contractor may be at higher risk of loosing money even in instances when the contractor is producing closer to the target compared with the case if he had been producing slightly farther from the target. It happens because, in a pay schedule scenario, the payment to the contractor does not change until the next step in mean quality characteristic is reached. Secondly, the sudden change in pay is in steps, for example 5% or 10%. Therefore, an error in the measured quality characteristic value putting it on one side of the step would lead to a substantially different pay factor compared to that on the other side of the step for the same level of quality achieved. This is an undesirable feature of a QA program.

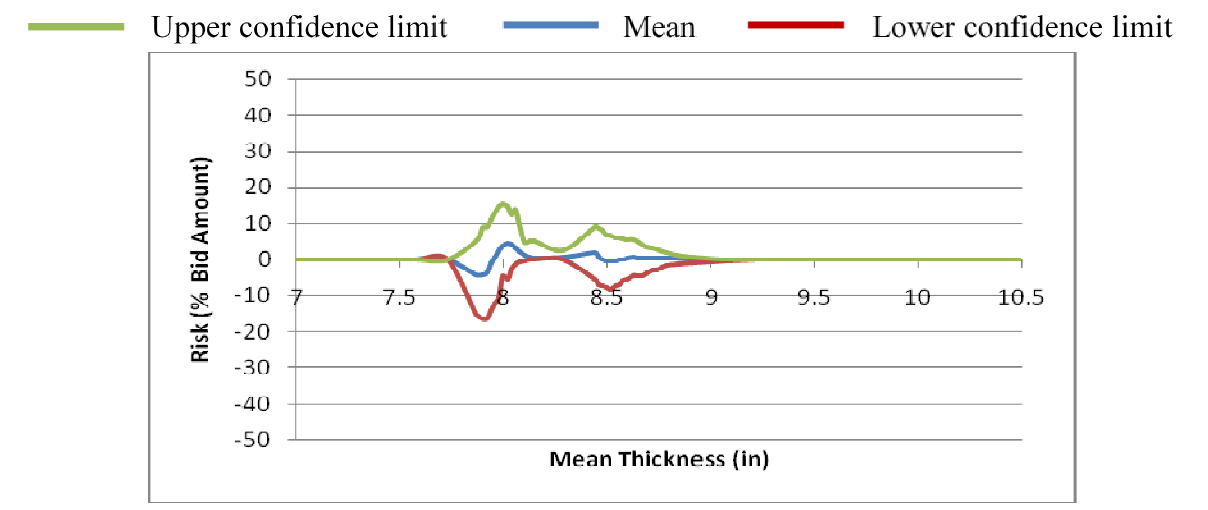
(2) Figure 17.1 shows the effect of production variability on risk. Plots a, b and c correspond to low, medium and high production variability with all other factors being the same. It is interesting to note that as the production variability increases from 0.1 to 0.3 inch the increase in payment risk is very sharp. Also, for low production variability, the agency and the contractor share risk. However, for medium

and high production variability the risk is almost always for the agency i.e. the agency is expected to overpay the contractor. With further increase in production variability, the risk seems to go down. Although it may seem counterintuitive, it can be explained by the following reasoning: When the production variability is high even in the case of mean quality characteristic being close to the target, several individual samples would have quality characteristic values much farther away from the target and probably falling outside the specification window. Once the value is well outside the allowable window, an error in measurement does not really change the payment to be made to the contractor according to the current QA program and hence lowers the risk values.

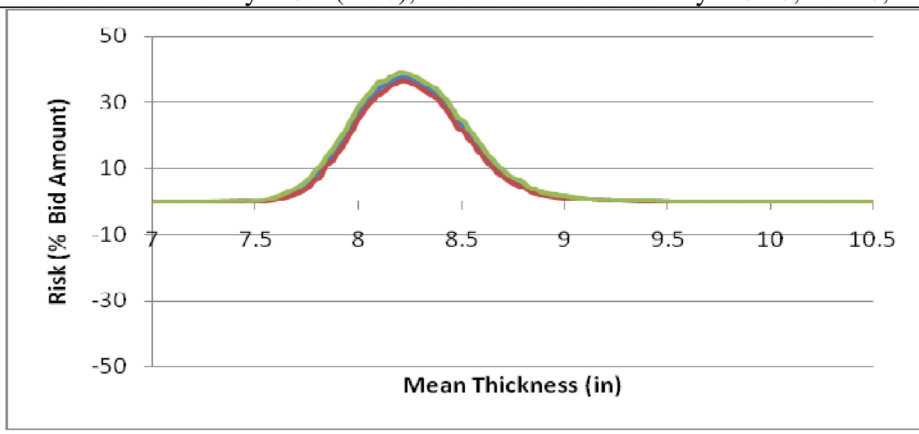
(3) The plots in Figure 17.2 show the effect of measurement variability on payment risk. An increase in measurement variability leads to a substantial increase in payment risk (compare plot (a) to plots (b) and (c)). The increase in risk across the full range of thickness is more than that from production variability. Therefore, measurement variability is a very important factor that needs to be controlled to lower payment risk in MDOT QA program for PCC thickness.

(4) Sample size also has a significant influence on payment risk as is evident from the plots presented in Figure 17.3. In all the three cases measurement variability was 0.3 inch, which generally leads to very high risk. However, as the sample size becomes larger risk goes down considerably. In addition to the lowering of the risk, an increase in sample size also leads to redistribution and therefore balancing of risk between the agency and the contractor.

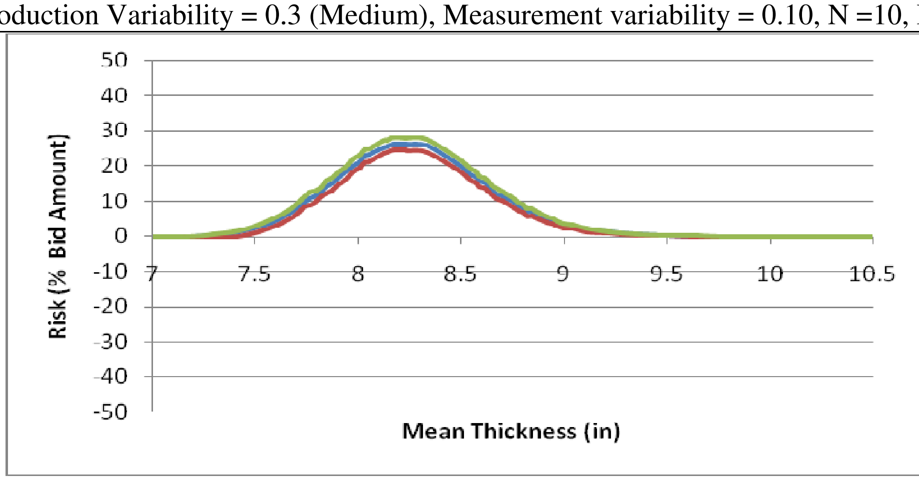
(5) Figure 17.4 shows the effect of measurement bias on risk. The first plot in Figure 17.4 corresponds to a bias of -0.2 inch, the second plot to no bias and the third to a positive bias of 0.2 inch. A bias of -0.2 inch means that the agency consistently measures thickness to be lower than what it would, even if measurement variability were present. In other words, mean of a large sample of thickness measurements would be lower than the actual value by roughly 0.2 inch. If there was no bias and only measurement error was present, the mean of such a large sample of thickness testing would be very close to the actual thickness. The three plots in Figure 17.4 clearly show that bias can not only affect the magnitude of the risk, but also it can completely change the sign of the risk as well. When the bias is -0.2 inch (negative) just left of the target most of the risk is expected to be born by the contractor. But, when bias is +0.2 inch (positive) in the same region, most of the risk is born by the agency. Therefore, bias must be controlled carefully and eliminated from measured value through proper testing and calibration of the test equipment/methods in the initial lot.



(a) Production Variability = 0.1 (Low), Measurement variability = 0.10, N =10, Bias=0 (2*)



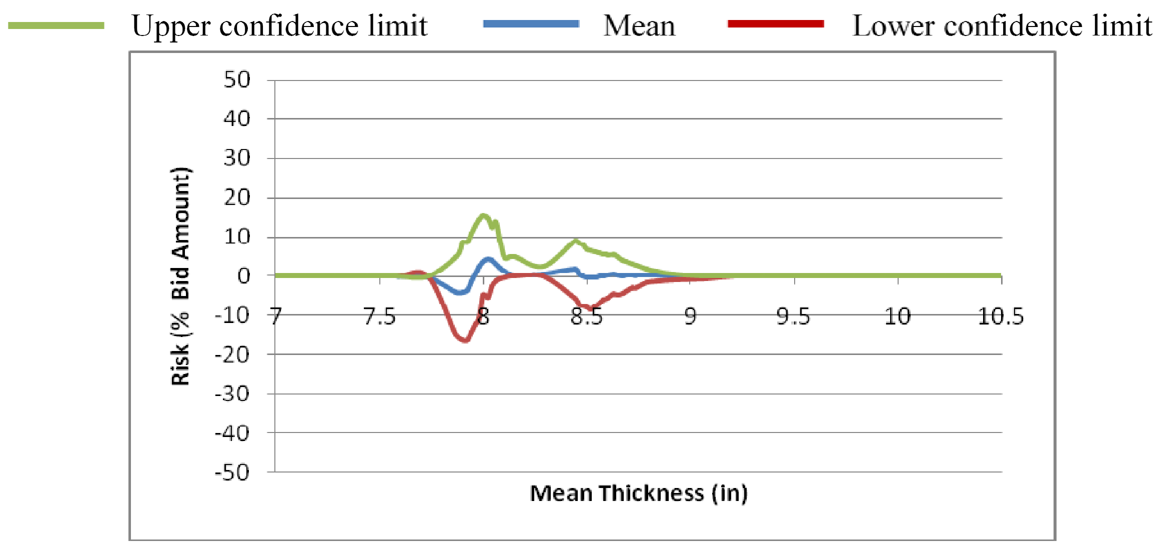
(b) Production Variability = 0.3 (Medium), Measurement variability = 0.10, N =10, Bias=0 (29)



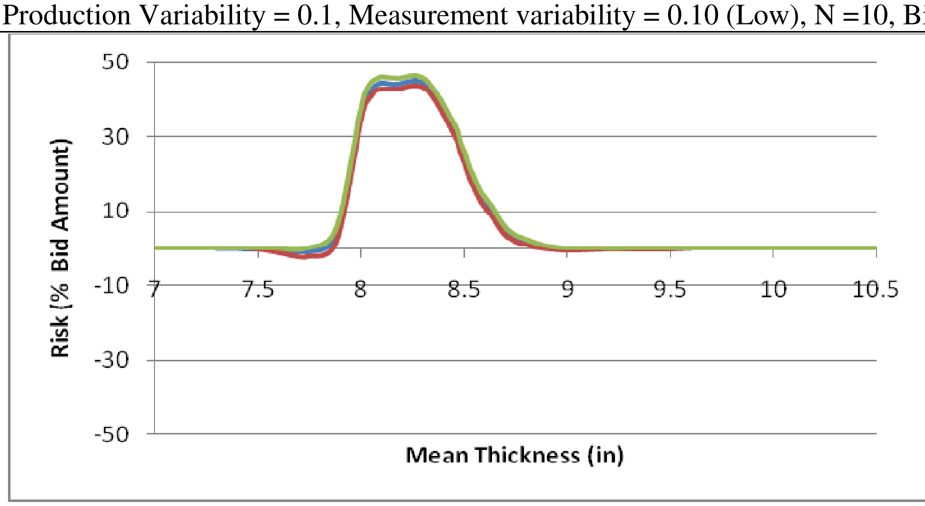
(c) Production Variability = 0.5 (High), Measurement variability = 0.10, N =10, Bias=0 (56)

Figure 17. 1 Effect of production variability on risk for rigid pavements

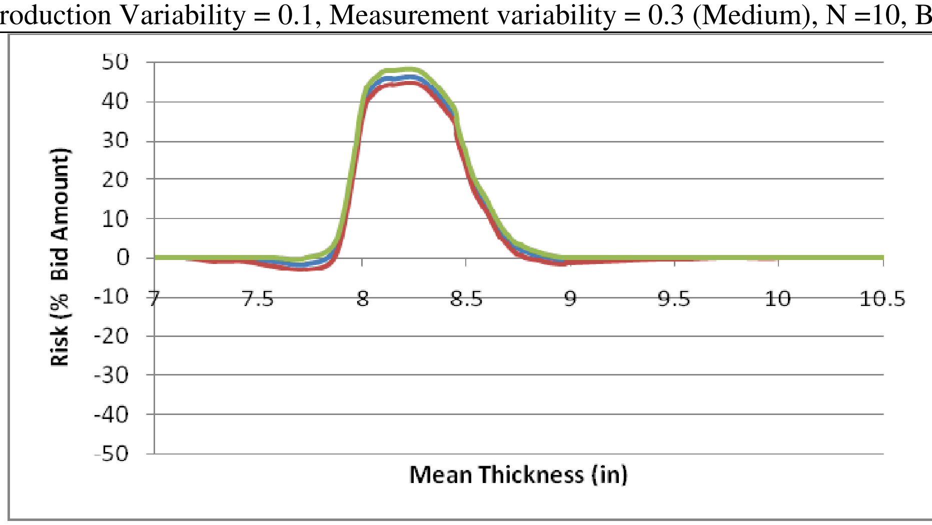
*Run number for the case in the run matrix (Table 17.1)



(a) Production Variability = 0.1, Measurement variability = 0.10 (Low), N =10, Bias=0 (2)

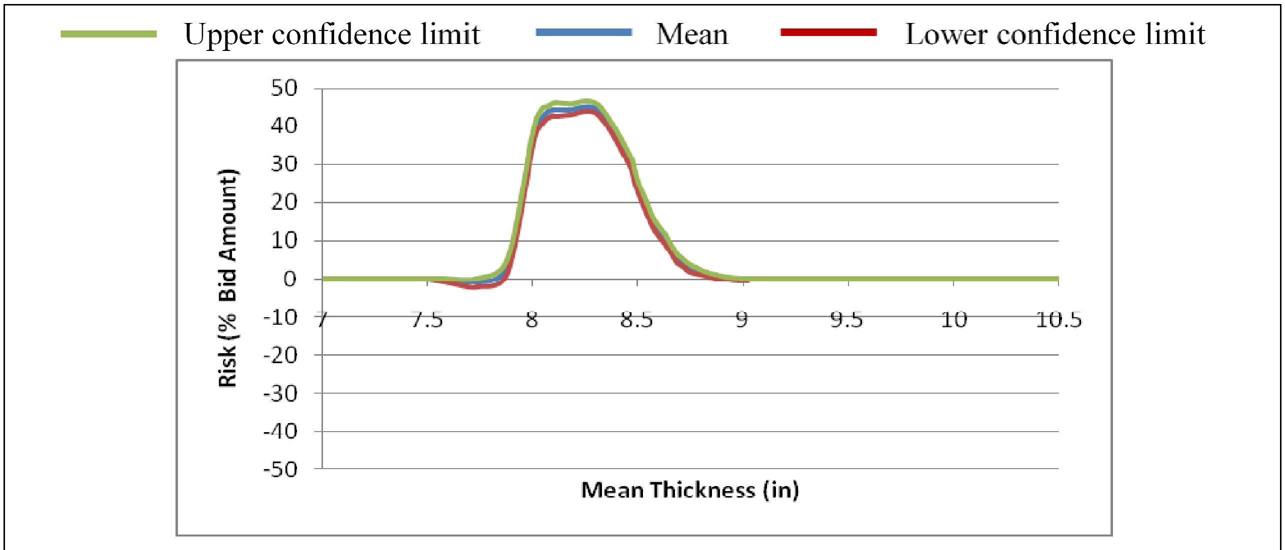


(b) Production Variability = 0.1, Measurement variability = 0.3 (Medium), N =10, Bias=0 (11)

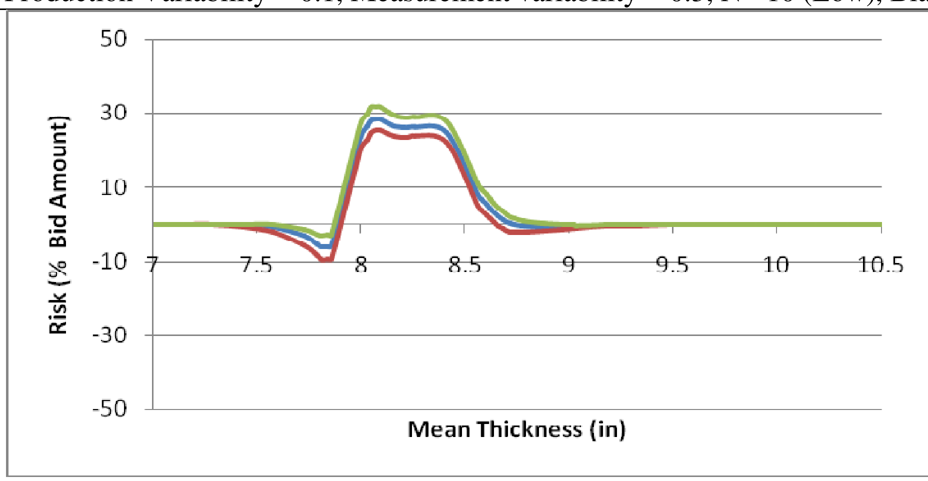


(c) Production Variability = 0.1, Measurement variability = 0.5 (High), N =10, Bias=0 (20)

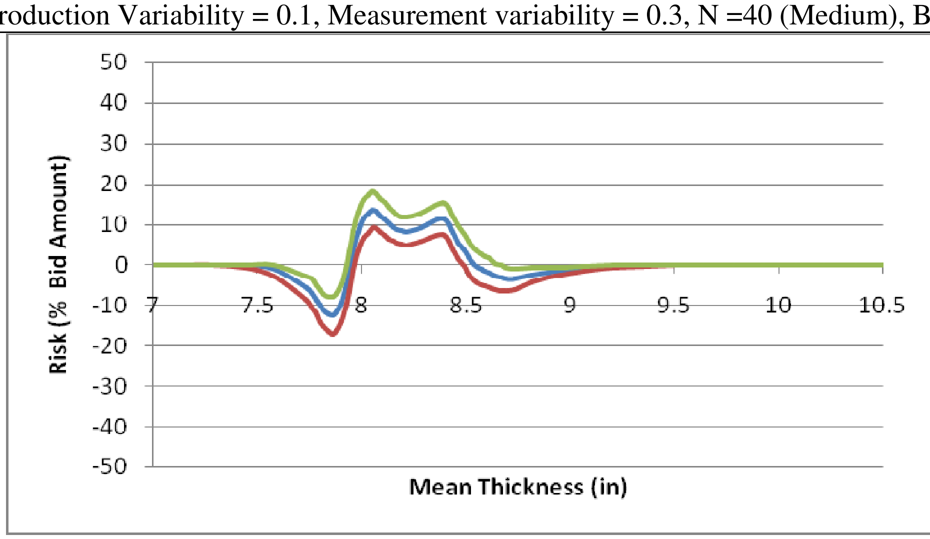
Figure 17. 2 Effect of measurement variability on risk for rigid pavements



(a) Production Variability = 0.1, Measurement variability = 0.3, N =10 (Low), Bias=0 (11)

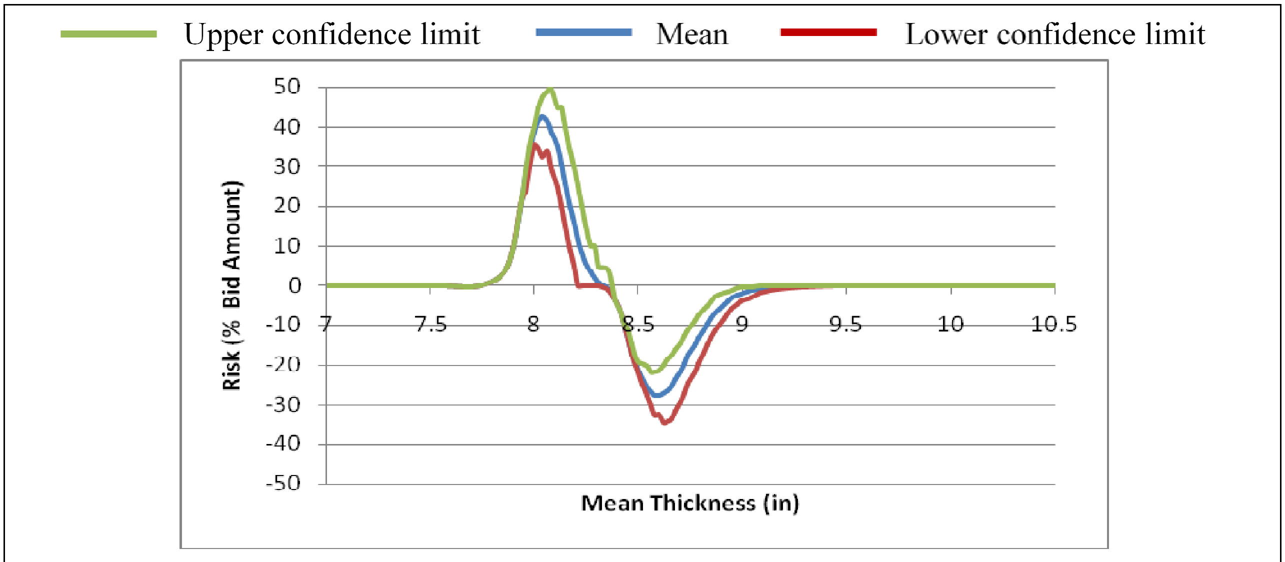


(b) Production Variability = 0.1, Measurement variability = 0.3, N =40 (Medium), Bias=0 (14)

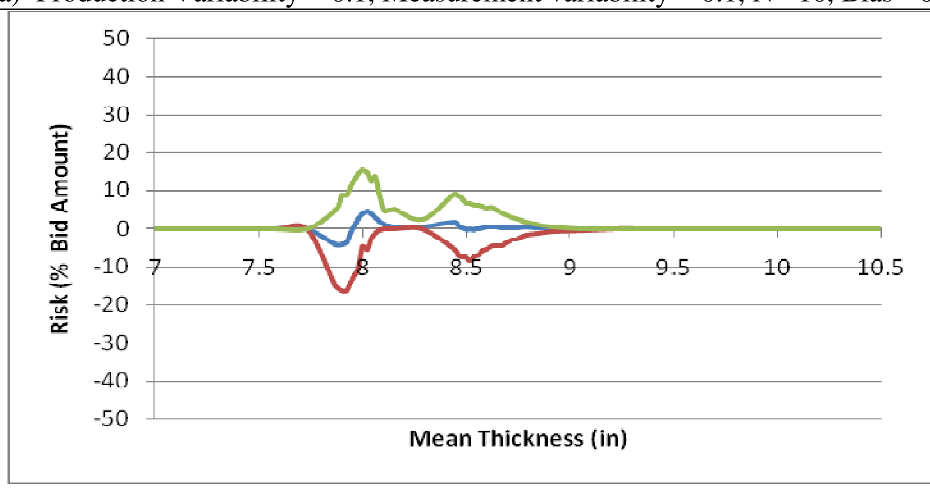


(c) Production Variability = 0.1, Measurement variability = 0.3, N =70 (High) , Bias=0 (17)

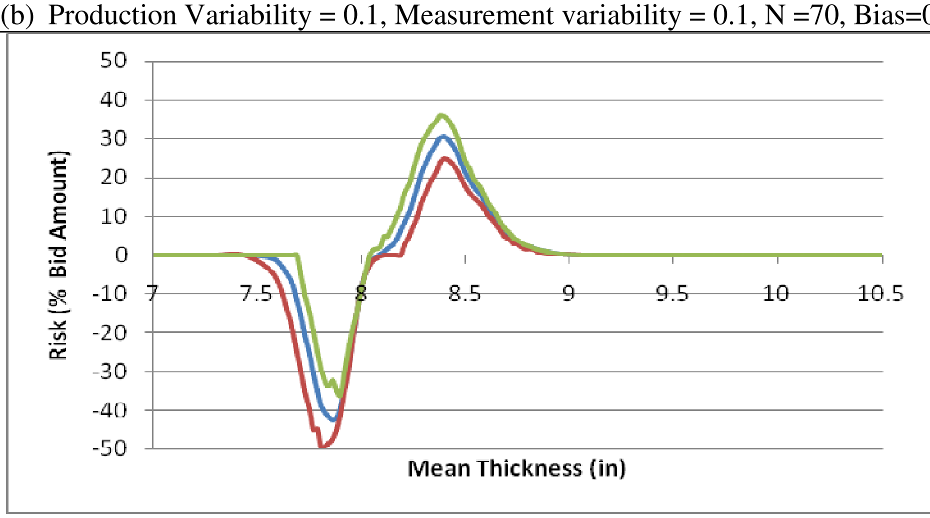
Figure 17. 3 Effect of sample size on risk for rigid pavements



(a) Production Variability = 0.1, Measurement variability = 0.1, N =10, Bias=-0.2 (1)



(b) Production Variability = 0.1, Measurement variability = 0.1, N =70, Bias=0 (2)



(c) Production Variability = 0.1, Measurement variability = 0.1, N =70, Bias=0.2 (3)

Figure 17. 4 Effect of measurement bias on risk for rigid pavements

17.7 Analysis of Variance (ANOVA)

The comparisons in the preceding section among different cases from the run matrix show the effect of the four variables, namely production variability, measurement variability, sample size and bias. Analysis of variance can not only help quantify the effect of these factors on payment risk but also it can give insight into the interaction effects of these factors. However, to be able to run ANOVA, the risk plots by themselves cannot be used. The concept of Composite Risk Index (CRI) was presented earlier in this chapter. CRI helps assign one index value to a risk plot representing one case scenario considering several factors simultaneously. Without the use of such an index, thorough statistical analysis with such scenarios would be nearly impossible.

Table 17.3 shows the values of CRI for all the 81 cases in the run matrix.

Table 17. 3 Calculated CRI Values for the Scenarios Identified in the Run Matrix.

Sample Size	Bias	Prod Var** = 0.1			Prod Var = 0.3			Prod Var = 0.5		
		Meas* var=0.1	Meas var=0.3	Meas var=0.5	Meas var=0.1	Meas var=0.3	Meas var=0.5	Meas var=0.1	Meas var=0.3	Meas var=0.5
10	-0.2	18.1	25.3	26.5	20.4	20.8	21.7	16.5	16.8	17.4
	0	1.8	25.1	26.3	20.8	21.3	21.8	17.2	17.3	17.5
	0.2	11.5	26.7	13.1	22.1	22.3	22.7	18.0	18.1	18.2
40	-0.2	17.9	16.8	18.6	10.7	12.3	15.0	7.4	8.5	10.1
	0	1.0	13.0	11.5	9.0	3.9	13.0	8.0	8.4	9.5
	0.2	11.4	17.3	12.3	13.7	8.3	14.8	11.0	11.1	11.1
70	-0.2	17.8	16.4	18.5	12.0	11.9	13.8	7.4	7.8	9.1
	0	0.9	4.9	11.0	1.4	3.6	8.0	1.8	2.7	4.9
	0.2	11.4	10.9	12.2	9.1	8.3	8.7	6.2	5.8	5.6

* Measurement Variability, in inches

** Production Variability, in inches

Table 17.4 shows the ANOVA table for CRI for all the 81 cases in the run matrix. Note that X1 through X4 represent the four factors being analyzed here and have been listed below the table. The following conclusions can be derived from the table.

(1) The p-values for the four factors show that all four of them are statistically significant.
(2) Looking at the main effects alone may indicate that sample size is relatively much more significant than the other three factors. However, the interaction effects show that the variables have significant interactions among themselves. Only the interaction between production variability and sample size is not significant. Although the p-value for interaction effect between measurement variability and sample size is larger than 0.05, it is not far from this level of confidence.
(3) Interaction effect between production variability and measurement variability leads to confounding results. For example, when X1 is 0.1 and X2 is 0.5 CRI is 26.3 (N=10, Bias = 0.0: Case 20). On the other hand, CRI is lower (17.5) when X1 and X2 are both high (0.5). This happens because as production variability increases, the thickness values would widely vary and be away from the target in many cases. This leads to masking the effect of measurement variability. Also, since several of the cases would fall outside the specification window, measurement error would not lead to higher or lower pay, thus leading to lower risk.
(4) Measurement error can be controlled by the agency although it can most likely never be reduced to zero. If the measurement error is kept at a minimum possible level the risk in payment would go down while using the same QA program. Maintaining control over measurement variability is generally not too difficult. It would require that repeated measurements be taken in the beginning to assess the repeatability of the instrument/method and that calibration be checked while doing the test section in the beginning of the project.
(5) Bias seems to have the most drastic effect on payment risk because it can not only change the magnitude of risk but also alter the sign of risk. However, CRI does not catch this phenomenon because CRI treats the positive and negative risk as equally undesirable and does not discriminate between the two. This can be seen as a shortcoming of CRI. However, the authors have found through experience that it is very difficult, if not impossible, to design an index which is sensitive to the magnitude as well as sign of risk in the same plot. It means that most likely an accompanying risk index would have to be defined to cater to the needs of balancing risk between the agency and the contractor.

Table 17.4 ANOVA table for CRI for all 81 cases in the run matrix

Source	Sum Sq.	d.f.	Mean Sq.	F	Prob>F
X1	339.1	2	169.551	30.02	0
X2	248.37	2	124.184	21.99	0
X3	1818.26	2	909.132	160.98	0
X4	286.35	2	143.175	25.35	0
X1*X2	218.61	4	54.652	9.68	0
X1*X3	34.54	4	8.635	1.53	0.2087
X1*X4	120.1	4	30.025	5.32	0.0013
X2*X3	54.54	4	13.635	2.41	0.0616
X2*X4	78.93	4	19.733	3.49	0.0139
X3*X4	125.96	4	31.491	5.58	0.0009
Error	271.07	48	5.647		
Total	3595.84	80			

X1: Production variability

X2: Measurement variability

X3: Sample size

X4: Bias

17.8 Summary of Results from ERS Risk Analysis

This chapter presents the details of the Monte-Carlo based simulation that was developed as part of this project to assess the current QA program of MDOT for rigid pavements. The analysis conducted using the simulation showed that production variability, measurement variability, sample size and bias have significant influence on the risk in payment to be made to the contractor. This knowledge leads to identification of ways to reduce payment risk. The simulation can be used to analyze all other variables of a QA program and thereby improve it to achieve a lower risk of overpayment or underpayment. The analysis also showed that if production variability is high despite very low measurement variability and mean production being in the middle of the specification window, risk exists. Therefore, not only the contractor should produce right around the target he should be encouraged to maintain low variability in production quality. This is also significant from the point of view of pavement performance, as has been shown in chapter 16.

Generally the test methods and instruments are standardized and calibrated in the beginning of the construction project. For longer projects, the instruments may develop bias with continued use over several days. Bias has a very significant effect on payment risk. Such situations can lead to disputes and even law suits. Therefore, bias must be avoided through suitable inspection of the functioning of the test instruments.

CHAPTER 18: Feedback Process to Design for PCC Pavements

The aim of a quality assurance program for pavement construction is to assess the quality of the pavement constructed by the contractor and pay the contractor accordingly. It invariably involves testing for various quality characteristics. The data collected through this effort should therefore represent the quality of the end product as compared to the quality targeted through the design process. The QA program therefore, can not only be used for determining the payment to be made to the contractor but also to provide feedback to the design process itself.

A feedback process is required primarily to check if pavement materials and layers are being produced according to the design plans and if the variability is within acceptable limits. The as-constructed QA data can then be used to update the main statistics of input design variables (mean and standard deviation), which can be fed back into the design system to revise the expected performance. Figure 18.1 schematically shows the feedback process.

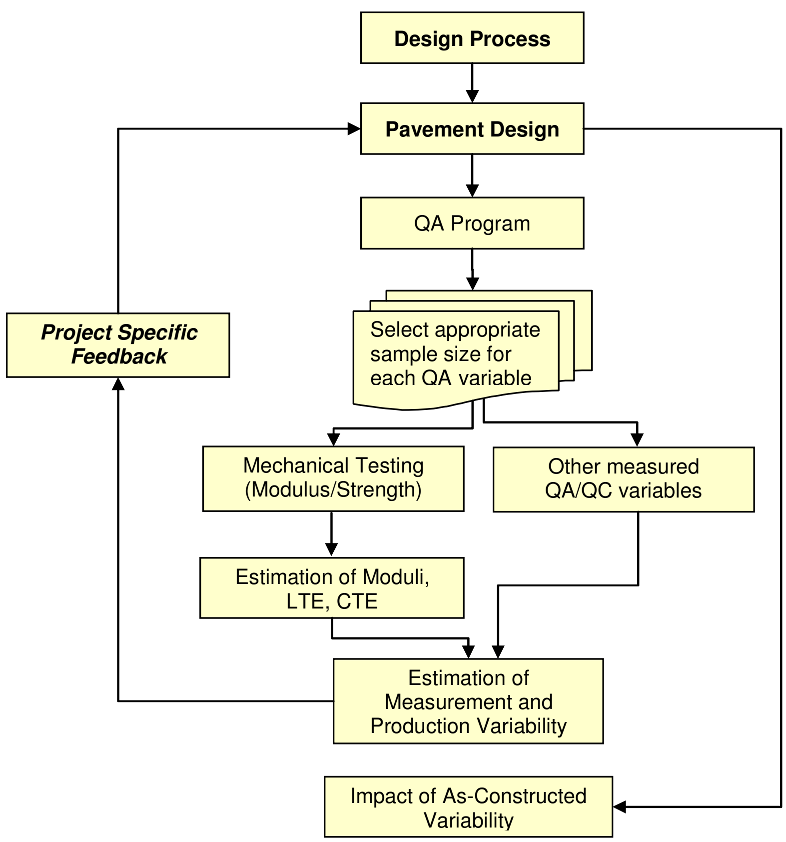


Figure 18.1 Flowchart showing feedback process for design

The following provides further description of the elements in this feedback process.

(1) *Selection of appropriate sample size:* One of the most important variables in the feedback process which needs to be optimized is the sample size. As the sample size becomes larger the confidence interval for a given design input (quality characteristic) tightens around the mean. The tighter the confidence interval the better the feedback process. This is discussed in detail in section 18.1.

(2) *Mechanical and/or material testing for modulus/strength*: MDOT currently uses the AASHTO 1993 design guide for designing its pavements. Modulus values of the constructed pavement are required for the pavement structural design. The modulus value for the PCC slab can be indirectly measured through non-destructive testing in the field (e.g., FWD test) or directly measured through laboratory testing of cores obtained from the field. Note that the M-E PDG method also requires modulus/strength testing in the form of PCC modulus of elasticity and modulus of rupture or compressive strength.

(3) *Estimation of moduli*: AASHTO 1993 uses the PCC modulus and the modulus of subgrade reaction to come up with the slab thickness. These can be estimated from mechanical and/or material testing (see item (2) above). In the M-E PDG framework, the E value is estimated through backcalculation using FWD test data (level 1) or from correlations with strength (levels 2 and 3). Therefore, in the latter case, the feedback will consist of updating the input strength data.

(4) *Use of other QA and QC data for design*: Quality characteristics data obtained through a QA program from pavement construction projects (e.g., slab thickness) can be used as input for design. An effort can also be made to collect the contractor's QC data as long as they are deemed comparable. This was described in sections 9.1 through 9.3 for HMA pavements where there was sufficient data for proper statistical inferences.

(5) *Estimation of measurement and production variability*: The overall variability that construction data shows has two components, namely (a) measurement variability and (b) production variability. Production variability is the actual variability in the constructed pavement because of variability in material, construction practices and equipment, and climatic conditions. When various tests are used to determine the level of quality achieved, production variability gets masked with measurement variability because of error in the test equipment and/or process. However, only production variability affects pavement performance. Therefore, in the beginning of a construction project measurement variability should be estimated for the various test methods to be used under the quality assurance program (by taking multiple measurements). This will help in estimating actual production variability in the constructed pavement.

(6) *Impact of as-constructed variability*: Production variability will lead to variability in pavement performance. In the AASHTO 1993 design procedure, the loss/gain in design life (ΔPSI) can be directly backcalculated using the design equation or by iteration using

the Darwin design software. In the M-E PDG framework, the software can be used directly to predict the loss/gain in pavement life.

18.1 Effect of Sample Size on Feedback Process using Simulation

Chapter17 of this report describes the development of a Monte-Carlo based simulation to assess risk in payment to be made to the contractor in the MDOT QA program for rigid pavements. The same fundamental concept of simulation can also be employed to develop an optimal feedback process to design. Section 9.2 in Chapter 9 established the validity of synthetically generated data being similar to the actual field data collected from MDOT construction projects for flexible pavements, both being normally distributed. The same exercise could not be done for rigid pavement QA data because of lack of availability of such data. However, it is expected that the nature of the data will be similar. The advantage of the synthetically generated data is that the error in the data is known a priori. Therefore, simulation using such synthetically generated data can be used to assess the extent to which data collected in the field represents true pavement quality compared to the design target. This section presents the details of this exercise.

One of the most important variables in the feedback process which needs to be optimized is the sample size. The feedback simulation developed in this project for concrete pavement construction was used to estimate the statistics enumerated below as a simultaneous function of sample size and mean of quality characteristic. Each scenario was simulated 10,000 times to identify the distribution of these statistics, allowing for a probabilistic study.

(1) Error in estimating the mean of a quality characteristic (concrete strength, slab thickness etc.) in a lot.

(2) Error in estimating the variability (standard deviation) in quality characteristic in a lot and

(3) Estimate of pavement life in terms of ESALs

All the above assessments were performed for a lot because the MDOT QA program determines pay factor on a lot basis. Figures 18.1 through 18.3 show the above mentioned statistics as a function of sample size and mean quality characteristic (Q/C). The middle surface in Figure 18.1 represents the mean error in estimate of mean quality characteristic and the surfaces above and below represent the 90% confidence interval for the error. This analysis was done for 28 day compressive strength for a pavement with slab thickness of 9 inches. The following observations can be made from this plot.

(4) The mean of the error is essentially equal to zero for all sample sizes and all values of mean Q/C.

(5) It can be clearly seen that as the sample size becomes larger the confidence intervals tighten around the mean. The tighter the confidence interval, the better the feedback process would be. A tight confidence interval means that the estimate of the error lies within a small window, or in other words, there is high probability that the error would be close to zero since the surface representing the mean of the error is essentially flat at zero level.

(6) Interestingly, the 90% confidence interval of error in ESALs estimate is wider for higher mean Q/C.

The decision regarding optimal sample size for feedback will have to be taken by MDOT. This is because defining the level of risk that MDOT is willing to take to save testing time by not having a very large sample size is a function of many considerations that only MDOT can weigh.

Figures 18.1 through 18.3 are very helpful in understanding the trend in error and therefore, how the optimal size should be selected. However, to be able to make this decision, MDOT would need a table or a plot showing a relationship between sample size and a metric tangible enough to make decisions (e.g., the width of the confidence interval).

Figure 18.2 shows the error in the estimate of variability (standard deviation) for different sample sizes and varying mean values of the quality characteristics. The overall behavior is similar to that observed in the case of error in estimate of mean Q/C. The difference is quite noticeable when sample size is small. For small sample size, the error is negative for all values of mean Q/C. In other words, a sample size would lead to an underestimation of variability.

Figure 18.3 shows the estimated pavement life (in ESALs) as a function of sample size and mean Q/C. The effect of sample size on pavement life is similar to that for the estimate of mean and standard deviation with the additional feature of wider confidence intervals for higher 28-day compressive strength. This happens because pavement life is nonlinearly related to strength of concrete.

Figures 18.4 through 18.6 were generated to get a better understanding of the magnitude of the effect of sample size on the three statistics being considered in this analysis.

Figures 18.4 and 18.5 have very similar trends and show that with increasing sample size the error in the estimate of mean and variability falls sharply in the beginning and then the reduction in error slows down. Therefore, MDOT will have to decide on the sample size beyond which the reduction in error is not worth the extra effort of having a larger sample size to increase gain. Figure 18.6 shows the reduction in error in estimated pavement life across the entire range of quality characteristic with increasing sample size.

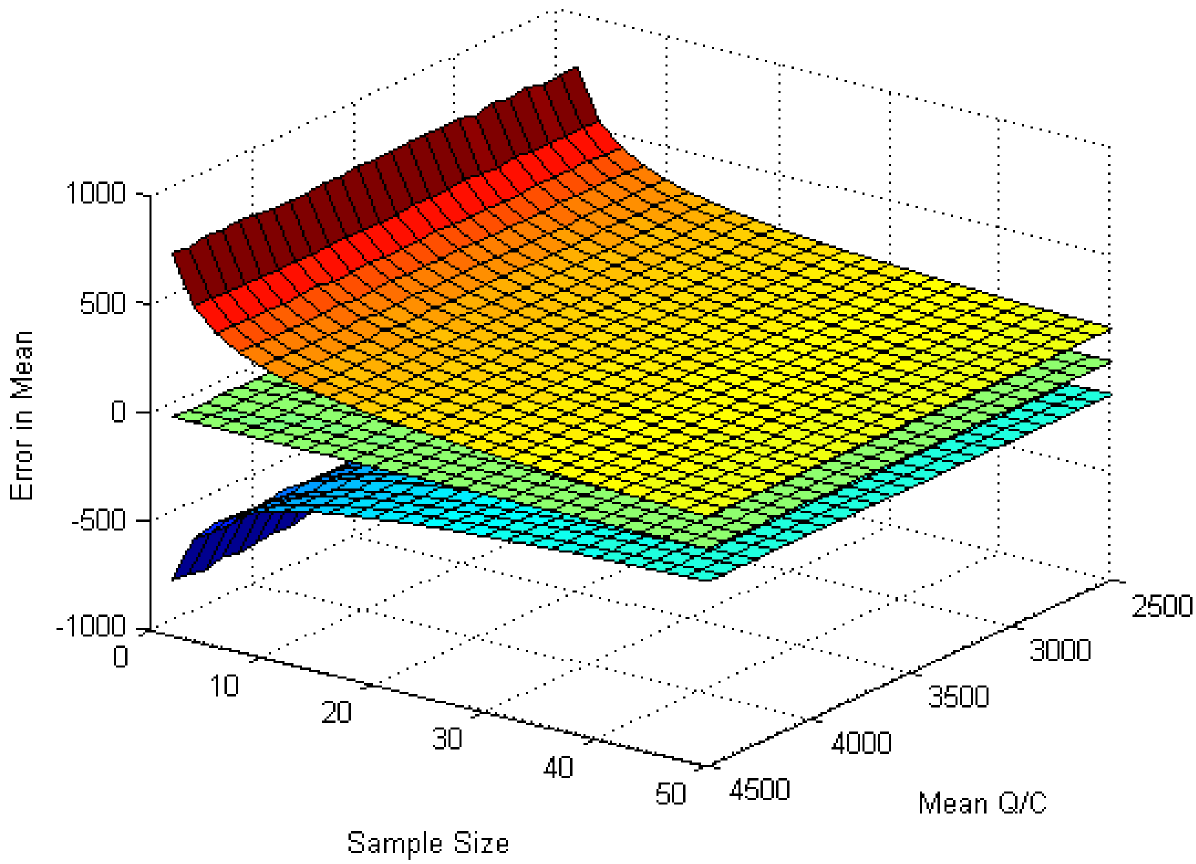


Figure 18.2 Error in estimate of mean quality characteristic with 90% confidence interval from feedback process

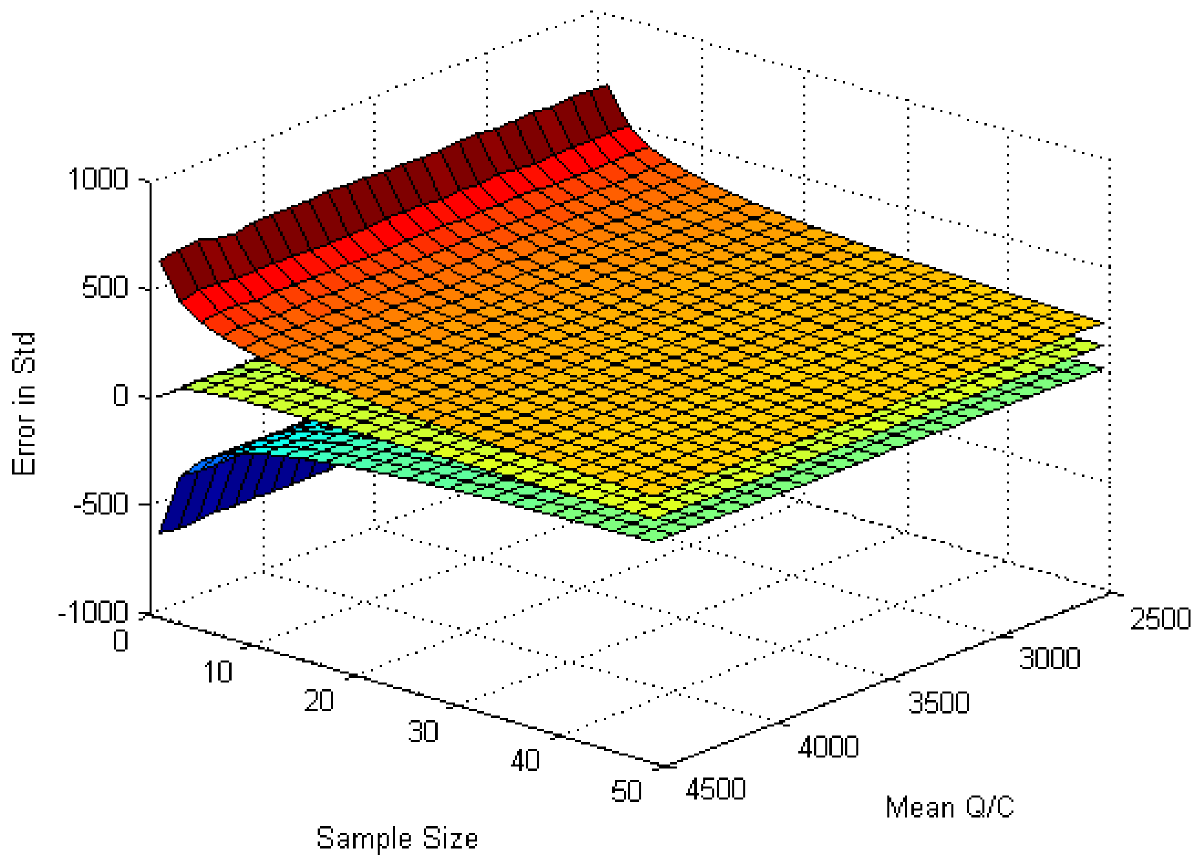


Figure 18.3 Error in estimate of variability in quality characteristic with 90% confidence interval from feedback process

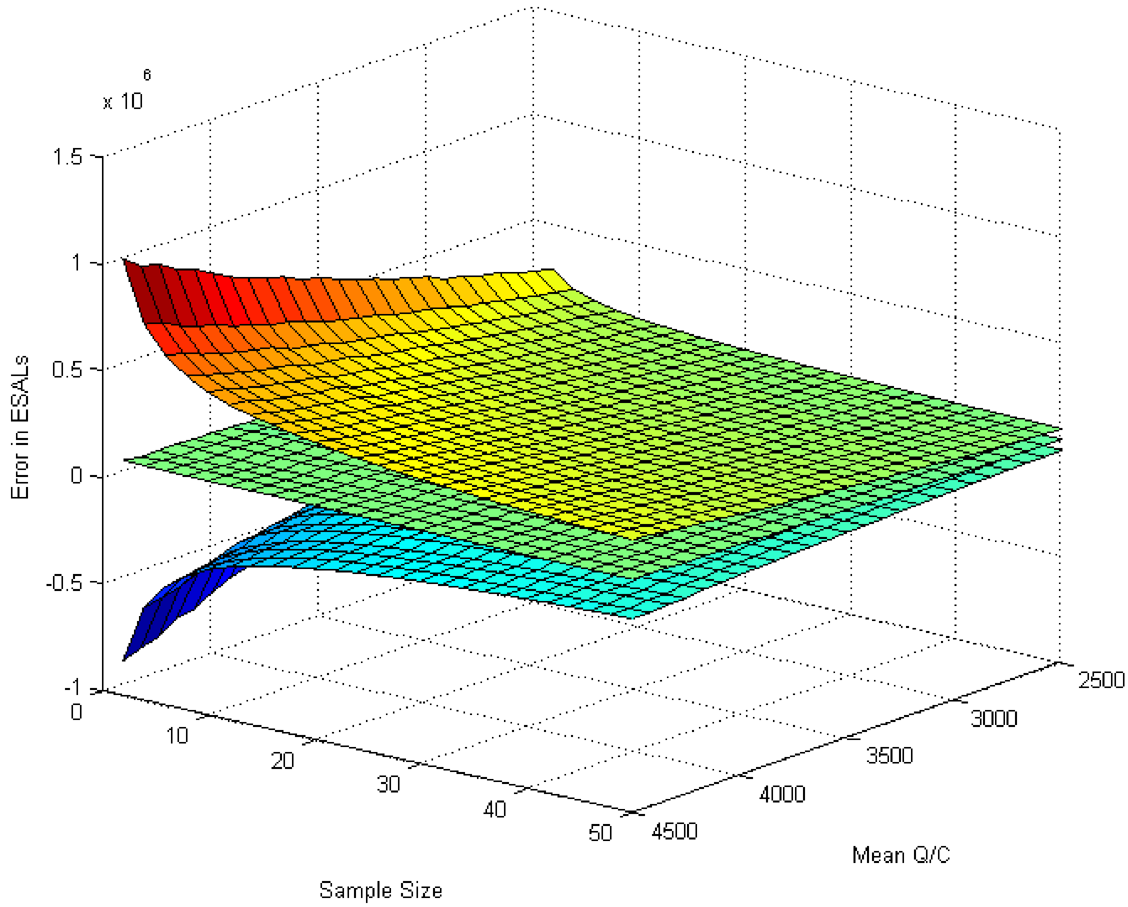


Figure 18.4 Error in estimate of pavement life (ESALs) with 90% confidence interval from feedback process

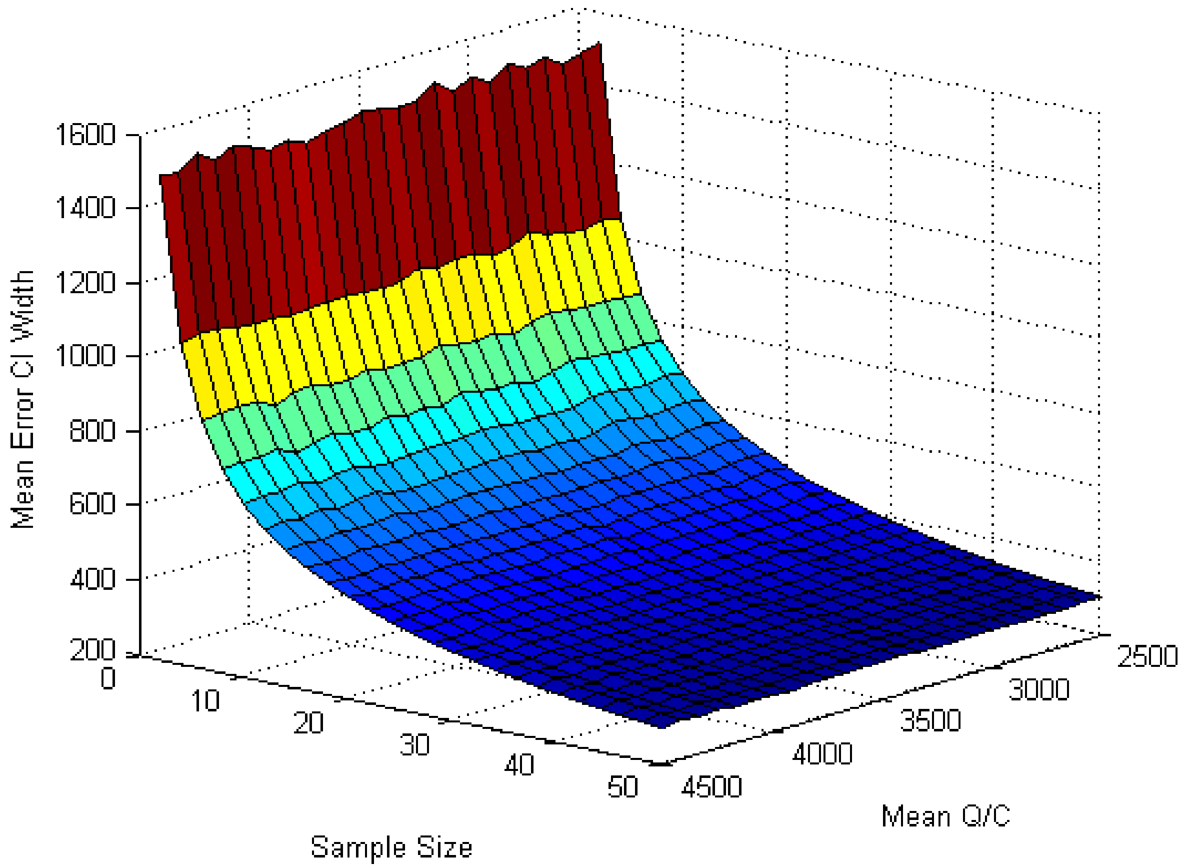


Figure 18.5 Width of 90% confidence interval in estimate of Q/C mean from feedback process

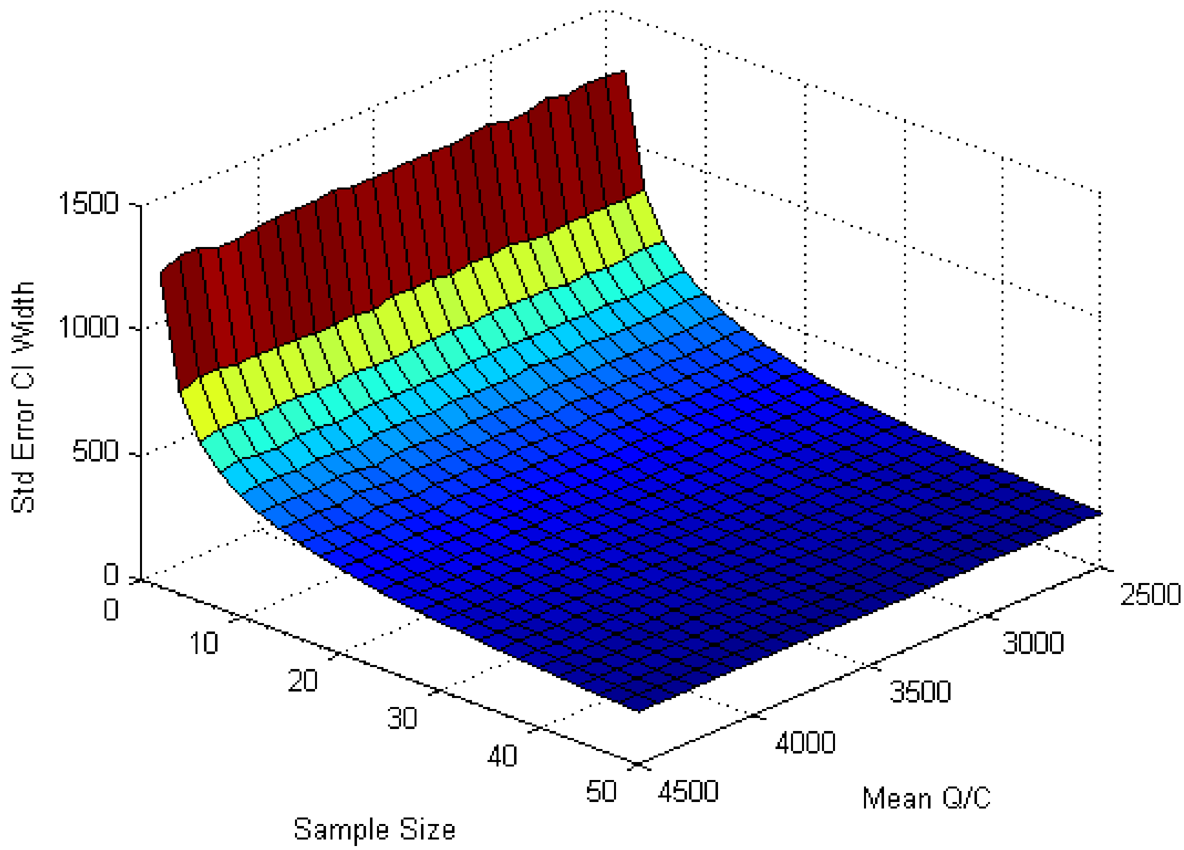


Figure 18.6 Width of 90% confidence interval in estimate of Q/C variability from feedback process

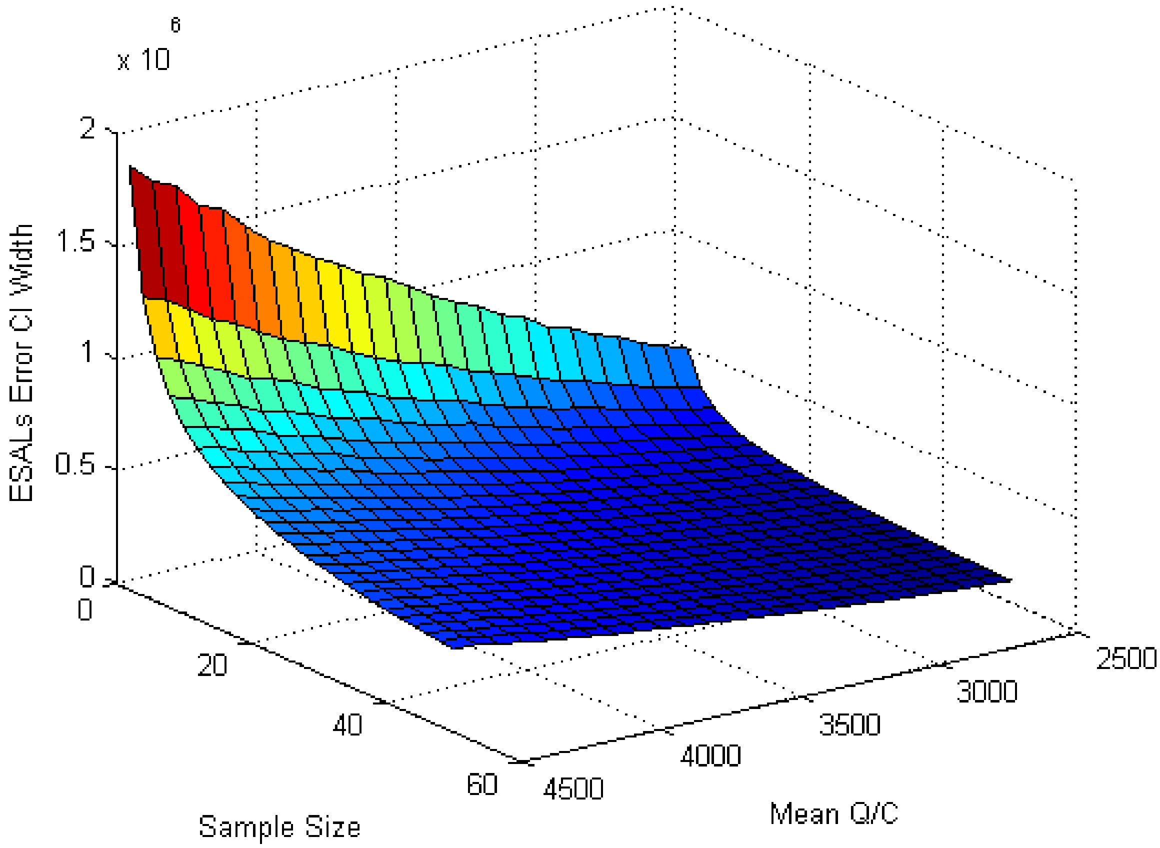


Figure 18.7 Width of 90% confidence interval in estimate of life (ESALs) from feedback process

Tables 18.1 through 18.3 present the same information as the preceding three figures (Figures 18.4 through 18.6) but in tabular form to be able to see the magnitudes of confidence intervals, which would enable making decisions. Since different projects will have different values for the mean quality characteristics, one should consider the width of the confidence interval for the entire range of the quality characteristic to decide on the sample size, although the sample size for the feedback process will probably have to be the same. It can be simplified one step further if we study the average width of the confidence interval for the entire range of quality characteristics versus sample size. The last row in the three tables presents this average value. The sample sizes used in the analysis were varied from 2 to 50 with a step of 2. For the sake of brevity, these tables present only a few selected values.

Table 18.1 Width of 90% confidence interval of error in estimate of mean quality characteristic (strength, in psi) for different sample sizes

Mean QC	Sample Size										
	2	6	10	14	18	22	26	30	34	40	50
2500	1513*	860	667	567	502	450	416	388	363	334	302
2583	1500	864	679	572	506	461	421	389	365	338	304
2667	1483	871	668	567	501	457	418	387	365	334	299
2750	1516	868	669	560	493	448	414	388	364	336	300
2833	1502	874	680	569	501	455	417	386	361	336	298
2917	1528	883	680	573	500	452	415	385	362	335	299
3000	1492	871	665	562	499	452	418	392	368	336	301
3083	1522	850	664	563	497	452	412	385	363	336	301
3167	1498	870	668	563	501	453	416	384	362	332	299
3250	1537	864	666	558	493	447	413	385	361	329	299
3333	1501	870	675	564	495	441	405	379	359	336	301
3417	1497	882	677	563	498	456	418	390	367	339	304
3500	1508	865	672	568	498	452	416	386	366	338	302
3583	1515	862	671	568	495	452	415	388	364	337	302
3667	1499	860	669	570	500	454	417	388	366	337	301
3750	1491	874	668	566	496	448	410	384	362	337	302
3833	1475	860	661	563	500	451	417	387	362	336	301
3917	1496	863	665	562	498	450	417	388	361	331	295
4000	1485	877	678	577	503	456	421	391	367	337	300
4083	1507	869	672	569	501	456	419	390	365	337	300
4167	1526	849	664	566	498	450	413	388	366	335	297
4250	1501	875	683	572	504	455	422	393	369	342	305
4333	1534	876	671	563	496	450	417	383	361	332	299
4417	1496	871	674	571	497	449	414	384	365	338	306
4500	1502	867	665	564	502	451	415	385	360	331	297
Avg.	1504.96	867.8	670.84	566.4	498.96	451.92	415.84	386.92	363.76	335.56	300.56

* ± 1513/2

Table 18.2 Width of 90% confidence interval of error in estimate of variability in quality characteristic (strength, in psi) for different sample sizes

Mean QC	Sample Size										
	2	6	10	14	18	22	26	30	34	40	50
2500	1270	646	494	412	360	319	290	271	254	234	208
2583	1280	651	500	415	360	326	299	278	260	241	217
2667	1285	645	492	408	357	320	294	271	255	236	211
2750	1270	637	488	406	356	319	295	273	256	236	210
2833	1294	649	489	409	359	322	294	275	260	239	211
2917	1273	638	489	405	356	320	294	273	257	238	214
3000	1265	653	497	413	362	327	299	277	261	239	215
3083	1290	653	490	410	360	324	296	273	256	236	210
3167	1279	658	491	407	356	320	293	272	256	238	214
3250	1272	659	503	414	361	325	297	275	256	235	210
3333	1274	644	486	402	352	318	291	269	253	234	207
3417	1275	654	492	410	357	320	292	271	256	235	210
3500	1272	651	490	406	357	318	291	269	252	233	208
3583	1272	651	497	410	357	320	293	273	256	236	211
3667	1270	642	484	403	355	322	292	273	255	235	210
3750	1290	645	493	412	360	325	298	277	260	240	214
3833	1278	653	496	409	357	321	295	275	258	238	212
3917	1279	645	493	415	361	326	297	275	259	239	213
4000	1281	644	480	403	351	323	295	274	257	236	210
4083	1271	650	491	412	357	324	299	277	260	240	213
4167	1258	652	494	419	364	325	299	278	261	241	213
4250	1258	642	483	401	355	322	295	273	257	236	211
4333	1293	654	494	411	358	322	294	273	257	236	210
4417	1289	656	492	408	356	320	295	277	260	240	213
4500	1256	639	483	409	359	322	295	274	258	238	211
Avg.	1275.76	648.44	491.24	409.16	357.72	322	294.88	273.84	257.2	237.16	211.44

Table 18.3 Width of 90% confidence interval of error in estimate of pavement life (in 100,000 ESALs) for the quality characteristic for different sample sizes

Mean QC	Sample Size										
	2	6	10	14	18	22	26	30	34	40	50
2500	4.99	2.87	2.20	1.87	1.65	1.48	1.37	1.28	1.20	1.10	0.99
2583	5.33	3.06	2.40	2.02	1.78	1.62	1.48	1.37	1.29	1.19	1.07
2667	5.68	3.31	2.56	2.18	1.91	1.73	1.59	1.47	1.39	1.27	1.13
2750	6.22	3.53	2.73	2.28	2.01	1.82	1.68	1.57	1.47	1.36	1.21
2833	6.54	3.78	2.96	2.47	2.17	1.96	1.80	1.67	1.56	1.45	1.28
2917	7.06	4.05	3.12	2.62	2.29	2.07	1.91	1.77	1.66	1.53	1.37
3000	7.33	4.29	3.30	2.77	2.46	2.24	2.06	1.93	1.81	1.65	1.48
3083	8.04	4.51	3.50	2.97	2.62	2.38	2.17	2.02	1.89	1.76	1.57
3167	8.37	4.83	3.71	3.14	2.79	2.53	2.32	2.15	2.02	1.85	1.67
3250	9.01	5.09	3.93	3.28	2.92	2.66	2.46	2.28	2.14	1.95	1.76
3333	9.41	5.41	4.20	3.53	3.09	2.77	2.54	2.38	2.24	2.09	1.88
3417	9.96	5.86	4.47	3.71	3.28	3.01	2.74	2.57	2.42	2.24	2.01
3500	10.60	6.08	4.69	3.97	3.47	3.15	2.90	2.69	2.55	2.36	2.11
3583	11.24	6.36	4.92	4.18	3.65	3.33	3.06	2.86	2.69	2.49	2.23
3667	11.63	6.66	5.20	4.42	3.87	3.52	3.24	3.02	2.85	2.62	2.33
3750	12.31	7.14	5.49	4.64	4.06	3.67	3.37	3.15	2.97	2.76	2.48
3833	12.82	7.45	5.72	4.88	4.32	3.89	3.60	3.34	3.11	2.88	2.59
3917	13.48	7.78	6.04	5.11	4.53	4.09	3.80	3.53	3.29	3.02	2.69
4000	14.09	8.31	6.41	5.47	4.78	4.33	3.99	3.71	3.48	3.20	2.85
4083	14.92	8.69	6.74	5.69	5.02	4.58	4.19	3.90	3.66	3.37	3.00
4167	15.93	8.94	6.96	5.95	5.23	4.73	4.33	4.07	3.83	3.52	3.12
4250	16.32	9.50	7.44	6.22	5.48	4.97	4.61	4.28	4.03	3.73	3.33
4333	17.45	9.98	7.69	6.46	5.69	5.16	4.78	4.39	4.13	3.81	3.42
4417	17.88	10.46	8.08	6.85	5.96	5.37	4.96	4.60	4.37	4.04	3.66
4500	18.82	10.85	8.31	7.05	6.26	5.64	5.18	4.80	4.49	4.12	3.69
Avg.	10.67	6.34	5.11	4.53	4.20	4.03	3.93	3.88	3.87	3.90	4.04

Figures 18.7 through 18.9 present the summarized form of the results obtained from this analysis. Figure 18.7 shows the maximum error in estimate of mean in 90% of the cases for different sample sizes. In other words, if the sample size is 50 (for example) the maximum error, in 90% of the cases, will be lower than 300 psi (in 28-day compressive strength in this case).

However, if the sample size is only 8, the error in mean could be as high as 750 psi. If only two samples were collected, the error in the mean could be as high as 1500 psi in 90% of the cases.

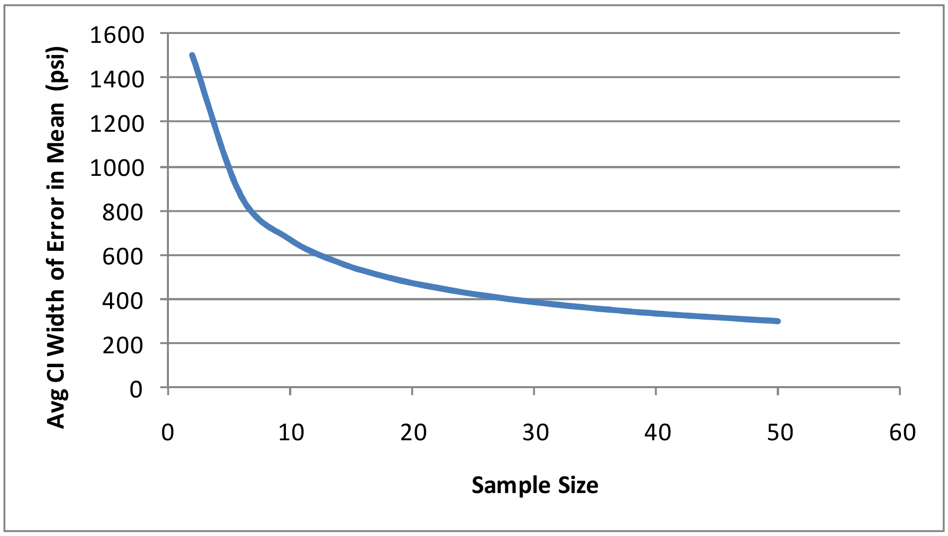


Figure 18.8 Average width of 90% confidence interval of error in estimate of mean

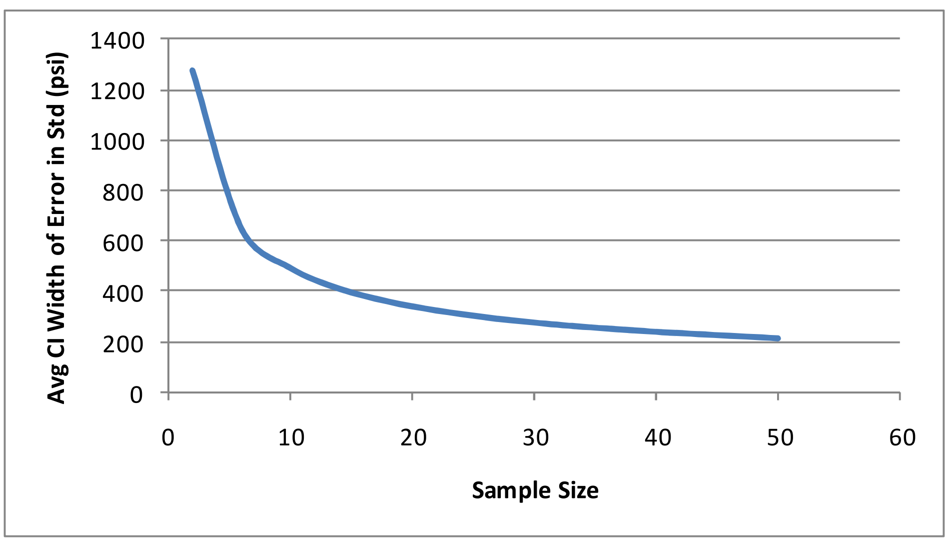


Figure 18.9 Average width of 90% confidence interval of error in estimate of variability

This analysis not only shows the benefit of having a larger sample size but also it quantifies the benefits in terms of reduction in error. Figure 18.8 shows the maximum error in estimate of standard deviation in 90% of the cases for different sample sizes. Figure 18.9 is probably even more relevant because it shows how the error in estimated pavement life in ESALs decreases (in 90% of the cases) with increasing sample size for a lot. It is also more relevant because it includes the effects of errors in the estimation of the mean as well as variability. It should be noted that these errors are for each of the lots.

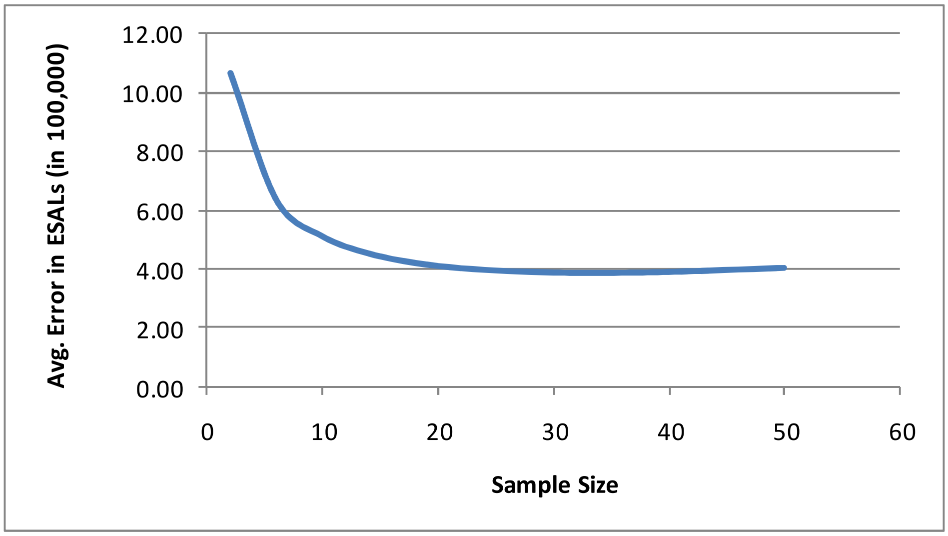


Figure 18.10 Average width of 90% confidence interval of error in pavement life (in ESALs)

A project will have several lots and the errors may cancel each other out, at least partially, when the payment is calculated for the entire project. However, a good quality assurance program should minimize risk in lot pay factors as well.

18.2 Conclusion

This chapter proposed a feedback process to design using QA and QC data. This chapter also presented the results of a simulation in order to develop an optimal feedback process for design. The simulation helped in estimating the errors that can be caused by the sample size that is used in the feedback process and the associated probabilities. Plots were developed to relate sample size to probabilistic error in estimating the mean and standard deviation of the quality characteristic and estimated pavement life. These plots can be used directly by MDOT to decide on the appropriate sample size (not too small to lead to higher errors and risk and not too large to be too costly or impossible to carry out).

CHAPTER 19: Conclusions and Recommendations for Rigid Pavement QA Program

This chapter presents the overall strategy adopted in this research followed by the conclusions and recommendations that have been derived. A good quality assurance program should use the quality characteristics which can ensure pavement performance that meets or exceeds the design target. While there are several components to the QA program, identification of the suitable quality characteristics is the most important one.

19.1 Identification of Suitable Quality Characteristics for QA Program

Identification of suitable QA characteristics requires preparing an exhaustive list of potential characteristics which should be considered for inclusion in the QA program. The different sources for preparing this list are enumerated below.

(1) Quality characteristics being used in other states' QA programs: Different states use varying combinations of quality characteristics. Some of those quality characteristics are used in determining payment to be made to the contractor for any project, while others are used merely to provide feed back for proper construction.
(2) Quality characteristics being used in other states' QC program: Any of the quality characteristics which are used in a QC program, i.e. monitored by the contractor, but are not used in the QA program should also be considered.
(3) Quality characteristics used in the Mechanistic-Empirical Pavement Design Guide Software: The MEPDG software uses models to predict pavement performance from the material, pavement structure, construction, traffic and environmental variables. The models used in the software are the result of studies carried out by many research teams after extensive testing and analysis to relate those variables to performance. Therefore, those variables or quality characteristics which are within the control of the contractor and which can be tested at the time of construction should also be included in the list.
(4) Quality characteristics studied in other research projects which have been shown to have impact on performance.

The second step is to shortlist those candidate QA variables which can be shown to affect pavement performance. These relationships can be established through one or more of the following options.

(1) Empirical data from Michigan: If empirical data can be obtained which establish that changes in levels of one quality characteristic leads to change in pavement performance, either individually or in conjunction with other quality characteristics, then it would be the most preferred way.
(2) Empirical data from other states: If Michigan data is not available or is not good enough to establish relationships mentioned in option 1 then empirical data from other states can be used. This is a slightly more indirect way of establishing whether a certain quality characteristic should be used in the Michigan QA program. This is because any other state may have climate, typical construction materials, construction practices and traffic different from those in Michigan. However, if some of these factors are matching with Michigan and/or if those parameters affect performance very significantly then they must be analyzed using such data.
(3) Analysis using MEPDG: MEPDG software has the best available response and performance models for flexible and rigid pavements. The models are generally mechanistic-empirical in nature. They have been developed and calibrated using empirical data. Therefore, analysis performed using these models is similar to using empirical data but with more flexibility, although they probably have greater error. However, it is important to note that quantification of the relationships is not important. Relative differences in performance because of these quality characteristics are more relevant. MEPDG also allows for studying the effect of variability in these quality characteristics on pavement performance.
(4) Use of other analysis tools: MEPDG does not include all the construction related inputs, particularly for rigid pavements. For example, the time of the day when pouring of concrete is done is quite important from the point of view of early age cracking and/or built-in curling in concrete pavements. Therefore, alternate software like HIPERPAV can be used to study such construction related issues.
(5) Other research studies: Other research studies firmly establishing relationships between the candidate QA variables and performance can also be used to verify the findings from the above four options. In the case of variables for which none of the above options can be feasibly used for analysis this may be the only option.

The third and last step is to identify those variables which should be incorporated into the Michigan QA program. The criteria for including these variables are that:

(1) They affect pavement performance either directly or in conjunction with other variables.
(2) They need to be tested individually and can not be estimated or calculated from other significant QA variables already being used in the QA program. For example there is a strong correlation between compressive strength and flexural strength of concrete.
(3) It is feasible to test for them within a reasonable amount of time during the construction.

(4) The testing for these candidate QA variables does not require very specialized or costly equipment.

It is possible for the contractor to control these variables through sound construction practices and tight quality control.

19.1.1 Comparison of MDOT QA programs with others in the US

The use of performance-related specifications for PCC pavements is not used by as many agencies as for flexible pavements; however their use is increasing more rapidly than the HMA pavements. Michigan uses air content, pavement thickness, slump and cylinder strength in its QA program. These are similar to the QA programs used by the majority of the states. In other words there is nothing alarmingly different in the QA program being used by MDOT as compared to other "ERS" states.

19.1.2 QA Parameters Indentified by Other Studies

Based on literature review for studies that looked at the effect of pavement design and construction variables on performance, the following variables were identified as key QA parameters:

1. Air Content
2. Thickness
3. Slump
4. cylinder strength
5. Gradation
6. Beam strength
7. Water-cement ratio
8. Ride quality
9. Aggregate fractured faces
10. Sand equivalence
11. Permeability
12. Core strength

Air content is used by 38 agencies; thickness is used by 36; and slump is used by 33. Thirty one agencies accept PCC structures based on cylinder strength, and 26 accept gradation. The lesser-used acceptance attributes are aggregate fractured faces, sand equivalence, permeability and core strength.

MEPDG was extensively used to analyze the candidate QA variables for flexible pavements earlier in this project. In the case of rigid pavements MEPDG software accepts inputs mainly corresponding to design of the pavement, e.g. amount of cementitious material, water to cement ratio etc., and fewer inputs with respect to construction, such as temperature of fresh concrete before pouring, time of the day when the concrete was poured etc. However, two of the expectedly most significant variables, namely slab thickness and 28-day compressive strength of concrete can be studied using MEPDG.

19.1.3 Summary of Results from Empirical Analysis

An attempt was made to collect data from Michigan rigid pavement construction projects. However, the data analysis showed that most of the construction data was either lost or unaccounted for. Therefore, alternative sources of data were explored for determining how quality characteristics used in QA programs affect pavement performance. A preliminary analysis was first performed to study the relationship of acceptance parameters such as thickness and strength to performance (e.g., cracking and faulting) using data from Long Term Pavement Performance (LTPP) projects. In this analysis data from several states were used. These states geographically lie in different climatic zones. The LTPP database contains performance data (cracking, faulting, IRI etc.) and design and construction data (including physical inventory data, material properties from in-situ and laboratory tests). For the preliminary analysis all the data were derived from the Specific Pavement Studies – 2 (SPS -2) experiment. This analysis was followed by another analysis with data from General Pavement Studies (GPS) experiments.

Since Percent-Within-Limits (PWL) takes into account the mean as well as standard deviation of the quality characteristic it is a good measure of quality control performed during construction. However, it is important that PWL be related to actual pavement performance. An effort was made to find out if this holds true for pavements for which construction and performance data are available in the LTPP database. However, no clear trend was observed between PWL for compressive strength and faulting and cracking performance. One reason for this is that despite the large number of data points available in the database, the variability in independent variables (i.e. strength, etc.) was much smaller compared to performance. Since the performance of the pavement is affected by many factors and, it may be getting confounded because of those other factors if PWL does have influence on performance. These conclusions, therefore, indicated that a thorough mechanistic-empirical analysis needed to be performed to derive firm conclusions required for assessing quality assurance programs like the one being used by the state of Michigan.

19.1.4 Summary of Results from Mechanistic-empirical Analysis

Two different mechanistic empirical approaches were used: MEPDG and HIPERPAV II. The analysis using MEPDG was performed to study the effect of compressive strength and thickness of PCC pavements on pavement performance (i.e., cracking, ride quality and faulting). HIPERPAV II was developed as a tool for predicting early age behavior and its influence on long-term pavement performance for JPCP and CRCP pavements.

MEPDG software accepts inputs mainly corresponding to design of the pavement, e.g. amount of cementitious material, water to cement ratio etc., and fewer inputs with respect to construction, such as temperature of fresh concrete before pouring, time of the day when the concrete was poured etc. However, two of the expectedly most significant variables, namely slab thickness and 28-day compressive strength of concrete can be studied using MEPDG. The analysis shows that when the PWL values are lower for both strength and thickness, percent cracking is high. The results also show that the effect of deviations from the target compressive strength and slab thickness is drastic. The analysis for IRI shows a similar trend to slab cracking. It was also observed that faulting does not seem to be affected by strength and thickness levels.

HIPERPAV II takes into account the effect of construction related variables. The variables considered in this analysis are (a) time of the day when concrete is poured, (b) month of construction and (c) temperature of the fresh concrete at the time of pouring. It may be argued that month or time of construction are not QA variables, and therefore, need not be studied in this project. This analysis shows that all the three factors analyzed here can have significant influence on pavement performance. However, the first two factors, namely time of the day and time of the year of construction are not QA variables. They can possibly be used to provide guidelines to the contractor for better construction. The third factor, i.e. temperature of fresh concrete, is strictly not a "performance-related" or even "end-result" variable either. However, the effect of these factors can possibly be checked. Built-in curling can possibly be checked after 24 to 48 hours of construction using either a dip-stick or falling weight deflectometer. Premature cracking because of these factors would also appear within the first few days of construction which should be checked by the state department of transportation as part of their QA program.

19.2 Summary of Results from ERS Risk Analysis

Monte-Carlo based simulations were developed as part of this project to assess the current QA program of MDOT for rigid pavements. The analysis was conducted using a simulation that showed that production variability, measurement variability, sample size and bias have significant influence on the risk in payment to be made to the contractor. This knowledge leads to identification of ways to reduce payment risk. The simulation can be used to analyze all other variables of a QA program and thereby improve it to achieve a lower risk of overpayment or underpayment. The analysis also showed that if production variability is high despite very low measurement variability and mean production being in the middle of the specification window, risk exists. Therefore, not only the contractor should produce right around the target he should be encouraged to maintain low variability in production quality. This is also significant from the point of view of pavement performance, as has been shown in chapter 16.

Generally the test methods and instruments are standardized and calibrated in the beginning of the construction project. For longer projects, the instruments may develop bias with continued use over several days. Bias has a very significant effect on payment risk. Such situations can lead to disputes and even law suits. Therefore, bias must be avoided through suitable inspection of the functioning of the test instruments.

19.3 Summary of Results from Feedback Process for Design

A feedback process to design using QA and QC data was proposed. In addition, an analysis was performed to design an optimal feedback process for design. The simulation helped in estimating the errors that can be caused by the sample size that is used in the feedback process and the associated probabilities. Plots were developed to relate sample size to probabilistic error in estimating the mean and standard deviation of the quality characteristic and estimated pavement life. These plots can be used directly by MDOT to decide the appropriate sample. Small sample size will lead to higher errors and risk; whereas, large sample size could be too costly or impossible to carry out.

19.4 Use of Non-destructive Tests in QA Program

A detailed review of non-destructive tests is provided in Appendix B. The following are some of the relevant tests for use in QA programs of rigid pavements:

- Ground Penetration Radar (GPR) testing for thickness measurement
- Falling Weight Deflectometer (FWD) testing for modulus estimation
- Dynamic Cone Penetrometer (DCP) and/or lightweight FWD for modulus measurement of unbound layers
- MIT SCAN-2 for verification of dowel bar positions
- Maturity test for monitoring of early concrete strength development
- Air Void Analyzer for measuring entrained air non-destructively.

19.5 Overall Conclusion and Recommendations

Based on the review of Michigan and other DOT QA programs, it is concluded that the MDOT QA program is on par with ERS based QA programs used by the majority of the states. In other words there is nothing alarmingly different in the QA program being used by MDOT as compared to other “ERS” states. The results presented in this report confirmed the importance of concrete strength and slab thickness for cracking performance. It is also largely accepted that air void content is critical for the long-term durability of concrete. It is recommended that the maturity and CTE tests be considered as additional candidate QA tests.

Because empirical analyses linking key characteristics to long-term performance were inconclusive (not enough data from the MDOT construction database and inconclusive results from the LTPP database), it is recommended that the mechanistic-empirical approach be adopted for this purpose. With the future possible adoption of the MEPDG by MDOT and other DOT's, it is suggested that the MEPDG be adopted for this purpose. The analyses conducted as part of this research study can serve as examples for such future efforts. The advantage of mechanistic-empirical approach is its ability to quantify the relative effects of deviations from the target on long-term performance and to include interactive effects between different QA characteristics. This allows for modifying/refining the pay formulae based on rational arguments.

Therefore, potential improvements to the QA program should focus on fine tuning the specification limits used and refining the pay formulae to minimize the risk associated with construction variability. In addition, combining certain QA construction quality characteristics (e.g., strength and slab thickness) in the lower limits within these formulae should help in preventing extreme combinations that have drastic negative effects on pavement performance. Ideally, these refinements should be made based on mechanistic analyses.

The QA data and the pavement surface distress data obtained by MDOT's PMS are the two most relevant data for evaluating the effectiveness of the current QA processes. Unfortunately, MDOT's QA data are either incomplete or missing. A good database system for storing QA data should therefore be developed.

The complexity of the QA processes increases as the number of characteristics is increased. If we rely on probability/statistics methods to investigate the impacts of acceptance sampling rules on the risks of accepting poor quality level of products and rejecting good quality level of products, it may suggest that there is a need to investigate how to reduce the number of characteristics for QA processes without affecting product quality level. However, if we use simulations based on mechanistic modeling, we can account for multiple QA characteristics and their interactions without the need for complex analyses.

Finally it is recommended that the QA data be used as part of the feedback process for design, as described in chapter 18 of this report. Results from probabilistic analyses like those described in chapter 18 can be used for the selection of optimal sample size for QA testing in order to minimize the error in estimating the mean and standard deviation of the quality characteristic and estimated pavement life.

The use of non-destructive testing to quantify as-constructed material properties should be made a systematic part of the QA program. For PCC pavements, GPR and FWD testing should be conducted as they offer complementary information on the pavement structure and material properties/parameters. MIT scanning should be used to verify dowel bar positions. In-situ tests for measuring entrained air content and early concrete strength gain should also be made part of the QA program.

References

AASHTO, “AASHTO Guide for Design of Pavement Structures”, American Association of state Highway and Transportation Officials, 1993.

AASHTO Implementation Manual for Quality Assurance, AASHTO Quality Construction Task Force, American Association of state Highway and Transportation Officials, Washington, D.C., Feb .1996, pp 84 .

AASHTO Quality Assurance Guide Specification. AASHTO, Washington, D.C., 1995.

AASHTO Technology Implementation Group, http://tig.transportation.org.

ACI Committee 363, "State of the Art Report on High Strength Concrete." American Concrete Institute Journal, 81(4), 364-411, 1984.

Afferton, K. C., J. Freidenrich, and R. M. Weed. Managing Quality, Time for a National Policy. In *Transportation Research Record 1340*, TRB, National Research Council, Washington, D.C., 1992, pp. 3–39.

Al-Ostaz, A., “Effect of Moisture Content on the Coefficient of Thermal Expansion of Concrete”, *Federal Highway Administrations*, FHWA/MS-DOT-RD-07-187, 2007

American Concrete Pavement Association, “Stabilized Subbases and Airfield Concrete Pavement Cracking”, *Concrete Pavement Research & Technology*, No. 3.06, 2002

Aurilio, V., Buttlar W. G., Manik A., Shanley L., “Accounting for Bias and Between-Laboratory Test Differences”, Proceedings of 47th Annual Conference of the Canadian Technical Asphalt Association, Calgary, 2002.

Asphalt Institute, “Research and Development of the Asphalt Institute Manual (MS-1)”, 9th ed. Report 82-2, 1982.

ASTM Designation C 805 -94,”Standard Test Method For Rebound Number of Hardened Concrete”,”1995 Annual Book Of ASTM Standards”,Volume 04.02,”Concrete And Concrete Aggregates”, American Society For Testing And Materials,Philadelphia ,PA.

Barros, R. T., R. M. Weed, and J. H. Willenbrock. Software Package for the Design and Analysis of Acceptance Procedures Based Upon Percent Defective. In *Transportation Research Record 924*, TRB, National Research Council, Washington, D.C., 1983, pp. 85–93.

Benson, P.E., “Comparison of End Result and Method Specifications for Managing Quality,” TransportationResearch Record 1491, Transportation Research Board, National Research Council, Washington, D.C., 1995, pp. 3–10.

Bowery, F.E., Jr., and S.B. Hudson, NCHRP Synthesis of Highway Practice 38: Statistically Oriented End-Result Specifications, Transportation Research Board, National Research Council, Washington, D.C., 1976, 40 pp.

Box, G. E. P. and Wilson, K.B. 1951. On the Experimental Attainment of Optimum Conditions (with discussion). Journal of the Royal Statistical Society Series B, 13(1), pp. 1-45.

Brandes H. G. and Robinson C. E., Correlation of Aggregate Test Parameters to Hot Mix Asphalt Pavement Performance in Hawaii, *Journal of Transportation Engineering*, Vol. 132, No. 1, January 1, 2006.

Brown, E. R., “Density of Asphalt Concrete-How Much is Needed”, NCAT Auburn University, Research Report-NCAT Report No. 90-3, Alabama, U.S.A., 1990.

Brown, E. R., and Bassett, C. E., “Effect of Maximum Aggregate Size on Rutting Potential and Other Properties of Asphalt-Aggregate Mixtures”, *Transportation Research Record*, No. 1259, pp. 107-119, 1990

Brown, E.R. and Cross, S.A., “A National Study of Rutting in Hot Mix Asphalt (HMA) Pavements”, Proceedings of the Association of Asphalt Paving Technologists, Vol. 61, pp. 535—582, 1992.

Brown, E. R., and Cross, Stephen A., “A Study of In-Place Rutting of Asphalt Pavements”, *National Center of Asphalt Technology*, Report No. 89-02, 1989

Brown, E. R., and Cross, Stephen A., “Comparison of Laboratory and Field Density of Asphalt Mixtures”, *National Center of Asphalt Technology*, Report No. 91-1, 1991

Brown, E. Ray, M. Rosli Hainin, Allen Cooley, Graham Hurley. Relationships of HMA In-Place Air Voids, Lift Thickness, and Permeability. NCHRP report 531, project 9-27, Volume One-four, National Cooperative Highway Research Program. September 2004.

Buttlar, W G and Harrell, M, “Development of End-Result and Performance-Related Specifications for Asphalt Pavement Construction in Illinois”, Crossroads 2000, 1998, p. 195-202.

Buttlar WG, Hausman, J. J., "ILLISIM Program for End-Result Specification Development," Journal of the Transportation Research Board, National Research Council, National Academy Press, Washington, D. C., 1712 125-138, 2000.

California Department of Transportation, “METHOD TO DETERMINE PAVEMENT THICKNESS USING MAGNETIC TOMOGRAPHY TECHNOLOGY”, March 2007.

Carino, N.J., Chapter 5"The Maturity Method, Handbook on Nondestructive Testing of Concrete", CRC Press, Inc., Boca Raton ,FL,1991, pp.101-146

Chatti K., Buch N., Haider S. W., Pulipaka A., Lyles R. W., and Gilliland D. "LTPP Data Analysis: Influence of Design and Construction Features on the Response and Performance of New Flexible and Rigid Pavements,", Final Report, NCHRP Project 20-50(10/16), Transportation Research Board, National Research Council, NCHRP Web. Document 74, http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w74.pdf, February 2005.

Collop, A. C., Cebon D. "Parametric study of factors affecting flexible pavement performance", ASCE J. Transp. Eng., Vol. 121, No 6, pp 485-494, 1995.

Crawford, G.I., September 1997,U.S. Department of Transportation, Federal Highway Administration, "Guide to Nondestructive Testing of Concrete".

Malhotra,V.M., Surface Hardness Methods, Handbooks of Nondestructive Testing of Concrete", Chapter 1,"CRC Press,Inc.,Boca Raton,FL,1991,pp. 1-17

Crouch, L. K., Pitt, J., and Hewitt, R., "Aggregate Effects on Pervious Portland Cement Concrete Static Modulus of Elasticity", *Journal of Materials in Civil Engineering*, ASCE, July 2007, pp. 561-568,

Daniels J. J., "Fundamentals of Ground Penetrating Radar", Soft Earth Associates Inc., Westerville, Ohio, 1990.

Darter, M. I., Abdelrahman, M., Okamoto, P. A., and Smith K. D., "Performance-Related Specifications of Concrete Pavements, Volume I: Development of a Prototype Performance-Related Specification", *Federal Highway Administration*, FHWA-RD-93-042, November 1993

Deacon, J. A., C. L. Monismith, and J. T. Harvey .Pay Factors for Asphalt-Concrete Construction: Effect of Construction Quality on Agency Costs. Technical Memorandum, TM-UCB-CAL/APT-97-1, April 1997.

European Asphalt Pavement Association, Long-Life Asphalt Pavements – Technical version, http://www.eapa.org/START/positionprs_publications/papers/EAPA Paper - LLP Technical version - 2007.pdf, July 2007.

Finn, F., C.L. Saraf, R.Kulkarni, K. Nair, W. smith, and A. Abdullah, NCHRP Report 291 : Development of Pavement Structural Subsystem., TRB, National Research council , Washington , D.C., 1986.

Flintsch, G. W., et. al., "Asphalt Materials Characterization in Support of Implementation of the Proposed Mechanistic-Empirical Pavement Design Guide", Virginia Transportation Research Council, *Virginia Department of Transportation*, 2007

Force technology, "High Energy X-ray Radiography for Examining Reinforced Concrete", The e-Journal of Nondestructive Testing & Ultrasonics, July 1999.

Ghafoori, N., Dutta, S. “Laboratory Investigation of Compacted No-Fines Concrete For Paving Materials.” Journal of Materials in Civil Engineering. v7, No. 3, Aug., 1995, pp 183-191.

Haddock, J., Pan, C., Feng, A., White, T. D., “Effect of Gradation on Asphalt Mixture Performance”, *Journal of Transportation Research Board*, No. 1681, pp. 59-68, 1991

Halabe, U., A. Sotoodehnia, K. R. Maser, E. A. Kausel, .Modeling the Electromagnetic Properties of Concrete. *ACI Materials Journal*, Volume 90, No. 6, pp. 552-563, November-December 1993.

Hall, K. D.,.Williams, Stacy G, “Establishing Variability for Hot-Mix Asphalt Construction in Arkansas”, 81st Annual Meeting, Transportation Research Board, National Research Council, Washington D.C., 2002.

Hanna, A. N., “*Determination of In-situ Material Properties of Asphalt Concrete Pavement Layers from Nondestructive Tests”,* Research Results Digest Number 271, National Cooperative Highway Research Program, December 2002.

Hansen, W., Jensen, E., A., and Mohr, P., “The Effects of Higher Strength and Associated Concrete Properties on Pavement Performance”, *Federal Highway Administration Research, Development and Technology*, FHWA-RD-00-161, 2001

Harrigan, E.T., “LTPP Data Analysis: Significance of ‘As-Constructed’ Asphalt Concrete Air Voids to Pavement Performance”, Research Results Digest, *National Corporative Highway Research Program (NCHRP)*, No. 269, September 2002

Harris D. A., “A GPR Based Underground Workings Monitoring Program for Surface Mines”, M.S. Thesis, Department of Mining Engineering, Montana Tech of the University of Montana, Butte, MT, 1998.

Harvey, J. T, and Tsal, B., “Effects of Asphalt Content and Air Void Content on Mix Fatigue and Stiffness”, *Transportation Research Records*, No. 1543, pp. 38-45, 1996

Hasted J. B., “Aqueous Dielectrics”, London, Chapman and Hall, 1973

Hodgson, Shell S., “The Effects of Water/Cement Ratio and Air Entraining on Portland Cement Concrete Freeze/Thaw Durability”, *University of Wisconsin-Madison*, 2000

Holzschuhe, C., Lee H. S., and Greene J., “*Accuracy and Repeatability of Ground Penetrating Radar for Surface Layer Thickness Estimation of Florida Roadways”,* State Materials Office, Florida, April 2007.

Huang, Y., H. Pavement Analysis and Design, Prentice Hall, Englewood Cliffs, N.J., 1993.

Hudson, W.R., G. E. Elkins, W. Uddin, and K.T. Reilley,” Evaluation of Pavement Deflection Measuring Equipment, FHWA-TS-87-208, Federal Highway Administration, McLean, VA, 1987.

Hughes, C.S., NCHRP Synthesis of Highway Practice 346: State Construction Quality Assurance Programs, Transportation Research Board, National Research Council, Washington, D.C., 2005.

Humboldt Mfg. Co. (June 1999). "Report Estimating Dry Density from Soil Stiffness and Moisture Content".

Huseyin Y., Halit, Y., and Serdar, A., "Effects of Cement Type, Water/Cement Ratio and Cement Content on Sea Water Resistance of Concrete", *Building and Environment*, No. 42, pp. 1770–1776, 2007

Infrasense, Inc., "Non-Destructive Measurement of Pavement Layer Thickness*",* California Department of Transportation final report, April 25, 2003.

Infrasense, Inc., "Feasibility of Using Ground Penetrating Radar (GPR) for Pavements, Utilities, and Bridges"*,* South Dakota Department of Transportation, SD2005-05-F, http://www.state.sd.us/Applications/HR19ResearchProjects/Projects 5CSD2005-05_Final_Report.pdf, August 2006.

International Atomic Energy Agency (IAEA), "*Guidebook on non-destructive testing of concrete structures",* Training Course Series No. 17, Vienna, Austria, 2002.

International Atomic Agency (IAEA), *"non-destructive testing for plant life assessment "*Training Course Series No.26, Vienna, Austria, 2005.

Kandhal, P.S., and Mallick, R. B., "Evaluation of Pavement Analyzer for HMA Mix Design", NCAT Report No. 99-4, Auburn, Alabama, 1999

Kenis, William J., and Weijun Wang. (June 1997). "Analysis of Pavement Structural Variability," FHWA-RD-97-072.

Khanum T., Hossain M., Romanoschi, S. A., and Barezinsky R., "Concrete Pavement Design in Kansas Following the Mechanistic-Empirical Pavement Design Guide", *Proceedings of the 2005 Mid-Continent Transportation Research Symposium*, Iowa, 2005

Killingsworth, B. M., "Quality Characteristics for Use with Performance-Related Specifications for Hot Mix Asphalt", Research Results Digest, *National Corporative Highway Research Program*, No. 291, August 2004

Kim, S. H., Tutumluer E., Little D. N., and Kim N., "Effect of Gradation on Nonlinear Stress-Dependent Behavior of a Sandy Flexible Pavement Subgrade," *Journal of Transportation Engineering*, Vol. 133, No. 10, October 1, 2007.

Kimberly E. Kurtis and Paulo Monteiro, Analysis of Durability of Advanced Cementitious Materials for Rigid Pavement Construction in California, Report Prepared for CALIFORNIA DEPARTMENT OF TRANSPORTATION, Pavement Research Center, Institute of Transportation Studies, University of California at Berkeley, FHWA/CA/OR-99/02, April 1999.

Linden, R. N., et. al., "Effect of Compaction on Asphalt Concrete Performance", *Transportation Research Records*, No. 1217, pp. 38-45, 1988.

Manik, A., "Comprehensive Simulation for End-Result Specification Development", Ph.D. thesis, Graduate College, University of Illinois at Urbana–Champaign, 2006.

Manik, A., & Buttlar, W.G. (2006). Monte Carlo Based Simulation for Development of ERS. Accepted for Presentation and Publication at the *10th International Conference on Asphalt Pavements*.

Mallick, R. B., Cooley, A. L. Jr., Teto, M. R., Bradbury, R. L., Peabody D., "An Evaluation of Factors Affecting Permeability of Superpave Designed Pavements", *National Center for Asphalt Technology*, Report 03-02, June 2003

Maser, K. R., "Pavement Characterization Using Ground Penetrating Radar: State of the Art and Current Practice", Nondestructive Testing of Pavements and Backcalculation of Moduli: Third Volume, *ASTM STP 1375,* American Society for Testing and Materials, West Conshohocken, PA, 1999.

Maser, K. R., and Scullion T., "Influence of Asphalt Layering and Surface Treatments on Asphalt and Base Layer Thickness Computations using GPR". TTI Report TX-92-1923-1, September 1992.

Mauplin, G. W. Jr., Diefenderfer, B., K., "Design of a High-Binder High-Modulus Asphalt Mixture", Virginia Transportation Research Council, *Virginia Department of Transportation*, 2006

McCullough, B. F., and Rasmussen, R. O., "Fast Track Paving - Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays", *Federal Highway Administrations*, 1998

Naik, T.R. Chapter 4"*The Break-Off Test Method*", "*Handbook on Nondestructive Testing of Concrete* "CRC Press, Inc., Boca Raton, Fl, 1991, pp .83-100.

National Highway Institute (NHI), "Pavement Deflection Analysis, FHWA-HI-94-021, NHI Course No. 13127, 1994.

Nazarian, S., Baker, M.R., Crain, K., "*Development and Testing of a Seismic Pavement Analyzer*", Report SHRP-H-375, Strategic Highway Research Program National Academy of Sciences, Washington, DC, 1993.

Noureldin, A. S., "New Scenario for Backcalculation of Layer Moduli of Flexible Pavements", TRR 1384, Washington, D.C., 1993.

Noureldin, A. S., "Influence of Stress Levels and Seasonal Variations on In-Sit Pavement Layer Properties", TRR 1448, Washington, D.C., 1994.

Noureldin, A. S., Sharaf, E., Arafa, A., and Al-Sugair, “Rational Selection of Factors of Safety in Reliability Based Design of Flexible Pavements”, TRR 1540, Washington, D.C., 1996.

Noureldin, A. S., “Seasonal Deflections and In Situ Moduli Patterns of Polymer Modified Versus Unmodified Asphalt Pavements”, Proceedings, Mechanical Tests for Bituminous Materials, Lyon, France, March 1997 RILEM, PP 583-590.

Ohio Research Institute for Transportation and the Environment, June 2001, “Evaluation of Soil Stiffness Via Non-Destructive Testing “Final Report LTPP Manual for Falling Weight Deflectometer Measurements Operational Field Guidelines, Version 3.1 August 2000

Patel, A. J. *Development of Fatigue Based Pay Adjustment Factors.* Ph.D. thesis, Graduate College, University of Illinois at Urbana–Champaign, 1996, p. 45.

Pierce, L.M., N.C. Jackson, and J.P. Mahoney, Development and Implementation of a Mechanistic, Empirically Based Overlay Design Procedure for Flexible Pavements In Transportation Research Record 1388, TRB, National Research Council, Washington, D.C. , 1992 .

Puangchit, P., R. G. Hicks, J. E. Wilson, and C. A. Bell. Development of Rational Pay Adjustment Factors for Asphalt Concrete. In *Transportation Research Record 911*, TRB, National Research Council, Washington, D.C., 1983, pp. 70–79.

Rao, Ch., et. al., “Effects of Temperature and Moisture on the Response of Jointed Concrete Pavements”, 7th International Conference on Concrete Pavements, Orlando, FL, September 2001

Richter, C.A., and J.B. Rauhut, SHRP Plans for Nondestructive Deflection Testing in the Development of Pavement Performance Prediction Models” ASTM, STP 1026 A.J. Bush III and G.Y. Baladi, ASTM, Philadelphia, 1989.

Robert Parrillo and Roger Roberts Ph D,Geophysical Survey Systems Inc., Salem, NHIntegration of FWD and GPR, Gary Sanati ,Foundation Mechanics, Inc., El Segundo, CA

Saarenketo, T. and Roimela, P., “Ground Penetrating Radar Technique in Asphalt Pavement Density Quality Control". *Proceedings of the 7th International Conference on Ground Penetrating Radar*, Lawrence, KS, pp. 461-466, May 27-30, 1998.

Stakston, A. D., and Bahia, H., “The Effect of Fine Aggregate Angularity, Asphalt Content and Performance Graded Asphalts on Hot Mix Asphalt Performance”, *Wisconsin Department of Transportation*, WisDOT Highway Research Study 0092-45-98, 2003

Stark, D., and Klieger, P., “Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements”, *Highway Research Record*, No. 441, pp. 33-43, Washington D.C., 1973

Sargand, Shad M., Wasniak, Daniel L., Masada, Teruhisa and Beegle, David. (January 2000). "Evaluation of Initial Subgrade Variability on the Ohio SHRP Test Roads".

Schindler, A.K. (2002). “Concrete Hydration, Temperature Development, and Setting at

Early-Ages," Ph.D. Dissertation, The University of Texas at Austin, Texas.

Schindler, A., K., and McCullough, D. F., "The Importance of Concrete Temperature Control During Concrete Pavement Construction in Hot Weather Conditions", *University of Texas at Austin*, 2002

Sebaaly, P. E., Bazi, G. M., "Impact of Construction Variability on Pavement Performance", *Nevada Department of Transportation*, Nevada, 2004

Selezneva O. I., Jiang Y. J., and Mladenovic G. "Evaluation and Analysis of LTPP Pavement Layer Thickness Data", FHWA-RD-03-041, July 2002.

Seong-Min Kim and B. Frank McCullough, CENTER FOR TRANSPORTATION RESEARCH, THE UNIVERSITY OF TEXAS AT AUSTIN, Project Summary Report 0-4382-S, Project 0-4382: Establish an Acceptable Pavement Thickness Tolerance to Allow for Non-Destructive Continuous Concrete Pavement Thickness Measurements, October 2002, ACCEPTABLE CONCRETE PAVEMENT THICKNESS TOLERANCE

Shell Pavement Design Manual-Asphalt Pavements and Overlays for Road Traffic , Shell International Petroleum Company, Ltd., London, England, 1978 .

Smith, G.R., NCHRP Synthesis of Highway Practice 263: State DOT Management Techniques for Materials and Construction Acceptance, Transportation Research Board, National Research Council, Washington, D.C., 1998, 51 pp.

Tachibana, D., Imai Y., Kawai T., and Inada Y., (1981) High Strength Concrete Incorporating Several Admixtures, ACI SP-121, ed. W.T. Hester, American Concrete Institute, Farmington Hills, Michigan. et al., 1981.

Thompson, M.R. and R.P. Elliot. ILLI-PAVE Based Response Algorithm for Design of Conventional Flexible Pavements. In Transportation Research Record 1043, TRB, National Research Council, Washington, D.C., 1985.

Tran, N. H., Hall, K. D., "Development of Simplified Asphalt Concrete Stiffness-Fatigue Testing Device", *Arkansas State Highway and Arkansas Transportation Department*, 2004

Transportation Circular Number E-C037: Glossary of Quality Assurance Terms, Transportation Research Board, national Research council, Washington, D.C., Apr.2002, pp 29.

Ulriksen C., "Application of Impulse Radar in Civil Engineering", Doctoral Thesis, Department of Engineering Geology, Lund University of Technology, Lund Sweden, Reprinted and distributed by Geophysical Survey Systems Inc., North Salem, New Hampshire, 1982.

Verlag Beuth. (June 1990). "Plate Load Test, DIN 181 34 (Construction Industry standards Committee)".

Voigt, G. F., "Early Cracking Of Concrete Pavement - Causes And Repairs", Federal Aviation Administration Airport Technology Transfer Conference, 2002.

Vivar, E. D. P., Haddock, J., E.,. “HMA Pavement Performance and Durability”. FHWA/IN/JTRP-2005/14, final report, Indiana Department of Transportation. April 2006.

Von Quintus H. L., Simpson A. L. “Structural Factors for Flexible Pavements - Initial Evaluation of the SPS 1 Experiment”, FHWA-RD-01-166, U.S. Department of Transportation, Federal Highway Administration. November 2003.

Weed, R. M. OCPLOT: PC Program to Generate Operating Characteristics Curves for Statistical Construction Specifications. In *Transportation Research Record 1491*, TRB, National Research Council, Washington, D.C., 1996, pp. 18–26.

Wenzlick, J. T. S., and Maser, K. R., “High Accuracy Pavement Thickness Measurement using Ground Penetrating Radar”, Report No. RDT 99-003, Missouri Department of Transportation, February, 1999.

Willenbrock, J. H. *Statistical Quality Control of Highway Construction,* Vol. I and II. FHWA, U.S. Department of Transportation, 1976.

Wimsatt A., Hurlebaus S., Scullion T., and Fernando E., “Promising Existing and Emerging Technologies & Techniques”, Texas Transportation Institute, International Symposium on Nondestructive Testing for Design Evaluation and Construction Inspection, Strategic Highway Research Program 2 (SHRP2), Transportation Research Board, Washington, D.C., January 2008.

WisDOT, “Effect of Pavement Thickness on Superpave Mix Permeability and Density”, WisDOT Highway Research Study 0092-02-14, September 2004.

Yoder E., and M. Witczack, Principles of Pavement Design, 2nd ed., John Wiley & Sons, Inc., New York, 1975.

Yu, H. T. and Khazanovich, L., “Use of Magnetic Tomography Technology to Evaluate Dowel Placement”, Report FHWA-IF-06-006, Federal Highway Administration, Washington, DEC 20590, November 2005

Zaniewski, J. P., Nallamothu, S. H., “Evaluation of Binder Grades on Rutting Performance”, Morgantown, West Virginia, 2003

Appendix A:
MDOT Construction Data Details

APPENDIX A: Details of MDOT Construction Data Gathered

Control Section: **U 33011**

Job Number: **00434 A**

(Microfilm & Box)

- In-Station (Relative to Center-Line) **Actual Depth Measurement** for Base, Selected Subbase, and Subbase Course
- **Gradation** (Three Replicates) for Aggregates of Specification example 22A Class II
- **Specific Gravity at 25/25 C°**, **Penetration at 25 C°, 100 g, 5 Sec.** d_{mm} , **Flash Point** (Cleveland Open Cup), **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight), **Spot Test** (Oliensis), **Viscosity** (Kinemotic, 135 C°, cts), **Loss of Heating** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Penetration of Residue** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Ductility of Residue** at 25 C°, 5 Cm/Min, Cm (Thin Film Oven Test, 1/8", 163 C°, 5 hours) of Asphalt Cement for Bituminous Mix of Specification example 85-100, 1976 Standard Specification
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** d_{mm} , and **Temperature of Mix at Plant** of Bituminous Concrete Mix for Wearing Course (Type M) of Specification example 4.12, 1976 Standard Specification
- **Thickness**, **Absorption** (24 hours, Percent by Volume), **Bitumen** (Percent by Weight), **Resilience and Compression Test** (Recovery, Percent of Original Thickness, Compression Load, Loss of Bitumen (Percent by Weight), Extrusion), and **Density** of Preformed Fiber Joint Filler for Joint Filler
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** d_{mm} , and **Temperature of Mix at Plant** of Bituminous Base Mix for Base Course (Type 20C) of Specification example 3.05, 1976 Standard Specification
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** d_{mm} , and **Temperature of Mix at Plant** of Bituminous Aggregate Mix for Surfacing Course (Type 20A) of Specification example 4.11, 1976 Standard Specification
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** d_{mm} , and **Temperature of Mix at Plant** of Bituminous Concrete Mix for Binder Course (Type 9A) of Specification example 4.12, 1976 Standard Specification
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** d_{mm} , and **Temperature of Mix at Plant** of Bituminous Concrete Mix for Leveling Course (Type 25A) of Specification example 4.12, 1976 Standard Specification

- **Gradation**, and **Free Carbon Content** of Mineral Filler for Bituminous Mix of 1976 Standard Specification
- Daily **Aggregate Gradation**, and **Inspection of Crushed Material**, **Thin or Elongated Pieces**, and **Soft Particles** of Specification example 22A (Sampled from Stockpile, Job Site)
- **Gradation** of Granular Material Class I & II (Sampled from In-Place Materials)
- Daily **Aggregate Gradation**, and **Inspection of Crushed Material**, **Thin or Elongated Pieces**, and **Soft Particles** of Specification example 25A (Sampled from Stockpile, and Truck)
- Daily **Aggregate Gradation**, **Inspection of Crushed Material**, **Thin or Elongated Pieces**, **Incrusted Particles Less than 1/3 Area**, **Incrusted Particles More than 1/3 Area**, and **Soft Particles** of Specification example 25A (Sampled from Truck)
- **Gradation** of Granular Materials Class III (Sampled from In-Place Materials)
- Daily **Gradation** of Different Bins of Wearing Course Materials for Bituminous Plant (Sampled from Hot Bin)
- Daily **Gradation** of Different Bins of Bituminous Base Course Materials for Bituminous Plant (Sampled from Hot Bin)
- Daily **Gradation** of Different Bins of Binder Materials for Bituminous Plant (Sampled from Hot Bin)
- Daily **Gradation** of Different Bins of Leveling Course Materials for Bituminous Plant (Sampled from Hot Bin)
- **Gradation** of Edge Drain Backfill Material (Sampled from Trench, Stockpile, In-Place Material)
- Daily **Gradation** of Extracted Aggregate (Sampled from In-Place Material)
- Daily **Gradation** of Hot Aggregate (Fine and Coarse) Bin (Sampled from Plant)

Control Section: **U 33011**

Job Number: **00434 A**

(Microfilm & Box)

Actual Depth Measurement

- Base Course
- Subbase Course

Gradation

- Aggregates, sampled from Stockpile, In-Place Material, Truck, Bins and Extracted Aggregates
- Bituminous Aggregate Mix for Surfacing Course
- Bituminous Base Mix for Base Course
- Bituminous Concrete Mix for Binder Course
- Bituminous Concrete Mix for Leveling Course
- Bituminous Concrete Mix for Wearing Course
- Edge Drain Backfill, sampled from In-Place Material, Truck, Stockpile
- Mineral Filler for Bituminous Mix

Specific Gravity

- Asphalt Cement for Bituminous Mix

Penetration

- Asphalt Cement for Bituminous Mix
- Bituminous Aggregate Mix for Surfacing Course
- Bituminous Base Mix for Base Course
- Bituminous Concrete Mix for Binder Course
- Bituminous Concrete Mix for Leveling Course
- Bituminous Concrete Mix for Wearing Course

Flash Point

- Asphalt Cement for Bituminous Mix

Ductility

- Asphalt Cement for Bituminous Mix

Solubility in Trichloroethylene

- Asphalt Cement for Bituminous Mix

Spot Test

- Asphalt Cement for Bituminous Mix

Viscosity

- Asphalt Cement for Bituminous Mix

Loss of Heating (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Penetration of Residue (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Ductility of Residue (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Temperature of Mix

- Bituminous Aggregate Mix for Surfacing Course
- Bituminous Base Mix for Base Course
- Bituminous Concrete Mix for Binder Course
- Bituminous Concrete Mix for Leveling Course
- Bituminous Concrete Mix for Wearing Course

Free Carbon Content

- Mineral Filler for Bituminous Mix

Crushed Material

- Aggregates sampled from Stockpile, Truck

Thin or Elongated Piece

- Aggregates sampled from Stockpile

Soft Particle

- Aggregates sampled from Stockpile

Incrusted Particles Less than 1/3 Area

- Aggregates sampled from Truck

Incrusted Particles More than 1/3 Area

- Aggregates sampled from Truck

Control Section: **ACF U 41051**

Job Number: **25745 A**

(Box)

- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Base Mix for Base Course of Specification example 4.00 Mod, 1984 Standard Specification for Supperpave Material
- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Top Mix for Bituminous Top Mix of Specification example 4.00, 1984 Standard Specification
- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Mixture Recycled for Leveling Course (Special Blend) of Specification example 4.00, 1984 Standard Specification for Supperpave Material
- **Specific Gravity at 25/25 C°**, **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$, **Viscosity** (Absolute, 60 C°, Poises) of Asphalt Cement for Bituminous Mix of Specification example AC-2.5, 1984 Standard Specification
- **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$, **Flash Point** (Cleveland Open Cup), **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight), **Spot Test** (35% Xylene - 65% Heptane), **Viscosity** (Kinemotic, 135 C°, cts), **Viscosity** (Absolute, 60 C°, Poises), **Loss of Heating** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Penetration of Residue** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Ductility of Residue** at 25 C°, 5 Cm/Min, Cm (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Viscosity** (Absolute) 60 C° Poises (Thin Film Oven Test, 1/8", 163 C°, 5 hours) of Asphalt Cement for Bituminous Mix of Specification example AC-10, 1990 Standard Specification (Sampled from Contractor's Storage)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$ (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Mix for Top Course (Special Blend) of Specification example 4.00 Mod, 1984 Standard Specification for Supperpave Material (Sampled from Trucks)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$ (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Mix for Leveling Course (Special Blend + RAP) of Specification example 4.00 Mod, 1984 Standard Specification for Supperpave Material (Sampled from Trucks)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$ (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Mix for Base Course (Special Blend) of Specification example 4.00 Mod, 1984 Standard Specification for Supperpave Material (Sampled from Trucks)

- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Mix for Base Course (20C Blend) of Specification example 4.00 Mod, 1984 Standard Specification for Supperpave Material (Sampled from Trucks)
- **Gradation** of Granular Material Class II (Sampled from In-Place Materials, Belt Line, Pits)
- **Gradation** (Dense-Graded Aggregate), and **Percent of Crushed Material** of Dense-Graded Aggregate for Aggregate Base Course of Specification example 22A, 1984 Standard Specification (Sampled from Stockpile at Pit)
- Daily **Target Gradation** of Mixture and Aggregate (Sampled from Job Site)
- Daily **Gradation** (and Deviation from Job Mix Formula) of Bituminous Mix for Top Course (Sampled from In-Place Material)
- Daily **Gradation** (and Deviation from Job Mix Formula) of Bituminous Mix for Leveling Course (Sampled from In-Place Material)
- Daily **Gradation** (and Deviation from Job Mix Formula) of Bituminous Mix for Base Course (Sampled from In-Place Material, Belt Line)
- Daily **Gradation** and **Percent of Crushed Material** of Aggregate for Bituminous Mixture (Sampled from Belt Line); Tested by Distribution Laboratory and Plant Inspector

Available Data from Mix Design

- **Gradation** (Coarse Aggregate [P3/4 - R3/8], Fine Aggregate [P3/8], Sand [#4], and Dense-Graded Aggregate [-3/8 Crushed]), **Asphalt Content** (Type AC-10), **Density**, **Optimum Asphalt Content**, **Specific Gravity**, **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Concrete for Top Course of Specification example 4.00, 1984 Standard Specification
- **Gradation** (Coarse Aggregate [5/8 - 3/8], Fine Aggregate [-3/8 CR], Sand [Washed], and Dense-Graded Aggregate [-3/8 NAT]), **Asphalt Content** (Type AC-10), **Density**, **Optimum Asphalt Content**, **Specific Gravity**, **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Concrete for Leveling Course of Specification example 4.00, 1984 & 1990 Standard Specification
- **Gradation** (Dense-Graded Aggregate [Type 20C], Stone [RT #1 – 5/8 CR], Sand [Washed - #4 CR], Mod [20C]), **Asphalt Content** (Type AC-10), **Density**, **Optimum Asphalt Content**, **Specific Gravity**, **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Concrete for Base Course (20C Modified) of Specification example 4.00, 1984 Standard Specification

Control Section: **ACF U 41051**

Job Number: **25745 A**

(Box)

The following list shows the types of test historically performed during highway construction followed by the source of specimen for those tests.

Gradation

- Aggregates sampled from Stockpile, In-Place Material, Belt Line, Truck, Bins and Extracted Aggregates
- Bituminous Base Mix for Base Course
- Bituminous Mix for Leveling Course
- Bituminous Mix Recycled for Leveling Course
- Bituminous Top Mix
- Edge Drain Backfill, sampled from In-Place Material, Truck, Stockpile

Specific Gravity (Sampled from Truck)

- Aggregates
- Asphalt Cement for Bituminous Mix
- Mix Design

Penetration (Virgin AC/Mix Sampled from Truck)

- Asphalt Cement for Bituminous Mix (Virgin AC)
- Bituminous Mix for Base Course
- Bituminous Mix for Leveling Course
- Bituminous Mix for Top Course

Flash Point (Virgin AC)

- Asphalt Cement for Bituminous Mix

Ductility (Virgin AC)

- Asphalt Cement for Bituminous Mix

Solubility in Trichloroethylene (Virgin AC)

- Asphalt Cement for Bituminous Mix

Spot Test (Virgin AC)

- Asphalt Cement for Bituminous Mix

Viscosity (Virgin AC/ Sampled from Truck)

- Asphalt Cement for Bituminous Mix
- Bituminous Mix for Base Course

Loss of Heating (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Penetration of Residue (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Ductility of Residue (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Temperature of Mix (Mix at Plant)

- Bituminous Mix for Base Course
- Bituminous Mix for Leveling Course
- Bituminous Mix for Top Course

Crushed Material

- Aggregates sampled from Stockpile

Moisture Content

- Aggregates, sampled from Bins

Wear Index

- Coarse Aggregates for Bituminous Mix, sampled from Plant Stockpile

Data available from Mix Design

- Air Voids
- Asphalt at Optimum
- Density
- Flow
- Specific Gravity
- Stability
- Voids in Fine Aggregates
- Voids in Mineral Aggregates

Control Section: **IM 33083**

Job Number: **29581 A**

(Microfilm)

- **Specific Gravity at 25/25 C°**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Viscosity** (Absolute, 60 C°, Poises), **Flash Point** (Cleveland Open Cup), **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight), **Spot Test** (35% Xylene – 65% Heptane), **Viscosity** (Kinemotic, 135 C°, cts), **Loss of Heating** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Penetration of Residue** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Ductility of Residue** at 25 C°, 5 Cm/Min, Cm (Thin Film Oven Test, 1/8", 163 C°, 5 hours), and **Viscosity** (Absolute) 60 C° Poises (Thin Film Oven Test, 1/8", 163 C°, 5 hours) of Asphalt Cement for Bituminous Mix of Specification for Penetration Grade 85-100, 1990 Standard Specification (Sampled from Contractor's Storage)
- Petrographic Determination of **Wear Index** of Bituminous Aggregate for Bituminous Top Mixture of Specification example 11A, 1990 Standard Specification for Supperpave Material
- **Gradation**, and **Free Carbon Content** of Mineral Filler (Fly Ash) for Bituminous Mix of Specification example 3MF, 1990 Standard Specification (Sampled from Contractor's Storage)
- **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Penetration at 4 C°, 200 g, 60 Sec. d_{mm}** , **Softening Point** (ASTM, Ring and Ball, C°), **Viscosity** (Poises, 60 C°, #200 Koppers), and **Elastic Recovery** (10 Cm, 5Cm/Min, 25 C°) of Polymer Modified Asphalt Cement for Bituminous Mixture of Special Provision for Polymer Modified Asphalt Cement
- **Gradation** and **Percent of Crushed Material** of Aggregate (34R) for Bituminous Mixture (Sampled from Stockpile at Pit, Job Site)
- **Gradation** and **Percent of Crushed Material** of Aggregate (22A) for Bituminous Mixture (Sampled from Stockpile at Pit, R&D)
- **Gradation** and **Percent of Crushed Material** of Aggregate (22A Mod.) for Bituminous Mixture (Sampled from Stockpile at Pit)
- **Gradation** and **Percent of Crushed Material** of Aggregate (23 Mod.) for Bituminous Mixture (Sampled from Stockpile at Pit)
- **Gradation** and **Percent of Crushed Material** of Aggregate (23A Mod.) for Bituminous Mixture (Sampled from Stockpile at Pit)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Viscosity** (60C°, Poises) of Bituminous Concrete Mix (No. 2C Mod. Rubber)

for Base Course (Blend) of Specification example 4.00 Mod, 1990 Standard Specification for Superpave Material (Sampled from Truck)

- **Gradation** (Laboratory and Plant Inspectors Results), **Marshall Density**, **Theoretical Maximum Density**, and **Average Core Density** of Bituminous Mix (No. 2C, and 3C) for Leveling Course (Blend) of Specification example 4.00, 1990 Standard Specification for Supperpave Material
- **Gradation** (Laboratory and Plant Inspectors Results), **Marshall Density**, and **Theoretical Maximum Density** of Bituminous Mix (No. 2C) for Base Course of Specification example 4.00, 1990 Standard Specification for Supperpave Material
- **Gradation** (Laboratory and Plant Inspectors Results), **Marshall Density**, and **Theoretical Maximum Density** of Bituminous Mix (No. 13) for Top Course of Specification example 4.00, 1990 Standard Specification
- Daily **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$ of Original Asphalt Cement (Mix No. 13A, Penetration Grade 200-250) for Leveling Course
- Daily **Penetration at 25 C°, 100 g, 5 Sec.** $\mathbf{d_{mm}}$ of Recovered Asphalt Cement (Mix No. 13A, Penetration Grade 200-250) for Leveling Course
- **Gradation** of Granular Material Class II for Bituminous Mixture (Sampled from Stockpile at Job Site, Pit)
- **Gradation**, **Percent Air Voids**, **Marshall Density**, **Theoretical Maximum Density**, **Percent Filler**, **V. M. A**, and **Asphalt Content** of Bituminous Mix (No. 11A, 13, 13A, 2C and 3C); Quality Assurance Test
- **Gradation**, **Percent Air Voids**, **Marshall Density**, **Theoretical Maximum Density**, and **V. M. A** of Bituminous Mixture (No. 11A, 13, 13A, 2C, 3C, 4C and 4C Mod) and Aggregate (Sampled from Plant); Contractor's Quality Control Test
- **Gradation**, **Percent Air Voids**, **Marshall Density**, **Theoretical Maximum Density**, and **V. M. A** of Bituminous Mixture (No. 11A, 13A, 2C, 2C Mod, 2C Mod. [Rubber], 3C and 4C Mod) and Aggregate (Sampled from Plant); Verification/Acceptance Testing and Core Density
- **Summary of Bituminous Field and Laboratory Test Results**

Available Data from Mix Design

- **Gradation** (Coarse Aggregate [013, 053, 051, 052], and Dense-Graded Aggregate 054, 439]), **Asphalt Content** (Type 85-100), **Theoretical Maximum Density**, **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix of Specification example 4.00, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [411, 051], Fine Aggregate [054], and Dense-Graded Aggregate [013]), **Asphalt Content** (Type 200-250), **Theoretical Maximum Density**, **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and

Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 11A) of Specification example 4.00 Mod, 1990 Standard Specification

- **Gradation** (Coarse Aggregate [1/2 x 3/8, 3/8 x 4], Fine Aggregate [5/16 Sand, MFG Sand, and Bag House Fine], and Mineral Filler [Flyash]), **Asphalt Content** (Type 85-100), **Density** (Theoretical Maximum, and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 4C) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [051, 082], and Dense-Graded Aggregate [013, 053, 054]), **Asphalt Content** (Type 85-100), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 2C) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [053, 051], and Dense-Graded Aggregate [013, 054]), **Asphalt Content** (Type 120-150), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 13A) of Specification example 4.00 Mod, 1990 Standard Specification

Control Sction: **IM 33083**

Job Number: **29581 A**

(Microfilm)

Gradation

- Aggregates, sampled from Stockpile, In-Place Material, Truck, Bins and Extracted Aggregates
- Bituminous Concrete Mix for Leveling Course
- Bituminous Mix for Base Course
- Bituminous Mix for Top Course
- Mineral Filler (Fly Ash) for Bituminous Mix

Specific Gravity

- Asphalt Cement for Bituminous Mix

Penetration

- Asphalt Cement for Bituminous Mix
- Bituminous Mix for Base Course
- Polymer Modified Asphalt Cement for Bituminous Mix

Flash Point

- Asphalt Cement for Bituminous Mix

Ductility

- Asphalt Cement for Bituminous Mix

Solubility in Trichloroethylene

- Asphalt Cement for Bituminous Mix

Spot Test

- Asphalt Cement for Bituminous Mix

Viscosity

- Asphalt Cement for Bituminous Mix
- Bituminous Mix for Base Course
- Polymer Modified Asphalt Cement for Bituminous Mix

Loss of Heating (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Penetration of Residue (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Ductility of Residue (Thin Film Oven Test)

- Asphalt Cement for Bituminous Mix

Free Carbon Content

- Mineral Filler (Fly Ash) for Bituminous Mix

Crushed Material

- Aggregates sampled from Stockpile, Truck

Wear Index Petrographic Determination

- Bituminous Aggregate for Bituminous Top Mix, sampled from Stockpile

Softening Point

- Polymer Modified Asphalt Cement for Bituminous Mix

Elastic Recovery

- Polymer Modified Asphalt Cement for Bituminous Mix

Marshall Density

- Bituminous Mix for Base Course
- Bituminous Mix for Leveling Course
- Bituminous Mix for Top Course

Theoretical Maximum Density

- Bituminous Mix for Base Course
- Bituminous Mix for Leveling Course
- Bituminous Mix for Top Course

Average Core Density

- Bituminous Mix for Base Course
- Bituminous Mix for Leveling Course
- Bituminous Mix for Top Course

Air Voids

- Bituminous Mix

Voids in Mineral Aggregate

- Bituminous Mix

<u>Data available from Mix Design</u>

- Air Voids
- Asphalt at Optimum
- Density (Bulk)
- Flow
- Specific Gravity (Actual, Bulk & Maximum)
- Stability
- Voids in Fine Aggregates
- Voids in Mineral Aggregates

Control Section: **FR 23092**

Job Number: **10729 A**

(Microfilm)

- **Gradation** (Laboratory and Plant Inspectors Results), **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Base Mix for Base Course (Type 20C, End Result) of Specification example 3.05 Mod, 1976 Standard Specification for Superpave Material (Sampled from Truck)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Concrete Mix for Leveling Course (End Result) of Specification example 4.12 Mod, 1976 Standard Specification for Superpave Material (Sampled from Truck)
- **Gradation**, and **Free Carbon Content** of Mineral Filler (Fly Ash) for Bituminous Mix of Specification example 3MF, 1976 Standard Specification (Sampled from Contractor's Storage)
- **Specific Gravity at 25/25 C°**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Viscosity** (Absolute, 60 C°, Poises), **Flash Point** (Cleveland Open Cup), **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight), **Spot Test** (Oliensis), **Viscosity** (Kinemotic, 135 C°, cts), **Loss of Heating** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Penetration of Residue** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Ductility of Residue** at 25 C°, 5 Cm/Min, Cm (Thin Film Oven Test, 1/8", 163 C°, 5 hours), and **Viscosity** (Absolute) 60 C° Poises (Thin Film Oven Test, 1/8", 163 C°, 5 hours) of Asphalt Cement for Bituminous Mix of Specification example 120-150, 1976 Standard Specification (Sampled from Contractor's Storage, Tanks)
- **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Viscosity** (Absolute, 60 C°, Poises) of Asphalt Cement for Bituminous Mix of Specification example AC-5, 1976 Standard Specification for Superpave (Sampled from Contractor's Storage)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Aggregate Mix for Surfacing Course (End Result) of Specification example 4.11 Mod, 1976 Standard Specification for Superpave Material (Sampled from Trucks)
- **Specific Gravity at 25/25 C°**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Penetration at 46.1 C°, 50 g, 5 Sec. d_{mm}** , **Penetration at 0 C°, 200 g, 1 Min. d_{mm}** , **Flash Point** (Cleveland Open Cup), **Softening Point** (Ring and Ball, C°), **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight),), **Loss of Heating** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Penetration of Residue** (Thin Film Oven Test, 1/8", 163 C°, 5 hours) of Asphalt

Cement for Membrane Waterproofing of Specification example WOA, 1976 Standard Specification (Sampled from Job Site)

- **Gradation** (Laboratory and Plant Inspectors Results), of Bituminous Concrete Mixture for Wearing Course (Type C, End Result) of Specification example 4.12 Mod, 1976 Standard Specification for Superpave Material
- **Gradation** (Hand Washed and Hand Sieved), and **Percent of Crushed Material** of Dense-Graded Aggregate for Aggregate Base Course of 1976 Standard Specification for Superpave Material (Sampled from In-Place Material)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Shoulder Mix for Shoulder Course (20A) of Specification example 4.25, 1976 Standard Specification (Sampled from Trucks)
- **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** of Asphalt Cement for Bituminous Mix of Specification example 85-100, 1976 Standard Specification (Sampled from Contractor's Storage)
- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Shoulder Mix for Shoulder Course (20A) of Specification example 4.25, 1976 Standard Specification for Supperpave Material
- **Specific Gravity at 15.6/15.6 C°**, **Flash Point** (Cleveland Open Cup), **Viscosity** (Kinemotic, 135 C°, cts), Distillation Test (To 190 C°, 225 C°, 260 C°, 315.5 C°), **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight), **Spot Test** (Oliensis) of Liquid Asphalt for Waterproofing Primer of Specification example RC-250, 1976 Standard Specification (Sampled from Job Site)
- **Gradation**, **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered), and **Temperature of Mix at Plant** of Bituminous Aggregate Mix for Shoulder Course (End Result) of Specification example 4.25 Mod, 1976 Standard Specification for Superpave Material (Sampled from Trucks)
- **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** , **Viscosity** (Kinemotic, 60 C°, Poises) of Asphalt Cement for Bituminous Mix of Specification example AC-10, 1976 Standard Specification for Superpave (Sampled from Contractor's Storage)
- Daily **Aggregate Gradation**, **Inspection of Crushed Material**, **Thin or Elongated Pieces**, **Incrusted Particles Less than 1/3 Area**, **Incrusted Particles More than 1/3 Area**, **Soft Particles**, **Chert**, and **Hard Absorbent Particles** of Specification example 22A (Sampled from Stockpile)
- **Aggregate Gradation** of Granular Material Class I & II and Specification example 23A (Sampled from In-Place Materials)
- Daily **Aggregate Gradation**, **Inspection of Crushed Material**, and **Fineness Modulus** of Specification example 2NS (Sampled from Stockpile)

- Daily **Aggregate Gradation**, **Thin or Elongated Pieces**, **Incrusted Particles Less than 1/3 Area**, **Incrusted Particles More than 1/3 Area**, and **Soft Particles**, and **Chert** of Specification example 6A (Sampled from Stockpile)
- Daily **Gradation** of Bituminous Plant (Sampled from Stone, Sand, and Bitumen Bin)
- In-Place (Station Relative to Center-Line) **Actual Depth Measurement** for Base, Selected Subbase, and Subbase Course

<u>Available Data from Mix Design</u>

- **Gradation** (Coarse Aggregate [Type 25A], Fine Aggregate [Type 3CS], and Extracted Aggregate), **Asphalt Content** (Type 85-100), **Marshall Density**, **Optimum Asphalt Content**, **Specific Gravity**, **Stability**, **Air Voids**, **Voids in Mineral Aggregate**, **Flow**, and **Voids filled with Asphalt** of Bituminous Concrete for Wearing Course (Type C) of Specification example 4.12 Mod, 1976 Standard Specification for Superpave Material
- **Gradation** (Dense-Graded Aggregate Type 20A), **Asphalt Content** (Type 120-100), **Marshall Density**, **Theoretical Maximum Specific Gravity**, **Stability**, **VFA**, **VMA**, and **Flow** of Bituminous Aggregate for Surfacing Course (Type 20A) of Specification example 4.11, 1976 Standard Specification

Control Section: **NH 19033**

Job Number: **20046 A**

(Microfilm)

- **Summary of Bituminous Field and Laboratory Test Results**
- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Mixture (No. 11A) for Base Course (Blend) of Specification example 4.00 Mod, 1990 Standard Specification for Superpave Material
- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Mixture (No. 11A - Recycled) for Base Course (Blend + RAP) of Specification example 4.00 Mod, 1990 Standard Specification for Superpave Material
- **Gradation**, and **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** (Original and Recovered) of Bituminous Mixture (No. BTM) of Specification example 4.00, 1990 Standard Specification for Supperpave Material (Sampled from Trucks)
- Daily **Penetration at 25 C°, 100 g, 5 Sec. d_{mm}** of Original Asphalt Cement (Mix No. 3C, Penetration Grade 120-150) for Leveling Course
- **Gradation** (Laboratory and Plant Inspectors Results) of Bituminous Mixture (No. PATB) for Asphalt Treated Base of Specification example 4.00 Mod, 1990 Standard Specification for Superpave Material
- **Specific Gravity at 25/25 C°**, **Penetration at 25 C° and 4 C°, 100 g, 5 Sec. d_{mm}** , **4° Penetration, 100g, 5 Sec**, **Viscosity** (Absolute, 60 C°, Poises), **Flash Point** (Cleveland Open Cup), **Ductility at 25 C°, 5 Cm/Min, Cm**, **Solubility in Trichloroethylene** (Percent by Weight), **Spot Test** (35% Xylene – 65% Heptane), **Viscosity** (Kinemotic, 135 C°, cts), **Softening Point** (Ring & Ball, C°), **Loss of Heating** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Penetration of Residue** (Thin Film Oven Test, 1/8", 163 C°, 5 hours), **Ductility of Residue** at 25 C°, 5 Cm/Min, Cm (Thin Film Oven Test, 1/8", 163 C°, 5 hours), and **Viscosity** (Absolute) 60 C° Poises (Thin Film Oven Test, 1/8", 163 C°, 5 hours) of Asphalt Cement for Bituminous Mix of Specification example 120-150 and 85-100, 1990 Standard Specification (Sampled from Storage Tank)
- **Gradation**, **Percent Air Voids**, **Marshall Density**, **Theoretical Maximum Density**, and **V. M. A** of Bituminous Mixture (No. 2C, New 2C, 3C, 4B, and 4C Mod) and Aggregate (Sampled from Plant); Contractor's Quality Control Test
- **Gradation**, **Percent Air Voids**, **Marshall Density**, and **Theoretical Maximum Density** of Bituminous Mixture (No. 2C, 3C, and 4C Mod) and Aggregate (Sampled from Plant); Verification/Acceptance Testing and Core Density
- Daily **Penetration at 4 C°, 200 g, 60 Sec. d_{mm}** of Original Asphalt Cement (Mix No. 4C, Penetration Grade 120-150 [Polymer Modified]) for Top Course

- **Gradation** (Laboratory and Plant Inspectors Results), **Marshall Density**, **Theoretical Maximum Density**, and **Average Core Density** of Bituminous Mix (No. 2C) of Specification example 4.00 Mod, 1990 Standard Specification for Supperpave Material
- Petrographic Determination of **Wear Index** of Bituminous Aggregate (1/2" x 0" Crushed, 013 - 4C, and Blend) for Bituminous Top Mixture of Specification example 4C, 1990 Standard Specification for Supperpave Material (Sampled from Stockpiles)
- **Specific Gravity** (Bulk and Apparent), and percent **Absorption** of Coarse Aggregate for Permeable Base of example 6A, 1990 Standard Specification, ASTM C127; Laboratory Test
- **Specific Gravity** (Bulk and Apparent), and percent **Absorption** of Open-Graded Aggregate for Permeable Base of example 3G, 1990 Standard Specification, ASTM C127; Laboratory Test
- **Gradation** of Aggregate (22A, 23A, 6AA, 6A Mod, 2NS) for Bituminous Mixture (Sampled from Pit, Stockpile at Pit and at Plant)
- **Gradation** and **Percent of Crushed Material** of Aggregate (21AA, 22A, 3G, 34R, 3Gm1, 3GM2, and 34G Mod) for Bituminous Mixture (Sampled from Stockpile on Job Site and at Pit)
- Daily **Target Gradation** of Mixture (Type 11A, 3B Recycled) and Aggregate (Sampled from Job Site)
- Daily Inspection of Bituminous Plant

<u>Available Data from Mix Design</u>

- **Gradation** (Aggregate Material: #407 [1*1-1/2"], #443 [Peastone], # 441 [1/2-3/4], #408 [1/2-1], #423 [1/2 Sand], #439 [3/8*0], and Mineral Filler [3MF]), **Asphalt Content** (Type 85-100), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 11A) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [1/2 x 3/4], Fine Aggregate [1/2 Sand, and CR. Sand], and Dense-Graded Aggregate [Peastone]), **Asphalt Content** (Type 85-100), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 3B - Recycled) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [1*1-1/2], Fine Aggregate [1/2 Sand], Mineral Filler [3MF], and Dense-Graded Aggregate [Peastone, and Crushed Sand]), **Asphalt Content** (Type 85-100), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical),

Stability, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 11A) of Specification example 4.00 Mod, 1990 Standard Specification

- **Gradation** (Coarse Aggregate [051, 052, 053, and 082], Fine Aggregate [054, 439 and Bag House Fines]), **Asphalt Content** (AC-5), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 2C) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [052, and 053], Fine Aggregate [054], and Dense-Graded Aggregate [013, and 439]), **Asphalt Content** (Type 120-150), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 4C) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [080, and 443], Fine Aggregate [054, and 439], and Dense-Graded Aggregate [013]), **Asphalt Content** (Type 120-150), **Density** (Theoretical Maximum and Bulk), **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 4B) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [3/4", 6A, and Roof-Stone], Fine-Graded Aggregate [Crush Dust, and Bird Pea], and Dense-Graded Aggregate [1/2" Sand]), **Asphalt Content** (Type 120-150), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. BTM) of Specification example 4.00 Mod, 1990 Standard Specification
- **Gradation** (Coarse Aggregate [051, 053, and 080], Fine Aggregate [Bag House Fines]), Dense-Graded Aggregate [054, and 419]), **Asphalt Content** (120-150), **Density** (Theoretical Maximum and Bulk), **Optimum Asphalt Content**, **Specific Gravity** (Bulk, and Max Theoretical), **Stability**, **Air Voids**, **V. M. A**, **Flow**, and **V. F. A.** of Bituminous Mix (No. 3C) of Specification example 4.00 Mod, 1990 Standard Specification

Control Section: **NHI 47065**

Job Number: **28215 A**

(Microfilm)

- **Compressive Strength** (Laboratory Results) of Concrete Pavement Cores for Concrete Shoulder of Specification example ASTM-C42, 1996 Standard Specification (Sampled from Shoulder)
- **Concrete Cylinder Compressive Test Results**

Control Section: **IM 11017**

Job Number: **32516 A**

(Microfilm)

- **Compressive Strength** (Laboratory Results) of Concrete Pavement Cores for Concrete Pavement of Specification example ASTM-C42, 1990 Standard Specification (Sampled from Pavement)
- **Compressive Strength** (Laboratory Results) of Concrete Pavement Cores for Concrete Ramp of Specification example ASTM-C42, 1990 Standard Specification (Sampled from Ramp)
- **Gradation** of Aggregate (6AA, 2NS, 23A, and 26A) for Concrete Mixture (Sampled from Project Concrete Plant, Stockpile at Job Site, Stockpile at Pit)
- **Gradation**, **Percent of Crushed Material**, **Soft Particles**, and **Chert** of Aggregate (3G) for Concrete Mixture (Sampled from Job Site)
- **Gradation** and **Percent of Crushed Material** of Aggregate (34R, and 22A) for Concrete Mixture (Sampled from Stockpile at Pit)

Control Section: **IM 63191**

Job Number: **36003 A**

(Microfilm)

- **Compressive Strength** (Laboratory Results) of Concrete Pavement Cores for Concrete Pavement of Specification example ASTM-C42, 1990 Standard Specification (Sampled from Pavement)
- **Compressive Strength** (Laboratory Results) of Concrete Pavement Cores for Concrete Shoulder of Specification example ASTM-C42, 1990 Standard Specification (Sampled from Shoulder)
- **Concrete Cylinder Compressive Test Results**
- **Gradation**, and **Percent of Crushed Material** of Aggregate (3GM1) for Concrete Mixture (Sampled from Job Site on Different Stations)

Appendix B:
Review of Different Non-Destructive Tests

Literature Review of NDT Tests

1. Thickness of pavement layers

1.1. Introduction

Pavement layer thickness is an important factor in determining the quality of newly constructed pavements and overlays, since deficiencies in thickness reduce the life of the pavement. For asphalt, the relationships between thickness deficiency and pavement life have been quantified using a performance model (1). These relationships show, for example, that a 13 mm (0.5 inch) thickness deficiency on a nominally 91 mm (3.6 inch) thick pavement can lead to a 40 % reduction in pavement life. This reduction in pavement life has significant economic implications.

In order to implement pavement thickness as a measure of quality assurance, it is necessary to have an accurate and reliable method for making the thickness measurement. Cores are accurate, but they are time consuming, they damage the pavement, and they represent a very small sample of the actual pavement. Therefore, it is desirable to have a thickness measuring method which is quick, non-destructive, and which can generate an accurate and representative population of pavement thickness data points.

GPR is a high resolution geophysical technique that utilizes electromagnetic radar waves to scan shallow subsurface, provide information on pavement layer thickness or locate targets (2 – 5). Frequency of GPR antenna affects depth of penetration (2 – 5). Lower frequency antennas penetrate further, but higher frequency antennas yield higher resolution. To successfully provide pavement thickness information or scan an interface, the following conditions have to be present (2 – 5);

- The physical properties of the pavement layers must allow for penetration of the radar wave.
- The interface between pavement layers must reflect the radar wave with sufficient energy to be recorded.
- The difference in physical properties between layers separated by interfaces must be significant.

Physical (electrical) properties of pavement layers, thickness of pavement layers, and magnitude of difference between electrical properties of successive pavement layers impact the ability to detect thickness information using GPR (2 – 5). Depth of penetration of radar wave into a pavement layer depends on electrical properties of that layer. Radar wave will penetrate much deeper in an electric resistive layer than in an electric conductive layer. Layers with similar physical properties will be detected as one layer (2 – 5).

Conductive losses occur when electromagnetic energy is transformed into thermal energy to provide for transport of charge carriers through a specific medium. Presence of moisture or clay content in a pavement layer will cause significant conductive losses and hence will increase the dielectric permittivity and decrease depth of penetration (2 – 5).

For asphalt pavement, ground-penetrating radar (GPR) is by far the most established technology for measuring pavement thickness. Evaluation studies have been carried out by over ten state highway agencies, by SHRP, MnROAD, and by the FHWA, all of which have documented the accuracy of GPR asphalt thickness vs. core samples (6)(7). The studies have generally compared the GPR results to cores, and have shown differences that range from 2- 10%. The lower

differences (2-5%) are generally associated with newly constructed pavements, while the bigger differences are generally associated with older pavements (8). In general, where there are large deviations between GPR and core values, the GPR gave the larger values, and the difference appeared to be due to portions of the core that remained in the hole (9). Studies have also shown, that with proper equipment and data processing, GPR can accurately determine thickness for overlays as thin as 25 mm (1 inch) (10). GPR can be collected continuously at various speeds, and thus allowing for the availability of a large number of thickness data points to be collected economically. Finally, GPR has also been effectively used to determine variations in asphalt density (11). Such additional information would enhance the overall quality assurance program. Most of these GPR layer thickness studies have been carried out with air-coupled horn antennas, since these can be implemented at driving speed without lane closures. However, for the purposes of quality assurance, lower data collection speeds permit consideration of ground-coupled antennas as well.
For concrete pavement, the situation is different. The GPR wave attenuates more rapidly in concrete, especially new concrete, than it does in asphalt (12). This is due to the free moisture and conductive salts that are present in the concrete mix. Also, the dielectric constant between concrete and base is much smaller than it is between asphalt and base. These two factors in combination often lead to a diminished, sometimes absent, reflection at the base of the concrete. Therefore, air-coupled GPR is not a feasible technology for thickness measurement on new concrete. Ground-coupled GPR, on the other hand, provides more energy input into the pavement, and can overcome some of the penetration limitations of the horn antenna.
Mechanical wave techniques (Impact-echo and others), on the other hand, work much more effectively than GPR in concrete. Concrete pavements are typically thick enough to fall within the measurement range of mechanical wave measurements. Mechanical waves travel well in concrete, and there is usually a strong mechanical contrast between the concrete and the base material. Data collection is considerably slower than with GPR, but certainly faster and less expensive than coring.

1.2. Description of the Non-Destructive Test (NDT) Methods for Evaluating Pavement Thickness

1. Electromagnetic Wave Methods (Ground Penetrating Radar)
2. Mechanical Wave Methods (Impact-Echo and others)

1.2.1. Ground Penetrating Radar Methods

Ground Penetrating Radar (GPR) operates using short electromagnetic pulses radiated by an antenna which transmits these pulses and receives reflected returns from the pavement layers. Analysis of these reflected return signals yields information on the pavement layer thickness and electromagnetic material properties. Pavement thickness is calculated from the arrival time of the GPR reflection from the bottom of the pavement and the velocity of travel. The determination of the arrival time is made directly from the GPR signal. The velocity calculation requires some other process, as discussed in the specific methods below. The velocity is related to a material property called the *dielectric constant*. Typical values for velocity and dielectric constant for pavement materials are shown in Table 1.1.

Table 1.1 - GPR Velocities and Dielectric Constants for Pavement Materials

velocity				
metric m/ns	metric cm/ns	English In/ns	Dielectric constant	Note
0.100	10.0	3.94	9.00	Typical for pcc
0.105	10.5	4.13	8.16	
0.110	11.0	4.33	7.44	Typical for pcc/ac
0.115	11.5	4.53	6.81	Typical for ac
0.120	12.0	4.72	6.25	
0.125	12.5	4.92	5.76	
0.130	13.0	5.12	5.33	
0.135	13.5	5.31	4.94	
0.140	14.0	5.51	4.59	
0.145	14.5	5.71	4.28	
0.150	15.0	5.90	4.00	
0.155	15.5	6.10	3.75	

Each GPR antenna operates at a range of frequencies and is characterized by its center frequency. The vertical resolution, or ability to resolve a feature such as a pavement layer, is mainly affected by the frequency, or wavelength, of the transmitted signal. The radar pulse has a finite width measured in nanoseconds and the pavement layers must be thick enough for reflections to appear without overlap. In general, higher operating frequencies are needed to resolve thinner layers and hence high frequency antennas with 1.0 GHz or 2.0 GHz center frequency are typically used for pavement thickness surveys.

The effective depth of penetration of the radar energy is primarily a function of the electrical properties of the material the signal is transmitted through, frequency of transmitted radar signal and overall system characteristics such as power output and receiver sensitivity. Lower frequencies achieve greater penetration depths but decrease vertical resolution.

Electromagnetic wave velocity and strength is determined primarily by a material's dielectric constant (ε), or its ability to store a charge from an electromagnetic field and then transmit that energy. In general, the greater the dielectric constant of a material, the slower the radar energy will travel through the material.

Attenuation is the measure of energy lost in travel related to the conductivity of the material. Attenuation of radar signals can be significant for conductive materials such as Portland cement concrete, clay and materials with a significant amount of moisture.

Sequential waveforms collected over a longitudinal profile can be stacked side by side to create a subsurface map of the pavement system as a function of radar signal travel time through the ground. Amplitudes and arrival times of the reflected signal can be used to estimate pavement thickness. Color coding waveforms to correspond to amplitude intensity is a common technique

to aid in visual interpretation of layer properties. Figure 1.1 shows GPR data collected on a typical flexible pavement. Sequential waveforms positioned vertically make up the first half of the profile while the second half utilizes color coded waveforms (13).
There are two basic types of GPR systems used for pavement evaluation: the non-contact horn antenna systems and the contact ground-coupled systems. The following paragraphs discuss methods for implementing these systems for pavement thickness quality assurance.

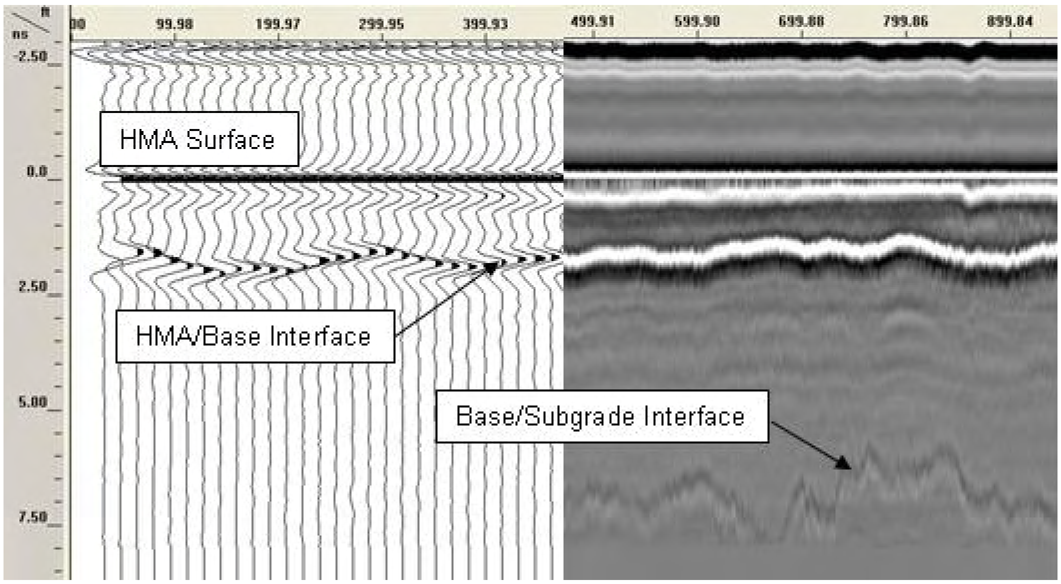


Figure 1.1- Stacked waveform and color coded GPR display

1.2.1.1. Horn Antenna GPR

The air-launched horn antenna is attached to the front or rear of a vehicle and suspended with the bottom of the antenna approximately 18 inches from the pavement surface. Consequently, the air-launched antenna is routinely used at highway speeds and is not physically affected by rough road conditions. But most importantly, it is not necessary to obtain cores to calibrate the air-launched horn antenna system. The system is calibrated by placing a metal plate under the antenna and collecting a GPR data file. The calibration file data collection process includes metal plate reflections recorded at the different heights that the antenna may experience during data collection over pavement. This metal plate file is later processed to produce a horn antenna calibration file that is used with subsequent data files to calculate the velocity of radar signal through the pavement. Thus, when using an air-launched horn antenna with the metal plate calibration technique the velocity through the pavement, and the corresponding thickness, is calculated for each individual GPR scan acquired. In addition, since the metal plate calibration file is applied to each scan, changes in the composition of the pavement are accommodated as they occur so that the accuracy of the system is not dependant on the last core location.
Another important advantage of the horn antenna is the ability to measure thin pavement layers. Since the antenna is suspended above the pavement surface, the direct-coupling (the portion of the transmitted energy radiated from the transmit antenna directly to the receive antenna) occurs at the antenna and not at the pavement surface where the ground-coupling occurs. With the

ground-coupled antenna the direct-coupling and ground-coupling occurs together, creating near-field interference that limits the minimum detectable pavement thickness. The 2 GHz air launched antenna can reliably resolve layer thicknesses of 1 inch while a 1.5 GHz ground -coupled antenna is normally able to reliable resolve first layer thicknesses greater than approximately 3 inches (14). Figure 1.2 shows an Air-launched Horn Antenna.

Figure 1.2 -Air-launched Horn Antenna

Implementation of the horn antenna method is shown in Figure 1.3. The figure shows the geometry of the antenna and the GPR ray paths. The reflected pulses are received by the antenna and recorded as a waveform as shown. As the equipment travels along the pavement, it generates a sequence of waveforms, also shown in the figure. The layer boundary between the asphalt and base is clearly visible in this sequence of waveforms. These waveforms are digitized and interpreted by computing the amplitude and arrival times from each main reflection. For the horn antenna method, the pavement thickness can be computed from these amplitudes and arrival times according to the following equations (2):

$$\text{Thickness (cm)} = \text{velocity} * \text{time}/2 \quad (1\text{-}1)$$

$$\text{Velocity (cm/ns)} = (150)/\sqrt{\varepsilon a} \quad (1\text{-}2)$$

$$\varepsilon a = [(A_{pl} + A)/ (A_{pl} - A)]^{2}$$

where velocity is calculated from εa, the dielectric constant of the asphalt; t is the time delay between the reflections from the top and bottom of the asphalt, computed automatically from each waveform; A is the amplitude of the reflection from the top of the asphalt, computed from each waveform; and Apl is the amplitude of the reflection from a metal plate, obtained during calibration. The constant, 150, is half the speed of light in air. The factor 2 converts the measured round-trip time to one-way time.

The above equations are based on the assumption that the transmitting and receiving antennas are in the same location, and that the GPR ray path is perpendicular to the pavement surface. These assumptions are not completely true, but the error introduced by this simplification has not had an adverse effect on accuracy for standard pavement thickness applications.

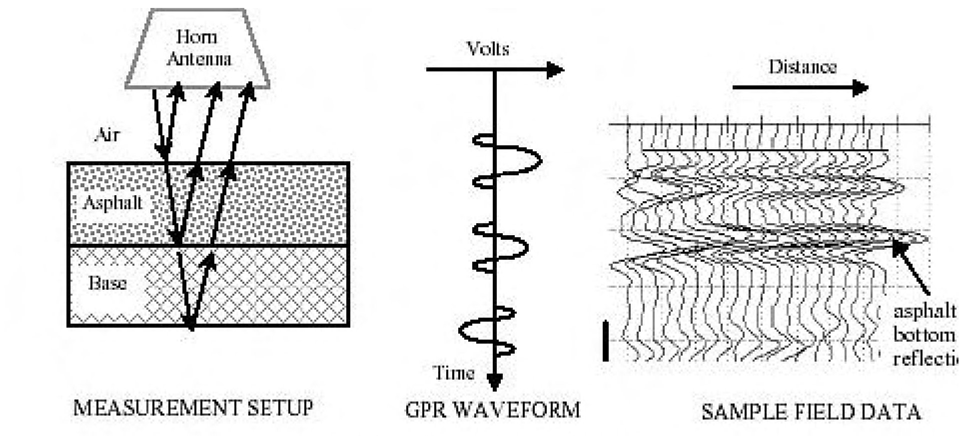


Figure 1.3 -Horn Antenna Method

1.2.1.1.1. Advantages and Disadvantages (15)

Advantages are:

- Only Highway speed subsurface pavement testing tool
- Excellent for Flexible pavement rehabilitation projects
- Can be merged with surface video and other NDT data

Limitations are (15):

- Depth limited to top 20 – 24 inches
- Attenuation problems with concrete layers
- Initially limited software available for processing data
- Pavements and materials can be complex – training and structured implementation approach required must have dielectric contrast between layers

Barriers to Implementation (15):

- In USA FCC restrictions on manufacturers
- Oversold - initial results disappointing
- No Certification of equipment and vendors

1.2.1.2. Ground-Coupled GPR

As the name suggests, a ground-coupled antenna needs to remain in contact with ground (or suspended very slightly above the ground) to properly couple the electromagnetic energy to and from the antenna. This presents some obvious limitations in using the ground-coupled antenna for high speed pavement surveys on roads in less than perfect condition. More importantly, to calibrate the ground-coupled system it is necessary to obtain cores from the pavement and physically measure the actual pavement thickness. The measured thickness value must be entered into the GPR analysis program so that the appropriate velocity of the radar signal through the pavement lavers may be derived to determine the pavement thickness. Since the composition of the pavement changes, it is necessary to obtain cores at regular intervals (1 core per km is one GPR manufacturers recommendation) to derive accurate pavement thicknesses. Even when the cores are acquired at regular intervals, the composition of the pavement is assumed to be constant between cores. Therefore, any change in pavement composition affecting radar signal velocity between cores is a source of error when using ground-coupled antennas. Ground-coupled systems operate with the antenna directly in contact with the pavement. Because of this configuration, equations (1) and (2) cannot be used, since the radar wave is launched directly into the pavement, and does not travel through air. Because of this configuration, the dielectric constant cannot be calculated directly from the data.

Figure 1.4 - Ground-Coupled Antenna

1.2.1.2.1. Advantages and Disadvantages (15)

Advantages are:

- Fairly inexpensive
- Robust Equipment – technology and software widely available
- Deep investigations possible with low frequency equipment

Limitations (15):

- Speed typically less than 10 mph
- Limited near surface information
- Penetration limited in clay material
- Qualitative info; usually need expert for interpretation

Barriers to Implementation (15)

• Technology not well understood by DOT's
• No significant barriers

1.2.1.2.2. Calibrated Single Antenna Method

One approach to using a ground-coupled antenna is to replace Equation (2) with a calibration curve. The calibration would relate the direct coupling of the antenna to the dielectric constant and velocity of the surface material. The direct coupling is the transmission which goes directly from the transmitter to the receiver. This direct coupling is observed on the data before the detection of the reflected arrivals. Since the direct coupling involves transmission through the pavement material, it is reasonable to assume that a correlation could be established between the direct coupling and the dielectric constant and velocity in the pavement material.

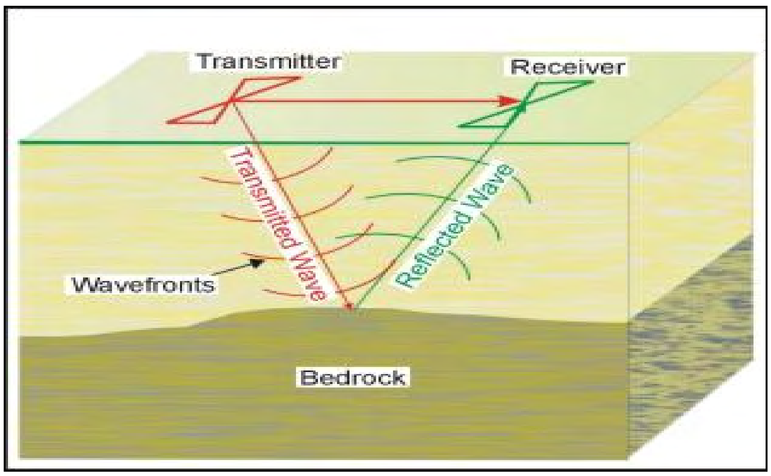


Figure 1.5 -Ground-coupled GPR Antenna Showing Direct Coupling

Given travel path equal to $V*t$, where V is the GPR velocity and t is the travel time, the thickness is calculated from the geometry as:

$$h = 0.5[(Vt)^2 . d^2]^{½} \qquad (1\text{-}3)$$

1.2.1.2.3. Dual Antenna Common Midpoint (CMP) Method

An alternative method involves using two ground-coupled antennas. This method, called the *common mid-point method* (CMP), is shown in Figure 1.6. The CMP method uses two ground-coupled antennas, one of which acts as a transmitter and the other as a receiver. The two antennas are initially adjacent to each other, and are then moved at equal distances from the initial midpoint. The implementation mechanism is such that a GPR scan is collected for each unit of movement (e.g., every 2 mm (0.08 inch)).
The reflected arrival from the bottom of the pavement takes on a hyperbolic pattern, whose

Equation is (16):

$$t_{tot}(i) = \frac{2}{V_2}\sqrt{x(i)^2 + d^2} \tag{1-4}$$

Where,
i = scan number
d = thickness of the pavement layer
V_2 = GPR velocity in pavement layer
$x(i)$ = antenna distance from common midpoint at scan i
$t_{tot}(i)$ = arrival time of GPR pulse for spacing $x(i)$

By fitting the observe data with this equation, both the pavement layer velocity and layer thickness can be determined.

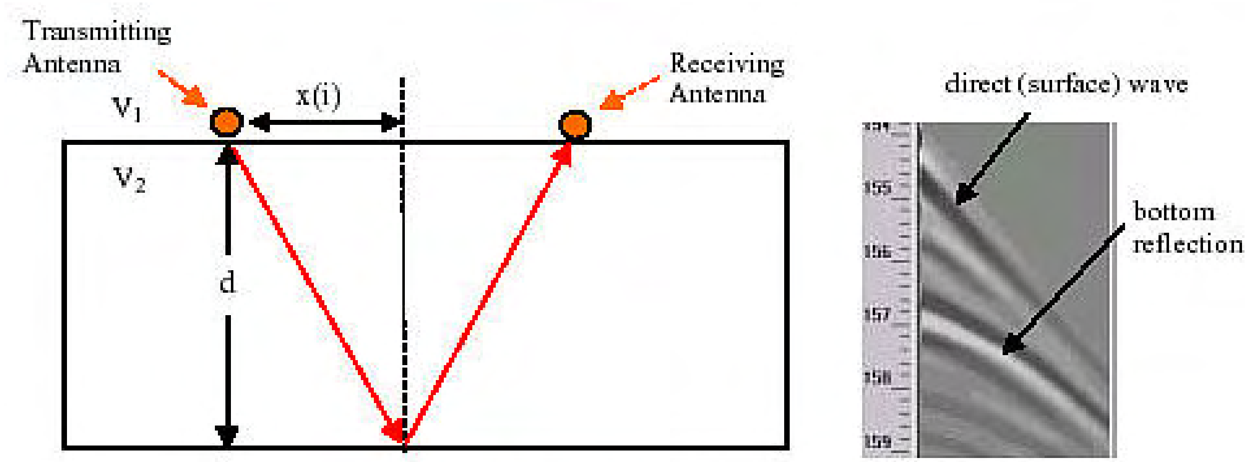


Figure 1.6- Ground-Coupled Common Midpoint (CMP) Method

Figure 1.7- CMP Measurement

1.2.2. GPR applications

Air coupled GPR:

- Thickness of pavement layers
- Moisture or density related defects in HMA and base layers
- Density of new Asphalt layers
- Delaminations in bridge decks (with HMA surfaces)
- Section uniformity (no surprises in construction)

Ground coupled GPR:

- Detecting buried objects
- Voids under thick concrete slabs
- Detecting steel presence and depth
- Locations where deep investigations are required

It is capable of detecting a number of parameters in reinforced concrete structures:

- the location of reinforcement
- the depth of cover
- the location of voids
- the location of cracks
- in situ density
- Moisture content variations

User expertise

User must have good knowledge of wave propagation behavior in materials in order to meaningfully collect and interpret results. Training and experience are required.

1.2.3. Advantages and Limitations

It can be used to survey large areas rapidly for locating reinforcement, voids and cracks. Results must be correlated to test results on samples obtained. Any features screened by steel reinforcement will not be recorded. With increasing depth, low level signals from small targets are harder to detect due to signal attenuation. It is expensive to use and uneconomical for surveying small areas. GPR technology lacks the ability to differentiate between layers of AC and layers of asphalt-treated materials in thickness estimation (19).

1.2.4. GPR Equipment and Software

Companies supplying GPR equipment and software are relatively few in number, and summaries of the key hardware and software providers relevant to this project are provided in Tables 1.2 and 1.3.

Table 1.2 – Summary of Commercial GPR Equipment

Manufacturer	Systems	System features	Antennas
GSSI	SIR-20	multiple antennas, laptop based, well suited for vehicle-based data collection	100, 200, 400, 900, 1500 MHz ground-coupled; 1.0, 2.0 GHz horn antennas
	SIR-3000	small, portable, single antenna	
Penetradar	IRIS	multiple antenna, vehicle based	500 MHz and 1 GHz air coupled; 400 & 500 MHz ground coupled 500 MHz, 1 GHz horn antennas
	IRIS-p	small, portable, single antenna	
Sensors and Softwares	Pulse Ekko 1000	multipurpose, single antenna	110, 225, 450, 900, and 1200 MHz ground coupled
	Noggin 1000	small, portable, single antenna	
Pulse Radar	Rodar	multi-antenna, vehicle based	500 MHz, 1 GHz horn antennas
Wave bounce	WB1	operates from laptop though USB	1 GHz horn antenna
Mala	RamacX3M	small, portable, single antenna	100, 250, 500, 800 1000 MHz ground coupled
	Ramac/GPR	modular, can have multiple antennas	

Table 1.3 – Summary of Commercial GPR Software

Supplier	Software Item	Capabilities
GSSI	Radan	general purpose GPR processing – can use data from other supplier's equipment
	Radan with Pavement Structure Module	adds picking and analysis of pavement layers to Radan
	Radan with BridgeScan	adds bridge deck condition analysis to Radan
Sensors and Software	Conquest 3D	3D imaging of concrete
	Ekko_View	General purpose display and analysis of GPR data
RoadScanners	Haescan	pavement layer thickness
	Road Doctor	adds videologging and georeferencing to above
Penetradar	PavePro	pavement layer analysis
	BridgePro	bridge deck condition analysis

Amongst the GPR systems, there are two types: small, portable, single antenna systems, and larger, vehicle mounted, multi-antenna systems. The smaller systems are useful for geotechnical applications where mobility and portability is important. The larger systems are useful for highway applications where speed of data collection and the possibility of multiple antennas are useful.

1.2.5. Accuracy and Interpretation of GPR

There are a number of factors to be taken into account when interpreting radar data and signals:

- hyperbolic shapes typically represent a point reflector
- the diameter of cylindrical objects ranging from rebars to metallic oil drums cannot be determined from radargrams
- radar wave velocity reduces when travelling through wet concrete
- radar waves are more rapidly attenuated when travelling through wet concrete
- radar waves cannot penetrate conductors such as: metals, clays, salt water, e.g. sea water
- radar antennas cannot identify objects in the near field which are closer to the surface than $\lambda/3$, where (17)

$$\text{Velocity } (v) = \text{frequency } (f) \times \text{wave length } (\lambda)$$

Therefore, $\lambda = v/f$.

Typical rear field resolutions for different antenna centre frequencies and dielectric constants are given in table 1.4.

Table 1.4-Typical rear field resolutions for different antenna centre frequencies at different dielectric constants

Antenna Center Frequency MHZ	Near Field Resolution ($\lambda/3$) cm		
	$\varepsilon=6$ Dry Concrete	$\varepsilon=9$	$\varepsilon=12$ Wet Concrete
500	25	20	18
900	13.5	11	10
1000	12	10	9
1500	8	7	6

1.2.6. Mechanical Wave Methods for Concrete Thickness Evaluation

Mechanical wave methods are very similar in concept to electromagnetic wave methods. With mechanical wave methods, a pulse of mechanical energy is transmitted into the pavement, and a transducer receives the reflected waves from the pavement layers. Analysis of these reflected return signals yields information on the pavement layer thickness and mechanical material properties.

1.2.6.1. Impact-Echo

Impact-echo (IE) is a technique developed for thickness measurement and delamination location in concrete. *S*everal different sources of commercial equipment are available. The system is based on a high resolution seismic reflection survey on concrete structures using an impact source, a broad band unidirectional receiver and a waveform analyzer.

The mechanical impact generates stress pulses in the structure (Fig. 1.8). The stress pulses undergo multiple reflections between the top and the bottom concrete layer. The surface displacements are recorded and the frequency of the successive arrivals of the reflected pulses is determined. Wave reflections are used for detection of discontinuities and voids in concrete structures. Discontinuities, defects and reinforcements could be identified in the resulting frequency spectra, as the wave reflects from their surfaces. Thus, knowing the thickness of a given layer, together with the derived frequencies, compression and shear wave velocities can be calculated. If, on the other hand, the thickness is unknown, the time-distance graph of the primary surface stress wave is used to calculate the thickness.

Recent studies show that impact-echo technique can be used for concrete early strength gain estimation and evaluation of micro-cracking and chemical attacks in concrete structures (18).

A source and receiver are co-located on the pavement surface. The arrangement is shown schematically in Figure 1.9. The impactor can be a hand-held hammer, a small steel bearing, or a mechanically actuated impact device. The impact generates a pressure wave (p-wave) which travels down through the pavement and is reflected back from the bottom of the pavement. The reflection occurs due to the difference in mechanical wave velocity and density between the pavement and the base. This difference does not always occur, such as when the concrete pavement is placed over a lean concrete base with very similar mechanical properties. With lean concrete base, however, there is often a lack of bonding between the concrete pavement and the base. The lack of bonding produces a mechanical discontinuity sufficient to provide the reflection from the bottom of the pavement.

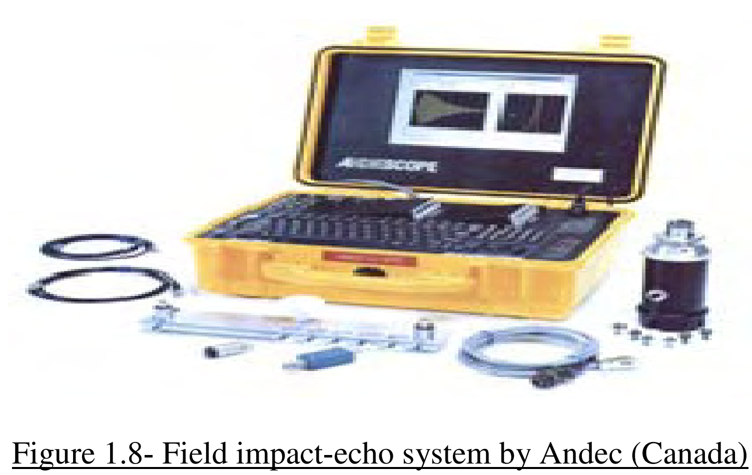


Figure 1.8- Field impact-echo system by Andec (Canada)

Much like in GPR, the wave travels twice the thickness of the pavement before returning to the surface, and the relationship between the thickness, the wave velocity, and the travel time is:

$$\text{Thickness (mm)} = V_p\,(t/2) \tag{1-5}$$

Where V_p is the p-wave velocity in the concrete and t is the round trip travel time. As shown in Figure 1.9, the wave reflects repeatedly back from the surface into the pavement and back from the pavement bottom, producing the repetitive reflection pattern shown. Rather than measure the travel time directly as in GPR, it has been shown that measurement of the frequency spectrum of the reflected signal is much more effective. The reflected signal frequency characteristics are shown in Figure 1.9. The frequency peak, f, or "thickness resonance" represents the repetition of reflected arrivals, or arrivals per second. The inverse of f is then the travel time. Therefore, Equation (1-5) becomes:

$$\text{Thickness (mm)} = V_p/2f$$

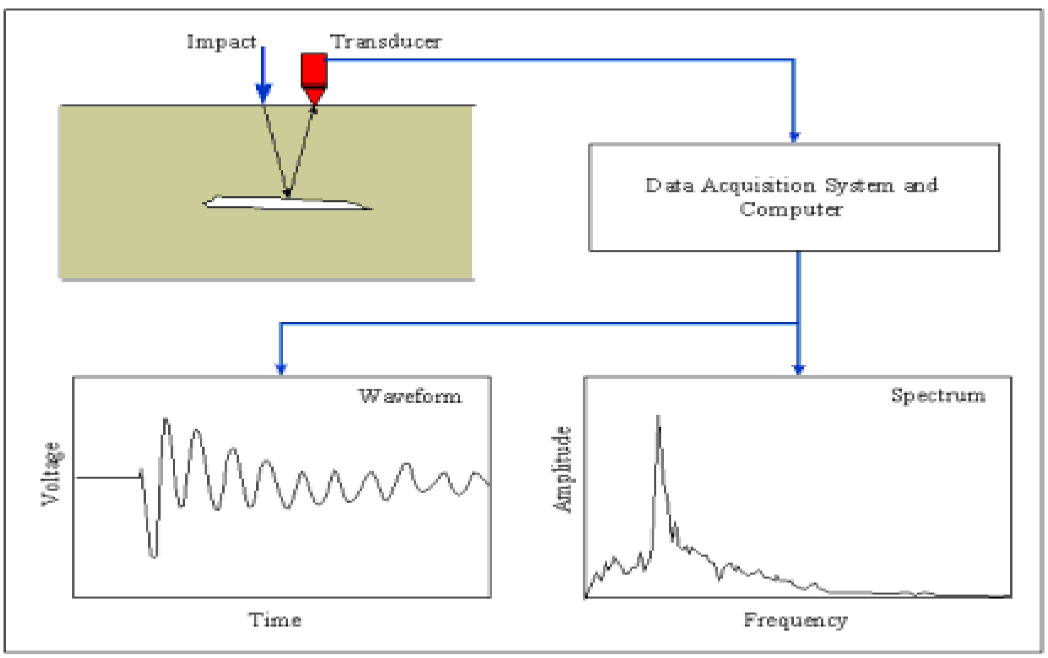


Figure 1.9-simplified diagram of the Impact-Echo method

The ASTM specification for this method (*12*) shows Equation (1-5) to be:

$$\text{Thickness (mm)} = 0.96\,V_p/2f \tag{1-6}$$

Where the 0.96 factor represents the "plate effect" on the p-wave propagation velocity.
The p-wave velocity, required for the above calculation, needs to be determined independently. The ASTM specification offers a method by which the p-wave velocity is measured along the exposed surface of the material. This method uses two transducers placed on the surface of the

material. An impactor strikes the concrete near the first transducer, and the p-wave arrives at the first and then at the second transducer. The time difference between p-wave arrivals is measured, and the time difference and transducer distance yields the velocity V_p. In practice, the velocity measurement is more difficult to make and to interpret than the impact-echo method.
The ASTM specification indicates that there is a 1%-2% error in thickness calculation introduced by the resolution limitations in measuring the thickness resonance. A second accuracy issue related to the impact-echo method is that the p-wave velocity measured at the surface does not necessarily represent the velocity through the depth. In fact up to 6% difference in V can be expected between surface and interior concrete.
An alternative method for calculating the p-wave velocity is to use calibration cores. Using a core with known thickness, Equation (1-4) can be used to calculate V. However, since V_p may change from location to location, it is not clear how effective a single calibration core may be, nor is it clear how many calibration cores will be needed.
A number of concrete pavement thickness accuracy studies have been carried out over the past several years. A summary of the results of these studies is shown in Table 1.5.

Table 1.5 -Summary of Previous Impact Echo Concrete Pavement Thickness Studies

Location/reference	subsite	Core(mm)		Impact Eco(mm)		Difference of Mean
		mean	ST Dev	mean	ST Dev	
Indiana	n.a.	361	9	364	15	-4
Nebraska	n.a.	256	4	253	4	3
Virginia	Route 460	242	9	242	9	0
	Route 64	208	6	209	8	-1
Arizona	200-LCB	205		203		2
	200-ASPB	209		212		-3
	200-DGAB-1	197		195		2
	200-DGAB-2	212		209		3
	300-LCB	294		291		3
	300-ASPB	294		300		-6
	300-DGAB-1	288		279		9
	300-DGAB-2	287		279		8

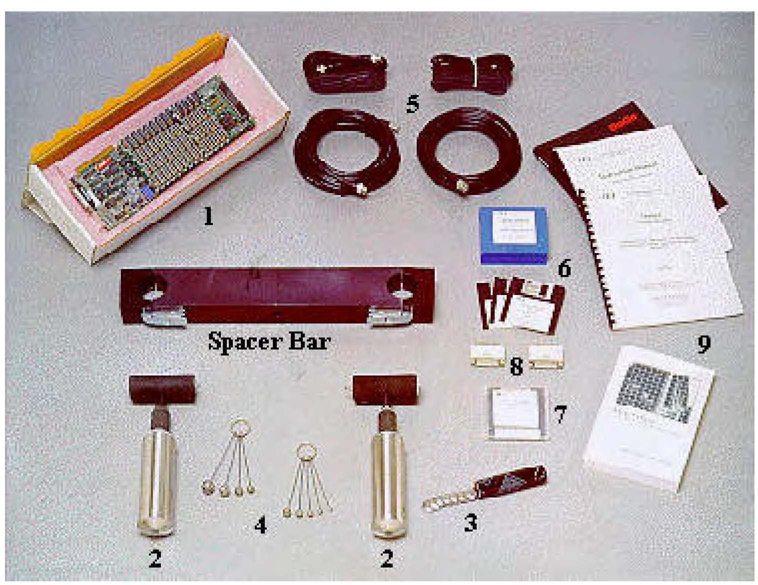
The differences shown between the impact-echo and core data in Table 1.5 are generally small. However, discussions with experienced practitioners have indicated that the small differences shown in Table 1.5 are not typical of field practice.
As indicated earlier, the accuracy of impact-echo depends on the base material type, the contact conditions, and the concrete surface conditions. Consequently, it was felt that an independent assessment of impact-echo was necessary to evaluate its application for concrete pavement quality assurance.

1.2.6.1.2. Equipment for impact-echo testing

Examples of the equipment used for impact-echo testing are the systems developed by Impact Echo Instruments as illustrated in Fig. 1.10. There are two systems. The Type A Test System is comprised of a Data Acquisition System, one cylindrical hand held transducer unit, 200 replacement lead disks for the transducer, Ten spherical impactors 3 mm to 19 mm in diameter (used to vary the contact time), one 3.7 m cable and one 7.6 m cable.

The Type B Test System is comprised of a Data Acquisition System, two cylindrical hand-held transducer units, 200 replacement lead disks for the transducer, ten spherical impactors 3 mm to 19 mm in diameter, one 3.7 m cable, one 7.6 m cable and a spacer bar to use with the two transducers.

Impact-Echo test system, Type A



Impact-Echo test system, Type B

Figure 1.10-Impact-Echo test systems

1.2.6.2. General procedure for Impact-Echo testing

Using the Impact–Echo Instruments System A, the technique used is to vary the diameter of the impactor until a clear dominant frequency is obtained. Typically the diameter of the impactor has to increase as the thickness of the material being tested increases to obtain reflections from the rear surface of the material being tested.

1.2.6.3. Applications of and examples of the use of the impact-echo testing method

One use has been in measuring the thickness of concrete pavements. The accuracy of the thickness measurement was found to vary depending on the sub base on which concrete is laid. For example the uncertainty of the thickness measurement was within 1% for a concrete pavement on lean concrete sub-base, 2% for pavement on an asphalt sub-base and 3% for pavement on an aggregate sub-base (17). Also Impact-Echo has been used in locating a variety of defects within concrete elements such as delaminations, voids, or honeycombing.

1.2.6.4. Range and limitations of Impact-Echo testing method

In generic terms the Impact-Echo method is a commercial development of the well-known frequency response function method (Frf) and the theory of vibration testing of piles.
The user should beware of the claimed accuracy of detecting defects or thickness in terms of an absolute measurement. It is better to think in terms of a multiple of the wavelength:

Velocity = frequency × wavelength
$V = f\lambda$
Where,
λ is wavelength

For impact test work, recent research has shown that the "near field" detection capability of impact-echo (Martin, Hardy, Usmani and Forde, 1998) is:

$$\text{Minimum depth of detectable target} = \lambda/2$$

Many test houses will deliberately or otherwise use the null hypothesis (17):

"If a defect is not identified – then none exists."

In order to determineλ, one could assume the velocity through the good concrete to be:

$$\text{Velocity} = 4{,}000 \text{ m/s}$$

(Poorer or younger (<28 days) concrete might have a velocity equal to 3,500 m/s), thus:

$$\lambda = 4000/\text{frequency meters}$$

When using impact-echo equipment, one would select the excitation frequency by turning a dial in order that the appropriate size of spherical hammer is chosen. For example, if a 10 KHz excitation frequency hammer is chosen, the near field minimum depth resolution would be

$$\lambda/2 = 4000 / 10 \text{ KHz} \times 2 = 4/20 = 0.2 \text{ meters}$$

It is argued by Sansalone, et al. that when one cannot detect the shallow “target”, the “anomaly” can be detected by observing the apparent depth to the base of a slab or depth to a back wall. This depth will appear to increase when a defect occurs. This method of interpretation must be used with some caution.
A check needs to be undertaken on actual impact frequency achieved as the surface of the concrete may crumble. If the surface crumbles, even a little, on impact:

- contact time increases
- lower frequency of excitation is achieved
- longer wavelength signal is generated
- lower “near field” resolution is achieved.

Good practice would be to take multiple impact-echo readings and discard the first two readings. This assumes that the third and subsequent readings are good.
The size of the test object plays an important role in the results obtained. Geometrical effects due to limited size are the cause of signals, which can be misleading. It is therefore necessary to perform the impact-echo test at several points on the surface to identify possible geometrical effects.

1.2.6.5. Advantages and Disadvantages

Advantages are:

- Equipment is commercially available,
- Capable of locating of variety of defects,
- Does not require coupling material,
- Access to only one face is required
- Light weight, portable
- Locate flaws as well as accurately determine at what depth the flaws are occurring
- Results are achieved very correctly (<10s) through the use of a portable computer

Limitations:

- Experienced operator is required,
- Current instrumentation limited to testing members less than 2 meters thick

1.3. Summary

The methods described in this section are summarized below.

Method	Technology	Application	Measurement Type	Measurement Rate	Prior Experience
Horn antenna	Non-Contact GPR (electromagnetic)	asphalt	continuous	up to 9 m/sec (30 feet/sec)	extensive
Calibrated Single Antenna	Ground-Coupled GPR (electromagnetic)	asphalt or concrete	continuous	up to 1.5 m/sec (5 feet/sec)	none documented
Dual Antenna CMP	Ground-Coupled GPR (electromagnetic)	asphalt or concrete	point	estimated 2 min./point	limited for pavement
Impact-Echo	Mechanical Wave	concrete	point	estimated 30 sec./point	extensive

The summary table distinguishes the methods which are continuous vs. those which are "point". The continuous methods can collect data while the equipment is moved continuously along the pavement. The "point" methods must be set up to make a measurement at a particular point. An estimated rate of data collection has been indicated. Note that some of the methods are well established, while others are relatively new for this application.

1.4. Conclusion

The results of the accuracy study showed that the GPR system is capable of estimating the layer thicknesses accurately, especially for HMA layers.

It should be emphasized that the accuracy of the GPR system may be significantly affected when noise is present in the data due to external interferences.

The repeatability of the GPR system was studied using the data collected at variable speeds. The system showed excellent repeatability for speeds ranging from less than 15 mph up to 70 mph (13).The thickness predictions from the data collected at highway speeds were very reliable. However, it is strongly recommended that when the data is collected at highway speeds, more markers be inserted in the GPR data in order to minimize the offset errors. These markers should be linked to physical objects with known mileposts.

Also the GPR system is reliable for surveying pavement thicknesses. It is strongly recommended that the GPR system be used as a tool for assisting in pavement thickness determination. More accurate thickness information can be obtained when the core thicknesses are used as feedback into the GPR analysis for calibration of radar velocities.

References

(1) Deacon, J. A., C. L. Monismith, and J. T. Harvey .Pay Factors for Asphalt-Concrete Construction: Effect of Construction Quality on Agency Costs. Technical Memorandum, TM-UCB-CAL/APT-97-1, April 1997.

(2) J. J. Daniels, “Fundamentals of Ground Penetrating Radar”, Soft Earth Associates Inc., Westerville, Ohio, 1990.

(3) J. B. Hasted, “Aqueous Dielectrics”, London, Chapman and Hall, 1973

(4) Ulriksen, C., F. Peter “Application of Impulse Radar in Civil Engineering”, Doctoral Thesis, Department of Engineering Geology, Lund University of Technology, Lund Sweden, Reprinted and distributed by Geophysical Survey Systems Inc., North Salem, New Hampshire, 1982.

(5) D. A. Harris, “A GPR Based Underground Workings Monitoring Program for Surface Mines”, M.S. Thesis, Department of Mining Engineering, Montana Tech of the University of Montana, Butte, MT, 1998.

(6) Maser, K. R., "Pavement Characterization Using Ground Penetrating Radar: State of the Art and Current Practice," *Nondestructive Testing of Pavements and Backcalculation of Moduli: Third Volume, ASTM STP 1375,* American Society for Testing and Materials, West Conshohocken, PA, 1999.

(7) Wenzlick, J. T. Scullion, and K. R. Maser .High Accuracy Pavement Thickness Measurement using Ground Penetrating Radar. Report No. RDT 99-003, Missouri Dept. of Transportation, February, 1999.

*(8)*Infrasense, Inc.California Department of Transportation*.”Non-Destructive Measurement of Pavement Layer Thickness”,* final repor, April 25, 2003.

(9) INFRASENSE, Inc. 14 Kensington Road ,Arlington, MA 02476,” *Feasibility of Using Ground Penetrating Radar (GPR) for Pavements, Utilities, and Bridges,* August 2006

(10) Maser, K. R., and T. Scullion .Influence of Asphalt Layering and Surface Treatments on Asphalt and Base Layer Thickness Computations using GPR. TTI Report TX-92-1923-1, September 1992.

(11) Saarenketo, T. and P. Roimela, .Ground Penetrating Radar Technique in Asphalt Pavement Density Quality Control". *Proceedings of the 7th International Conference on Ground Penetrating Radar*, Lawrence, KS, pp. 461-466, May 27-30, 1998.

(12) Ulriksen, C. P. F., .Applications of Impulse Radar to Civil Engineering., published by GSSI, N. Salem, NH, 1993.

(13) Charles Holzschuher, Hyung Suk Lee ,Jamie Greene ,April 2007 , STATE MATERIALS OFFICE, STATE OF FLORIDA ,“*ACCURACY AND REPEATABILITY OF GROUND PENETRATING RADAR FOR SURFACE LAYER THICKNESS ESTIMATION OF FLORIDA ROADWAYS”*

*(14)*Robert Parrillo and Roger Roberts Ph D,Geophysical Survey Systems Inc., Salem, NHIntegration of FWD and GPR, Gary Sanati ,Foundation Mechanics, Inc., El Segundo, CA

(15) A. Wimsatt, S. Hurlebaus, T. Scullion &E. Fernando Texas Transportation Institute College Station, TX, 77843 ,International Symposium on Nondestructive Testing for Design Evaluation and Construction Inspection, ”*Promising Existing and Emerging Technologies & Techniques”,*

(16) Halabe, U., A. Sotoodehnia, K. R. Maser, E. A. Kausel, .Modeling the Electromagnetic Properties of Concrete. *ACI Materials Journal*, Volume 90, No. 6, pp. 552-563,

November-December 1993.
(17) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2002 *"Guidebook on non-destructive testing of concrete structures"* TRAINING COURSE SERIES No. 17
*(18)*IAEA, International Atomic Agency, Vienna 2005 *"non-destructive testing for plant life assessment* "Training Course Series No.26

(19) National Cooperative Highway Research Program, RESEARCH RESULTS DIGEST , December 2002—Number 271, "*Determination of Insitu Material Properties of Asphalt Concrete Pavement Layers from Nondestructive Tests"*

2. Density

2.1. Thickness and Density (Radioisotope Gauges)

2.1.1. Fundamental principles

The use of radioisotopes for the non-destructive testing of concrete is based on directing the gamma radiation from a radioisotope against or through the fresh or hardened concrete. When a radiation source and a detector are placed on the same or opposite sides of a concrete sample, a portion of radiation from the source passes through the concrete and reaches the detector where it produces a series of electrical pulses. When these pulses are counted the resulting count or count rate is a measure of the dimensions or physical characteristics, e.g. density of the concrete. Although this radiometry method has not been commonly used on concrete, the increasing use of radioisotopes to measure the compaction of asphalt or bituminous concrete and the soil-aggregate mixtures used in road construction means that the method may be more commonly used in the future. The method has been used, for instance, for density determinations on roller compacted and bridge deck concrete (1).
The interaction of gamma rays with concrete can be characterized as penetration with attenuation that is, if a beam of gamma rays strikes a sample of concrete, (a) some of the radiation will pass through the sample, (b) a portion will be removed from the beam by absorption, and (c) another portion will be removed by being scattered out of the beam (when gamma rays scatter, they lose energy and change direction). If the rays are traveling in a narrow beam, the intensity I of the beam decreases exponentially according to the relationship:

$$I = I_0 e^{-\mu x} \qquad (2\text{-}1)$$

Where,

I_o is the intensity of the incident beam,
x is the distance from the surface where the beam strikes,
μ is the linear absorption coefficient.

For the gamma ray energies common in nuclear instruments used to test concrete, the absorption coefficient includes contributions from a scattering reaction called Compton scattering, and an absorption reaction called photoelectric absorption. In Compton scattering, a gamma ray loses energy and is deflected into a new direction by collision with a free electron. In photoelectric absorption, a gamma ray is completely absorbed by an atom, which then emits a previously bound electron. The relative contributions of Compton scattering and photoelectric absorption are a function of the energy of the incident gamma rays. In concrete, Compton scattering is the dominant process for gamma ray energies in the range from 60 keV to 15 MeV, while photoelectric absorption dominates below 60 keV.
The amount of Compton scattering, which occurs at a given gamma ray energy, is a function of the density of the sample being irradiated. The amount of photoelectric absorption that occurs is chiefly a function of the chemical composition of the sample; it increases as the fourth power of the atomic number of elements present.

The detectors for the radiometry techniques absorb a portion of the radiation and turn it into electrical pulses or currents, which can be counted or analyzed.

2.1.2. General procedure for thickness and density gauges

All gamma radiometry systems are composed of (a) a radioisotope source of gamma rays, (b) the object (concrete) being examined, and (c) a radiation detector and counter.
Measurements are made in either of two modes, direct transmission (Fig2.1) or backscatter (Fig.2.2).

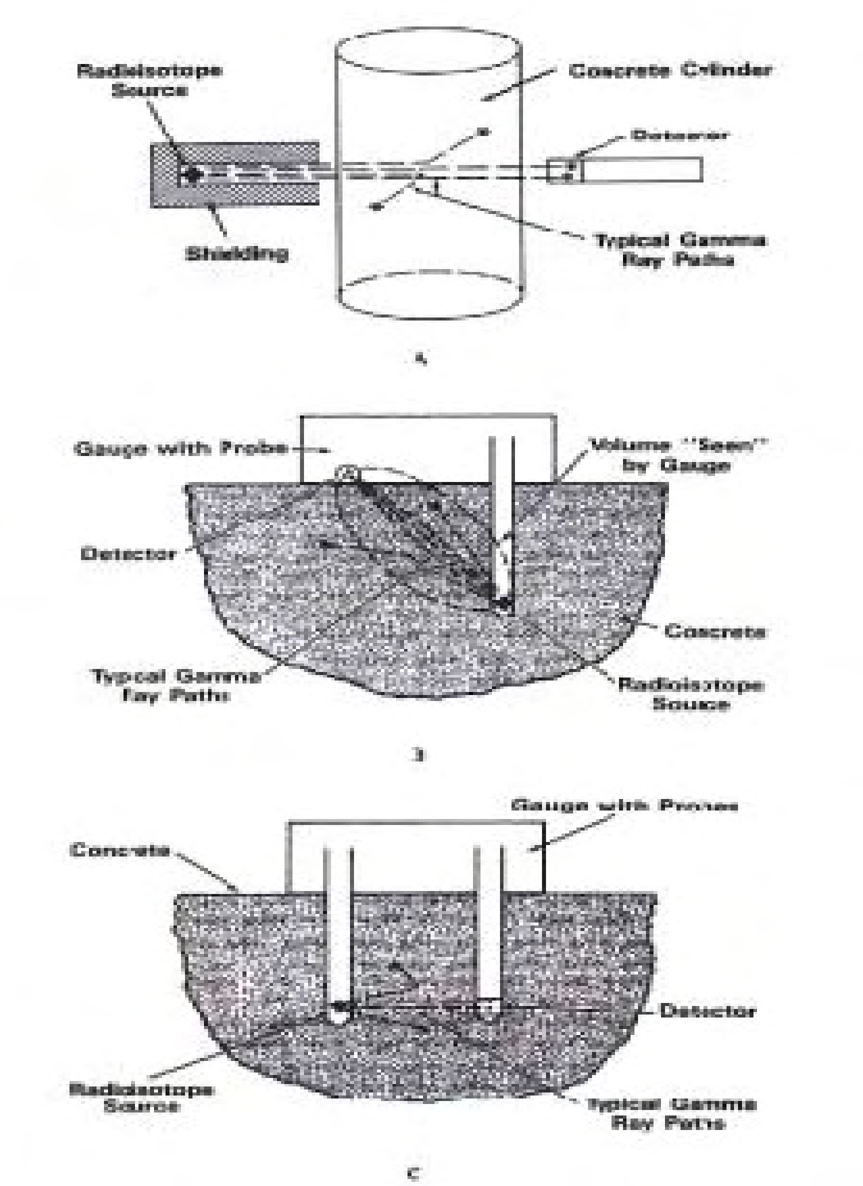


Figure2.1- Direct transmission (A) source and detector external to concrete, (B) source internal, detector external, and (C) source and detector both internal

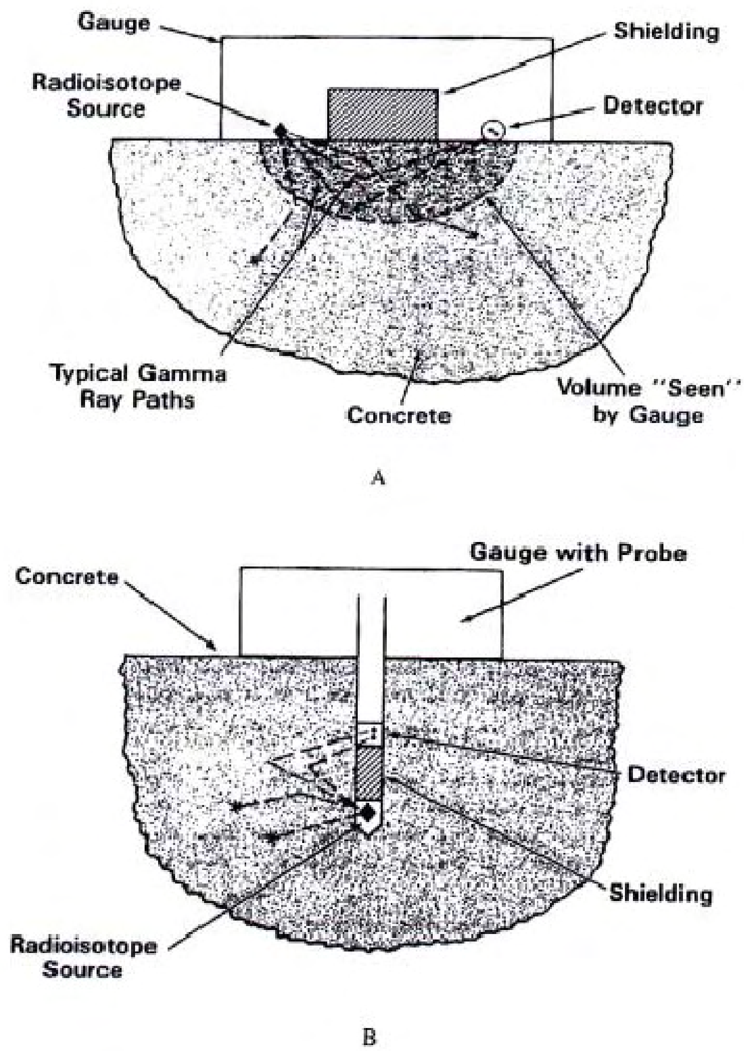


Figure 2.2- Backscatter (A) source and detector both external to concrete, (B) both in probe internal to concrete

In direct transmission, the specimen, or at least a portion of it, is positioned between the source and the detector. The source and detector may be both external to the concrete sample (Fig2. 1A); e.g. in making density scans on cores or thickness determinations on pavements. The source may be inside the concrete and the detector outside (Fig2.1B), e.g. in determining the density of a newly placed pavement. Or the source and detector may both be inside the concrete (Fig2.1C), e.g. in determining the density of a particular stratum in a newly placed pavement.
In direct transmission, the gamma rays of interest are those that travel in a straight (or nearly straight) line from the source to the detector. Gamma rays that are scattered through sharp angles, or are scattered more than once, generally do not reach the detector. The fraction of the originally emitted radiation that reaches the detector is primarily a function of the density of the concrete, and of the shortest distance between the source and the detector through the concrete, as shown in Equation 2-1. Typical gamma ray paths are shown in Fig2.1. The actual volume of

the concrete through which gamma rays reach the detector, i.e. the volume which contributes to the measurement being made, is usually ellipsoidal in shape (Fig2.1B), with one end of the volume at the source and the other at the detector. Sources typically used in direct transmission devices allow measurements to be made through 50 to 300 mm of concrete.
In backscatter measurement, the source and the detector are next to each other, although separated by radiation shielding. No portion of the concrete sample lies on a direct path between the source and detector. The source and detector may both be external to the concrete (Fig2.2A), e. g. in determining the density of a newly placed pavement or bridge deck from the top surface of the concrete.
In backscatter, only gamma rays that have been scattered one or more times within the concrete can reach the detector. Shielding prevents radiation from traveling directly from the source to the detector. Examples of gamma ray paths are shown in Fig2.2A. Each time a gamma ray is scattered it changes direction and loses some of its energy. As its energy decreases, the gamma ray becomes increasingly susceptible to photoelectric absorption.
Consequently, backscatter measurements are more sensitive to the chemical composition of the concrete sample than are direct transmission measurements in which unscattered gamma rays form the bulk of the detected radiation.
Backscatter measurements made from the surface are usually easier to perform than direct transmission measurements, which require access to the interior or opposite side of the concrete. However, backscatter has another shortcoming besides sensitivity to chemical composition: the concrete closest to the source and detector contributes more to radiation count than does the material farther away.

2.1.3. Equipment for thickness and density gauges

For typical, commercially available backscatter density gauges, the top 25 mm of concrete sample yields 50 to 70% of the density reading, the top 50 mm yield 80 to 95%, and there is almost no contribution from below 75 mm. The source, detector, and shielding arrangement can be modified to somewhat increase the depth to which a backscatter gauge will be sensitive. A gauge has been developed for mounting on the back of a slip form paver for continuous density monitoring. Slightly over 70% of the device's reading comes from the top 50 mm of concrete and about 5% comes from below 90 mm. Gauges with minimal depth sensitivity may be desirable for applications such as measuring density of a thin [25 to 50 mm] overlay on a bridge deck.
Backscatter measurement has another disadvantage: its sensitivity to surface roughness; however, this is rarely a concern for measurements on concrete. Gamma radiometry systems for monitoring density generally use ^{137}Cs (662 keV) sources, but ^{226}Ra (a wide range of gamma ray energies, which can be treated as equivalent, on the average, to a 750 keV emission) and ^{60}Co (1.173 and 1.332 MeV) are employed in some. These sources are among the few that have the right combination of long half-life and sufficiently high initial gamma ray energy for density measurements. The half-life of ^{137}Cs, for example, is 30 years.
Most commercially available density gauges employ gas filled Geiger-Muller (G-M) tubes as gamma ray detectors because of their ruggedness and reliability. Some prototype devices have employed sodium iodide scintillation crystals as detectors. The crystals are more efficient capturers of gamma rays than G-M tubes. They also can energy discriminate among the gamma rays they capture, a feature which can be used to minimize chemical composition effects in

backscatter mode operation. However, the crystals are temperature and shock sensitive and, unless carefully packaged, they are less suitable for field applications than the G-M detectors.
Portable gauges for gamma radiometry density determinations are widely available. A typical gauge is able to make both direct transmission and backscatter measurements, as shown in Figs.2.1B and 2.2A, respectively. The gamma ray source, usually 8 to 10 mCi of ^{137}Cs, is located at the tip of a retractable (into the gauge case) stainless steel rod. The movable source rod allows direct transmission measurements to be made at depths up to 200 or 300 mm, or backscatter measurements when the rod is retracted into the gauge case. The typical gauge would have one or two G-M tubes inside the gauge case about 250 mm from the source rod. With the source rod inserted 150 mm deep into the concrete, the direct transmission source-to-detector distance would be about 280 mm.
Detailed procedures for both direct transmission and backscatter measurements are given in ASTM Standard Test Method C 1040. Density measurements require establishment of calibration curves (count rate vs. sample density) prior to conducting a test on a concrete sample. Calibration curves are created using fixed density blocks, typically of granite, limestone, aluminum, and/or magnesium. Method C 1040 encourages users to adjust the calibration curves for local materials by preparing fresh concrete samples in fixed volume containers (the containers must be at least 450 mm × 450 mm × 150 mm for backscatter measurements). The nuclear gauge readings on the concrete in such a container are compared with the density established gravimetrically, i.e. from the weight and volume of the sample, and the calibration curve is shifted accordingly.
In-place tests on concrete are straightforward. For direct transmission measurement, the most common configuration is that shown in Fig.2.1B; the gauge is seated with the source rod inserted into a hole that has been formed by a steel auger or pin. For a backscatter measurement, the most common configuration is shown in Fig.2.2A, with the gauge seated on the fresh or hardened concrete at the test location. Care must be taken to ensure reinforcing steel is not present in the volume "seen" by the gauge. Reinforcing steel can produce a misleadingly high reading on the gauge display. Counts are accumulated, typically over a 1 or 4 minute period, and the density is determined from the calibration curve or read directly off a gauge in which the calibration curve has been internally programmed.
Tests with other gamma radiometry configurations (Figs2.1A, 2.1C, and 2.2B) employ the same types of sources and detectors. Various shielding designs are used around both sources and detectors in order to collimate the gamma rays into a beam and focus it into a specific area of a sample. The two-probe direct transmission technique (Fig2.2C) needs additional development but has considerable potential for monitoring consolidation at particular depths, e.g. below the reinforcing steel in reinforced concrete pavements.

2.1.4. Applications of thickness and density gauges

Currently no procedures are in standard use to measure the in-place quality of concrete immediately after placement; that quality is not assessed until measurements such as strength, penetration resistance, and/or smoothness can be made after the concrete has hardened.
Gamma radiometry is also being used extensively for monitoring the density of roller compacted concrete. Densification is critical to strength development in these mixtures of cement (and pozzolans), aggregates and a minimal amount of water. After placement the concrete is compacted by rollers, much the same as asphalt concrete pavements.

Commerica1ly available nuclear gauges have become standard tools for insuring the concrete is adequately compacted.

Gamma radiometry has found limited application in composition determinations on PCC. When radioisotope sources emit low energy (below 60 keV) gamma rays, photoelectric absorption is the predominant attenuation mechanism, rather than Compton scattering. Since the absorption per atom increases as the fourth power of the atomic number Z, it is most sensitive to the highest Z element present in a sample. Noting that calcium in portland cement is the highest Z element present in significant quantities in PCC (in mixtures containing noncalcareous aggregates). Because of the sensitivity of photoelectric absorption to Z, the cement content procedure required calibration on a series of mixtures of different cement contents for a given aggregate source. This sensitivity to aggregate composition remains a barrier to further application of the technique.

A short lived but interesting application of gamma radiometry is in pavement thickness determinations. As Equation 2.1 shows gamma ray absorption is a function of the thickness of a specimen. Therefore, a source could be placed beneath a PCC pavement, and, if a detector is positioned directly over the source, the count recorded by the detector would be a function of the pavement thickness. Researchers placed thumbtack-shaped 46Sc sources on a pavement sub-base before a PCC pavement was placed. The sources were difficult to locate after the concrete was placed, however, and the technique was abandoned albeit with a recommendation that it deserved further research.

2.1.5. Advantages and limitations of thickness and density gauges

Gamma radiometry offers engineers a means for rapidly assessing the density and, therefore, the potential quality of concrete immediately after placement. Direct transmission gamma radiometry has been used for density measurements on hardened concrete, but its speed, accuracy, and need for internal access make it most suitable for quality control measurements before newly placed concrete undergoes setting.

Backscatter gamma radiometry is limited by its inability to respond to portions of the concrete much below the surface, but it can be used over both fresh and hardened concrete and can be used, in non-contact devices, to continuously monitor density over large areas. Gamma radiometry techniques have gained some acceptance in density monitoring of bridge deck concrete and fairly widespread acceptance for density monitoring of roller-compacted concrete pavement and structures.

Summary of the advantages and limitations of backscatter and direct transmission gamma radiometry techniques is given in Table 2.1.

Table 2.1- Advantages and Limitations Various Gamma Radiometry Techniques

Technique	Advantages	Limitations
Gamma radiometry for density	Technology well developed; rapid, simple, rugged and portable equipment; moderate initial cost; minimal operator skill	Requires license to operate; requires radiation safety program
Backscatter mode	Suitable for fresh or hardened concrete; can scan large volumes of concrete continuously	Limited depth sensitivity; sensitive to concrete's chemical composition and surface roughness
Direct transmission mode	Very accurate; suitable primarily for fresh concrete; low chemical sensitivity	Requires access to inside or opposite side of concrete

2.2. MOISTURE GAUGES

2.2.1. Fundamental principles

Moisture gauges consist of a source of neutron radiation, which irradiates the material under test. As a result of radiation, gamma rays are created and detected. The result is a series of counts, which are a measure of the composition of the concrete. The sources used to generate the neutrons produce fast neutrons, which are scattered by the various elements in the material under test losing energy and changing direction after every collision. Neutron radiometric procedures usually employ a source/detector configuration similar to that used in gamma backscatter probes, as in Fig2.2B. The probe might contain a 100 mCi fast neutron source ($^{241}Am/Be$) and a gas filled BF_3 or ^3He detector. Because the detector is almost totally insensitive to fast neutrons, no shielding is employed between it and the source. The response of a neutron radiometry gauge arises from a much larger volume of concrete than does that of a gamma backscatter gauge. For example, a neutron radiometry probe completely surrounded by concrete with a water content of 250lb/yd^3(150 kg/m^3) will effectively be seeing the concrete up to 14 in. (350 ml) away; a gamma backscatter probe in the same concrete will be seeing the concrete no more than 4 in. (100 mm) away. Hydrogen atoms are the most effective scatterers of the neutrons and collisions with hydrogen atoms rapidly change neutrons from fast to slow. Neutrons with energies greater than 10 keV are described as "fast", between 0.5 eV and 10 keV, as "epithermal", and less than 0.5 eV as "slow". A measurement of the number of slow neutrons present, therefore, serves as an indicator of how much hydrogen is present in a sample. Since the only hydrogen present in concrete typically is in water molecules, slow neutron detection can be used as a measure of water content in concrete.

Neutrons do not ionize the gas in a gas filled tube directly, but are absorbed by boron trifluoride or ^3He in a tube. The latter gases emit secondary radiation that ionizes the gas in the tube and produces electrical pulses. Gas filled neutron detectors are widely used in moisture gauges in agriculture and civil engineering applications.

2.2.2. Applications of moisture gauges

It can be used to measure moisture content of concrete, soil and bituminous materials and to map moisture migration patterns in masonry walls. Their application to concrete testing is very recent and still in the exploratory stage.

2.2.3. Advantages and Disadvantages

Advantages are:

- Instrument is portable
- Moisture measurements can be made rapidly

Limitations are:

- A minimum thickness of surface layer is required for backscatter to be measured,
- It measures only the moisture content of surface layer (50 mm),
- It emits radiation,
- Results are inaccurate because hydrogen atoms of building materials are measured in addition to those of water,
- Its use in concrete is limited and requires calibration in order to calculate density or moisture content

References

(1) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2002,"*Guidebook on non-destructive testing of concrete structures*", TRAINING COURSE SERIES No. 17

3. Uniformity test

3.1. PULSE VELOCITY TEST

3.1.1. Fundamental principle

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete from the transducer using a liquid coupling material such as grease or cellulose paste, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. A complex system of stress waves develops, which include both longitudinal and shear waves, and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by a second transducer. Electronic timing circuits enable the transit time T of the pulse to be measured.

Longitudinal pulse velocity (in km/s or m/s) is given by:

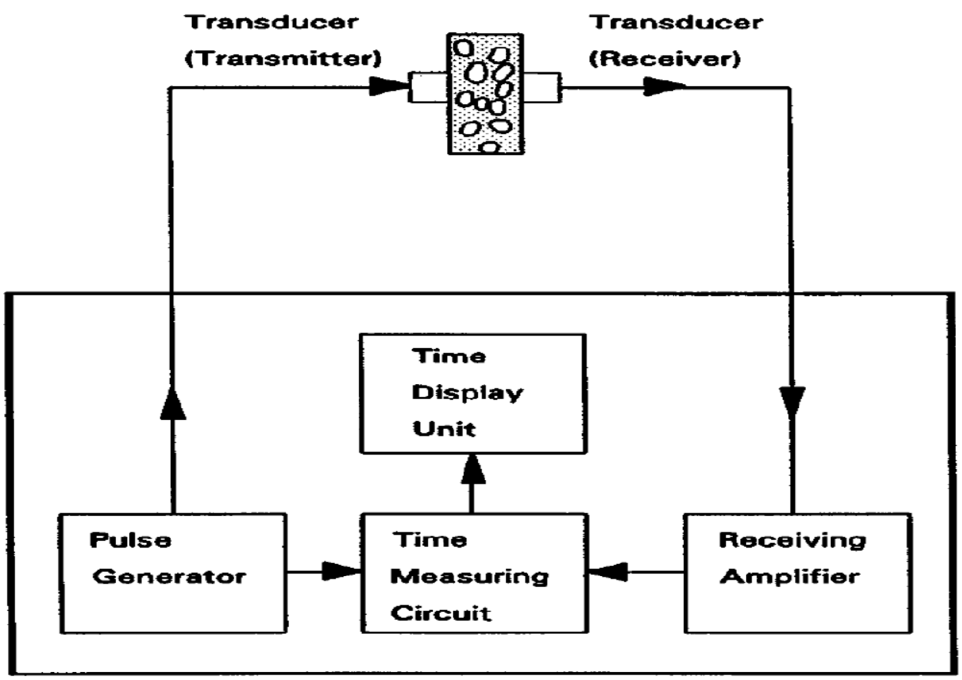
$$v = \frac{L}{T} \tag{3-1}$$

Where,

v is the longitudinal pulse velocity,
L is the path length,
T is the time taken by the pulse to traverse that length.

3.1.2. Equipment for pulse velocity test

The equipment consists essentially of an electrical pulse generator, a pair of transducers, an amplifier and an electronic timing device for measuring the time interval between the initiation of a pulse generated at the transmitting transducer and its arrival at the receiving transducer. Two forms of electronic timing apparatus and display are available, one of which uses a cathode ray tube on which the received pulse is displayed in relation to a suitable time scale, the other uses an interval timer with a direct reading digital display.
The equipment should have the following characteristics. It should be capable of measuring transit time over path lengths ranging from about 100 mm to the maximum thickness to be inspected to an accuracy of ±1%. Generally the transducers used should be in the range of 20 to 150 kHz although frequencies as low as 10 kHz may be used for very long concrete path lengths and as high as 1 MHz for mortars and grouts or for short path lengths.
High frequency pulses have a well defined onset but, as they pass through the concrete, become attenuated more rapidly than pulses of lower frequency. It is therefore preferable to use high frequency transducers for short path lengths and low frequency transducers for long path lengths. Transducers with a frequency of 50 kHz to 60 kHz are suitable for most common applications.



Schematic view of Pulse velocity device

3.1.3. Applications

Measurement of the velocity of ultrasonic pulses of longitudinal vibrations passing through concrete may be used for the following applications (1):

- determination of the uniformity of concrete in and between members
- measurement of changes occurring with time in the properties of concrete
- Correlation of pulse velocity and strength as a measure of concrete quality
- determination of the modulus of elasticity and dynamic Poisson's ratio of the concrete

The velocity of an ultrasonic pulse is influenced by those properties of concrete which determine its elastic stiffness and mechanical strength. The variations obtained in a set of pulse velocity measurements made along different paths in a structure reflect a corresponding variation in the state of the concrete. When a region of low compaction, voids or damaged material is present in the concrete under test, a corresponding reduction in the calculated pulse velocity occurs and this enables the approximate extent of the imperfections to be determined. As concrete matures or deteriorates, the changes, which occur with time in its structure, are reflected in either an increase or a decrease, respectively, in the pulse velocity. This enables changes to be monitored by making tests at appropriate intervals of time.

Pulse velocity measurements made on concrete structures may be used for quality control purposes. In comparison with mechanical tests on control samples such as cubes or cylinders, pulse velocity measurements have the advantage that they relate directly to the concrete in the structure rather than to samples, which may not be always truly representative of the concrete *in situ*.

Ideally, pulse velocity should be related to the results of tests on structural components and, if a correlation can be established with the strength or other required properties of these components, it is desirable to make use of it. Such correlations can often be readily established directly for pre-cast units and can also be found for *in situ* work.
Empirical relationships may be established between the pulse velocity and both the dynamic and static elastic moduli and the strength of concrete. The latter relationship is influenced by a number of factors including the type of cement, cement content, admixtures, type and size of the aggregate, curing conditions and age of concrete. Caution should be exercised when attempting to express the results of pulse velocity tests in terms of strengths or elastic properties, especially at strengths exceeding 60 MPa.

3.1.4. Determination of pulse velocity

3.1.4.1. Transducer arrangement

The receiving transducer detects the arrival of that component of the pulse, which arrives earliest. This is generally the leading edge of the longitudinal vibration. Although the direction in which the maximum energy is propagated is at right angles to the face of the transmitting transducer, it is possible to detect pulses, which have travelled through the concrete in some other direction. It is possible, therefore, to make measurements of pulse velocity by placing the two transducers on either:

- opposite faces (direct transmission)
- adjacent faces (semi-direct transmission): or
- The same face (indirect or surface transmission).

These three arrangements are shown in Figs. 3.1(a), 3.1(b) and3.1(c).

3.1.4.2. Determination of pulse velocity by direct transmission

Where possible the direct transmission arrangement should be used since the transfer of energy between transducers is at its maximum and the accuracy of velocity determination is therefore governed principally by the accuracy of the path length measurement. The couplant used should be spread as thinly as possible to avoid any end effects resulting from the different velocities in couplant and concrete.

3.1.4.3. Determination of pulse velocity by semi-direct transmission

The semi-direct transmission arrangement has a sensitivity intermediate between those of the other two arrangements and, although there may be some reduction in the accuracy of measurement of the path length, it is generally found to be sufficiently accurate to take this as the distance measured from center to center of the transducer faces. This arrangement is otherwise similar to direct transmission.

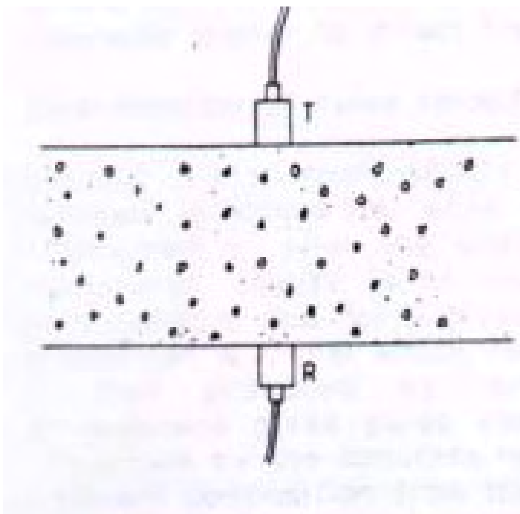


Figure 3.1(a) - Direct transmission

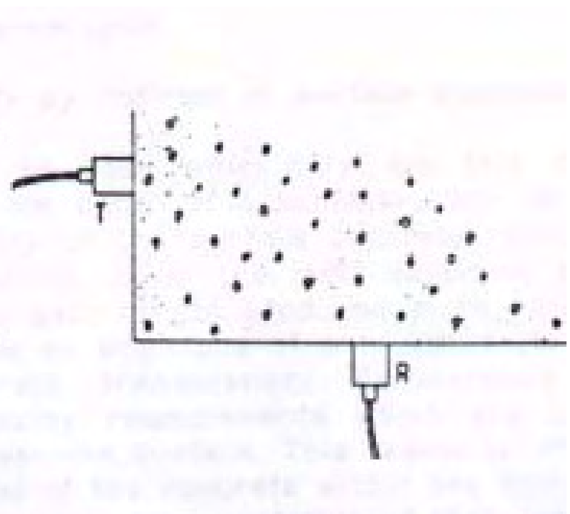


Figure 3.1(b) - Semi-direct transmission

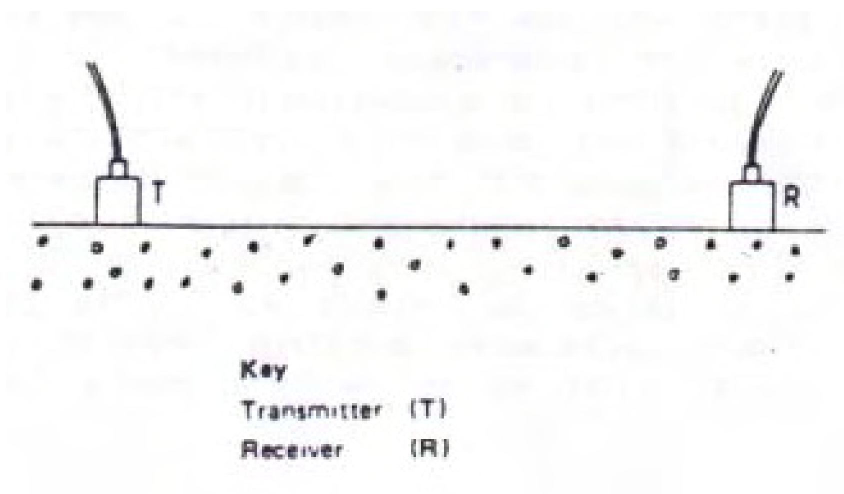


Figure 3.1(c) - Indirect or surface transmission

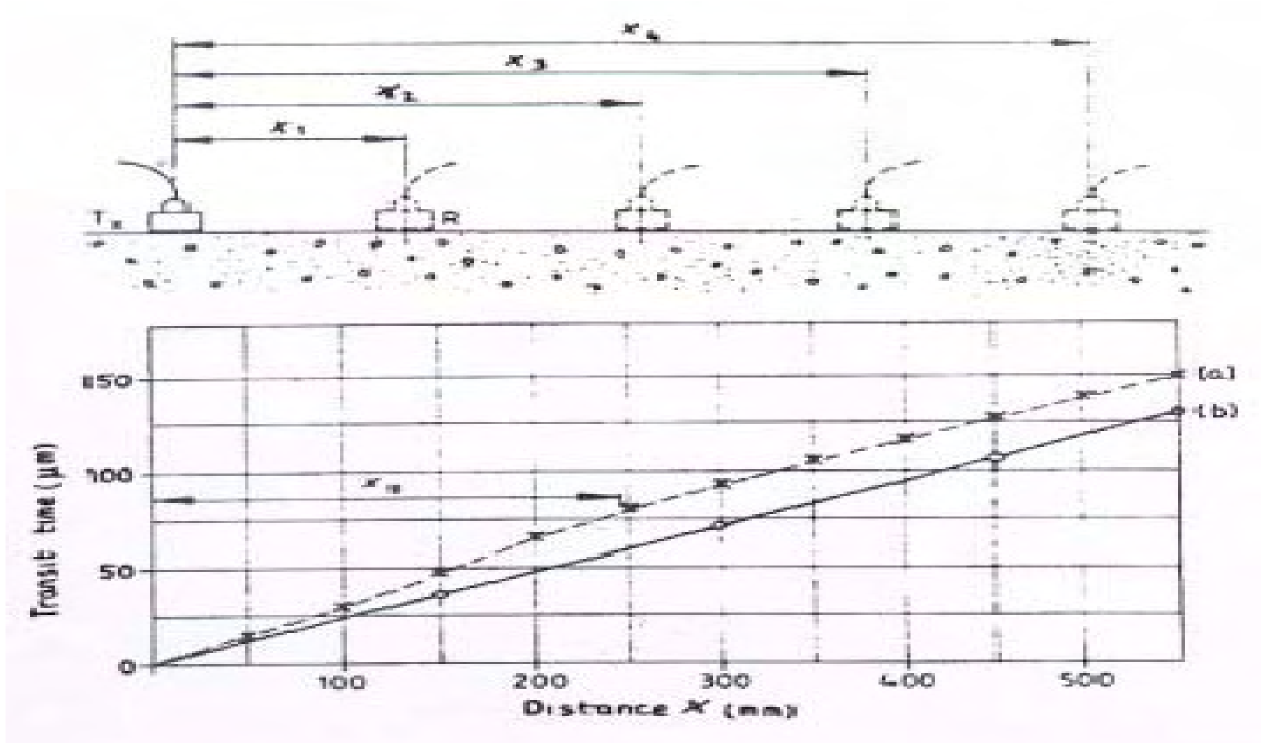
Figure 3.1(a) shows the transducers directly opposite to each other on opposite faces of the concrete. However, it is sometimes necessary to place the transducers on opposite faces but not directly opposite each other. Such an arrangement is regarded as semi-direct transmission, Figure 3.1(b).

3.1.4.4. Determination of pulse velocity by indirect or surface transmission

Indirect transmission should be used when only one face of the concrete is accessible, when the depth of a surface crack is to be determined or when the quality of the surface concrete relative to the overall quality is of interest. It is the least sensitive of the arrangements and, for a given path length, produces at the receiving transducer a signal which has an amplitude of only about 2% or 3% of that produced by direct transmission. Furthermore, this arrangement gives pulse velocity measurements which are usually influenced by the concrete near the surface. This region is often of different composition from that of the concrete within the body of a unit and the test results may be unrepresentative of that concrete. The indirect velocity is invariably lower than the direct velocity on the same concrete element. This difference may vary from 5% to 20% depending largely on the quality of the concrete under test. Where practicable site measurements should be made to determine this difference. With indirect transmission there is some uncertainty regarding the exact length of the transmission path because of the significant size of the areas of contact between the transducers and the concrete. It is therefore preferable to make a series of measurements with the transducers at different distances apart to eliminate this uncertainty. To do this, the transmitting transducer should be placed in contact with the concrete surface at a fixed point x and the receiving transducer should be placed at fixed increments x_n along a chosen line on the surface. The transmission times recorded should be plotted as points on a graph showing their relation to the distance separating the transducers. An example of such a plot is shown as line (b) in Figure 3.2. The slope of the best straight line drawn through the points should be measured and recorded as the mean pulse velocity along the chosen line on the concrete surface. Where the points measured and recorded in this way indicate a discontinuity, it is likely that a surface crack or surface layer of inferior quality is present and a velocity measured in such an instance is unreliable.

3.1.4.5. Coupling the transducer onto the concrete

To ensure that the ultrasonic pulses generated at the transmitting transducers pass into the concrete and are then detected by the receiving transducer, it is essential that there is adequate acoustical coupling between the concrete and the face of each transducer. For many concrete surfaces, the finish is sufficiently smooth to ensure good acoustical contact by the use of a coupling medium and by pressing the transducer against the concrete surface. Typical couplants are petroleum jelly, grease, soft soap and kaolin/glycerol paste. It is important that only a very thin layer of coupling medium separates the surface of the concrete from its contacting transducer. For this reason, repeated readings of the transit time should be made until a minimum value is obtained so as to allow the layer of the couplant to become thinly spread.
Where possible, the transducers should be in contact with the concrete surfaces, which have been cast against formwork or a mold. Surfaces formed by other means, e.g. trowelling, may have properties differing from those of the main body of material. If it is necessary to work on such a surface, measurements should be made over a longer path length than would normally be used. A minimum path length of 150 mm is recommended for direct transmission involving one unmolded surface and a minimum of 400 mm for indirect transmission along one unmolded surface.



(a) Results for concrete with the top 50 mm of inferior quality
(b) Results for homogeneous concrete

Figure 3.2- Pulse velocity determination by indirect (surface) transmission

When the concrete surface is very rough and uneven, the area of the surface where the transducer is to be applied should be smoothed and leveled. Alternately, a smoothing medium such as quick setting epoxy resin or plaster may be used, but good adhesion between the concrete surface and the smoothing medium has to be ensured so that the pulse propagates correctly into the concrete under test. It is important to ensure that the layer of smoothing medium is as thin as possible. If it is necessary to make a significant build up then the pulse velocity of the smoothing medium has to be taken into account.

3.1.5. Factors influencing pulse velocity measurements

3.1.5.1. Moisture content

The moisture content has two effects on the pulse velocity, one chemical the other physical. These effects are important in the production of correlations for the estimation of concrete strength. Between a properly cured standard cube and a structural element made from the same concrete, there may be a significant pulse velocity difference. Much of the difference is accounted for by the effect of different curing conditions on the hydration of the cement while some of the difference is due to the presence of free water in the voids. It is important that these effects are carefully considered when estimating strength.

3.1.5.2. Temperature of the concrete

Variations of the concrete temperature between 10˚C and 30˚C have been found to cause no significant change without the occurrence of corresponding changes in the strength or elastic properties. Corrections to pulse velocity measurements should be made only for temperatures outside this range as given in Table 3.1.

Table 3.1- Effect of temperature on pulse transmission

Correlation to the measured pulse velocity		
Temperature	Air dried concrete	Water saturated concrete
˚C	%	%
60	+5	+4
40	+2	+1.7
20	0	0
0	-0.5	-1
-4	-1.5	-7.5

3.1.5.3. Path length

The path length over which the pulse velocity is measured should be long enough not to be significantly influenced by the heterogeneous nature of the concrete. It is recommended that, except for the conditions stated in 3.1.4.5, the minimum path length should be 100 mm for concrete where nominal maximum size of aggregate is 20 mm or less and 150 mm for concrete where nominal maximum size of aggregate is between 20 mm and 40 mm. The pulse velocity is not generally influenced by changes in path length, although the electronic timing apparatus may indicate a tendency for velocity to reduce slightly with increasing path length. This is because the higher frequency components of the pulse are attenuated more than the lower frequency components and the shape of the onset of the pulse becomes more rounded with increased distance traveled. Thus, the apparent reduction of pulse velocity arises from the difficulty of defining exactly the onset of the pulse and this depends on the particular method used for its definition. This apparent reduction in velocity is usually small and well within the tolerance of time measurement accuracy for the equipment.

3.1.5.4. Shape and size of specimen

The velocity of short pulses of vibration is independent of the size and shape of the specimen in which they travel, unless its least lateral dimension is less than a certain minimum value. Below this value, the pulse velocity may be reduced appreciably. The extent of this reduction depends mainly on the ratio of the wavelength of the pulse vibrations to the least lateral dimension of the specimen but it is insignificant if the ratio is less than unity. Table 3.2 gives the relationship between the pulse velocity in the concrete, the transducer frequency and the minimum permissible lateral dimension of the specimen. If the minimum lateral dimension is less than the wavelength or if the indirect transmission arrangement is used, the mode of propagation changes and therefore the measured velocity will be different. This is particularly important in cases where concrete elements of significantly different sizes are being compared.

Table3.2. Effect of specimen dimension on pulse transmission

Transducer frequency	Pulse velocity in concrete (km/s)		
	v_c	v_c	v_c
Minimum permissible lateral specimen dimension			
KHz	mm	mm	mm
24	146	167	188
54	65	74	83
82	43	49	55
150	23	27	30

3.1.5.5. Effect of reinforcing bars

The pulse velocity measured in reinforced concrete in the vicinity of reinforcing bars is usually higher than in plain concrete of the same composition. This is because the pulse velocity in steel may be up to twice the velocity in plain concrete and, under certain conditions, the first pulse to arrive at the receiving transducer travels partly in concrete and partly in steel. The apparent increase in pulse velocity depends on the proximity of the measurements to the reinforcing bar, the diameter and number of bars and their orientation with respect to the propagation path. The frequency of the pulse and surface conditions of the bar may both also affect the degree to which the steel influences the velocity measurements. Corrections to measured values to allow for reinforcement will reduce the accuracy of estimated pulse velocity in the concrete so that, wherever possible, measurements should be made in such a way that steel does not lie in or close to the direct path between the transducers.

3.1.5.6. Determination of concrete uniformity

Heterogeneities in the concrete within or between members cause variations in pulse velocity, which in turn are related to variations in quality. Measurements of pulse velocity provide a means of studying the homogeneity and for this purpose a system of measuring points which covers uniformly the appropriate volume of concrete in the structure has to be chosen.

The number of individual test points depends upon the size of the structure, accuracy required and variability of the concrete. In a large unit of fairly uniform concrete, testing on a 1m grid is usually adequate but, on small units or variable concrete, a finer grid may be necessary. It should be noted that, in cases where the path length is the same throughout the survey, the measured time might be used to assess the concrete uniformity without the need to convert it to velocity. This technique is particularly suitable for surveys where all the measurements are made by indirect measurements.

It is possible to express homogeneity in the form of a statistical parameter such as the standard deviation or coefficient of variation of the pulse velocity measurements made over a grid. However, such parameters can only be properly used to compare variations in concrete units of broadly similar dimensions.

Variations in pulse velocity are influenced by the magnitude of the path length because this determines the effective size of the concrete sample, which is under examination during each measurement. The importance of variations should be judged in relation to the effect which they can be expected to have on the required performance of the structural member being tested. This generally means that the tolerance allowed for quality distribution within members should be related either to the stress distribution within them under critical working load conditions or to exposure conditions.

3.1.6. Detection of defects

The use of the ultrasonic pulse velocity technique to detect and define the extent of internal defects should be restricted to well-qualified personnel with previous experience in the interpretation of survey results. Attention is drawn to the potential risk of drawing conclusions from single results.

When an ultrasonic pulse travelling through concrete meets a concrete-air interface there is negligible transmission of energy across this interface. Thus any air filled void lying immediately between transducers will obstruct the direct ultrasonic beam when the projected length of the void is greater than the width of the transducers and the wavelength of sound used. When this happens the first pulse to arrive at the receiving transducer will have been diffracted around the periphery of the void and the transit time will be longer than in similar concrete with no void.

It is possible to make use of this effect for locating flaws, voids or other defects greater than about 100 mm in diameter or depth. Relatively small defects have little or no effect on transmission times but equally are probably of minor engineering importance. Plotting contours of equal velocity often gives significant information regarding the quality of a concrete unit. The method used to detect a void is to draw a grid on the concrete with its points of intersection spaced to correspond to the size of void that would significantly affect the concrete performance. A survey of measurements at the grid points enables a large cavity to be investigated by measuring the transit times of pulses passing between the transducers when they are placed so that the cavity lies in the direct path between them. The size of such cavities may be estimated by assuming that the pulses pass along the shortest path between the transducers and around the cavity. Such estimates are valid only when the concrete around the cavity is uniformly dense and the pulse velocity can be measured in that concrete.

The method is not very successful when applied to structures with cracks because the cracked faces are usually sufficiently in contact with each other to allow the pulse energy to pass unimpeded across the crack. This can happen in cracked vertical bearing piles where there is also sufficient compression to hold the faces close together. If the concrete is surrounded by water such that the crack is filled with water, the crack is undetectable since ultrasonic energy can travel through a liquid.

3.1.6.1. Estimating the thickness of a layer of inferior quality concrete

If concrete is suspected of having a surface layer of poor quality because of poor manufacture, or damage by fire, frost or sulphate attack, the thickness of the layer can be estimated from ultrasonic measurements of transit times along the surface.

The technique used is to place the transmitting transducer on the surface and the receiving transducer a distance “x_1” from the transmitting transducer. The transit time is measured and

then measured again at distances of "x_2", "x_3", etc. The transit times are plotted against distance as in Fig. 3.2 in which x is 50 mm. At the shorter distance of separation of the transducers, the pulse travels through the surface layer and the slope of the experimental line gives the pulse velocity in this surface layer. Beyond a certain distance of separation the first pulse to arrive has passed along the surface of the underlying higher quality concrete and the slope of these experimental points gives the velocity in that concrete.
The distance x_o at which the change of slope occurs together with the measured pulse velocities in the two different layers of concrete, enables an estimate of the thickness t (mm) of the surface layer to be made using the equation below.

$$t = \frac{X_0}{2}\sqrt{\frac{(vs - vd)}{(vs + vd)}} \tag{3-2}$$

Where

vd	is the pulse velocity in the damaged concrete (km/s),
vs	is the pulse velocity in the underlying sound concrete (km/s),
X_o	is the distance from the transmitter at which the slope changes (mm)

The method is applicable to extensive surface areas in which the inferior concrete forms a distinct layer of fairly uniform thickness. Localized areas of damaged or honeycombed concrete are more difficult to test but it is possible to derive an approximate thickness of such localized poor quality material if both direct transmission and surface propagation measurements are made.

3.1.6.2. Determination of changes in concrete properties

Pulse velocity measurements are particularly useful to follow the hardening process, especially during the first 36 h. Here, rapid changes in pulse velocity are associated with physiochemical changes in the cement paste structure, and it is necessary to make measurements at intervals of 1 h or 2 h if these changes are to be followed closely. As the concrete hardens these intervals may be lengthened to 1 day or more after the initial period of 36 h has elapsed.
Measurements of changes in pulse velocity are usually indicative of changes in strength and have the advantage that they can be made over progressive periods of time on the same test piece throughout the investigation. Since the quality of concrete is usually specified in terms of strength; it is, therefore, sometimes helpful to use ultrasonic pulse velocity measurements to give an estimate of strength. The relationship between ultrasonic pulse velocity and strength is affected by a number of factors including age, curing conditions, moisture condition, mix proportions, type of aggregate and type of cement. If an estimate of strength is required it is necessary to establish a correlation between strength and velocity for the particular type of concrete under investigation. This correlation has to be established experimentally by testing a sufficient number of specimens to cover the range of strengths expected and to provide statistical reliability. The confidence that can be ascribed to the results will depend on the number of samples tested. It is possible to establish a correlation between ultrasonic pulse velocity and strength either as measured in accordance with compressive strength tests or by carrying out tests on a complete structure or unit. The reliability of the correlation will depend on the extent to which the correlation specimens represent the structure to be investigated. The most appropriate

correlation will be obtained from tests in which the pulse velocity and strength are measured on a complete structure or unit. It is sometimes more convenient to prepare a correlation using tests on molded specimens. It should be noted that experience has shown that a correlation based on molded specimens generally gives a lower estimate of strength than would be obtained by cutting and testing cores.

3.1.6.3. Examples of relationships between pulse velocity and compressive strength

Some figures suggested by Whitehurst for concrete with a density of approximately 2400 kg/m³ are given in Table 3.3. According to Jones, however, the lower limit for good quality concrete is between 4.1 and 4.7 km/s. Fig.3.3, based on Jones' results, and illustrate this effect. Despite this relationship between pulse velocity and compressive strength, ultrasonic pulse velocity measurements are not usually used as a means of quality control on construction sites. Unfortunately there is no satisfactory correlation between the variability of the compression test samples, be they cubes or cylinders, and the variability of the pulse velocity measurements.

Table 3.3- Classification of the quality of concrete on the basis of pulse velocity

Longitudinal pulse Velocity		Quality of concrete
km/s.10³	ft/s	
>4.5	>15	excellent
3.5-4.5	12-15	good
3.0-3.5	10-12	doubtful
2.0-3.0	7-10	poor
<2.0	<7	Very poor

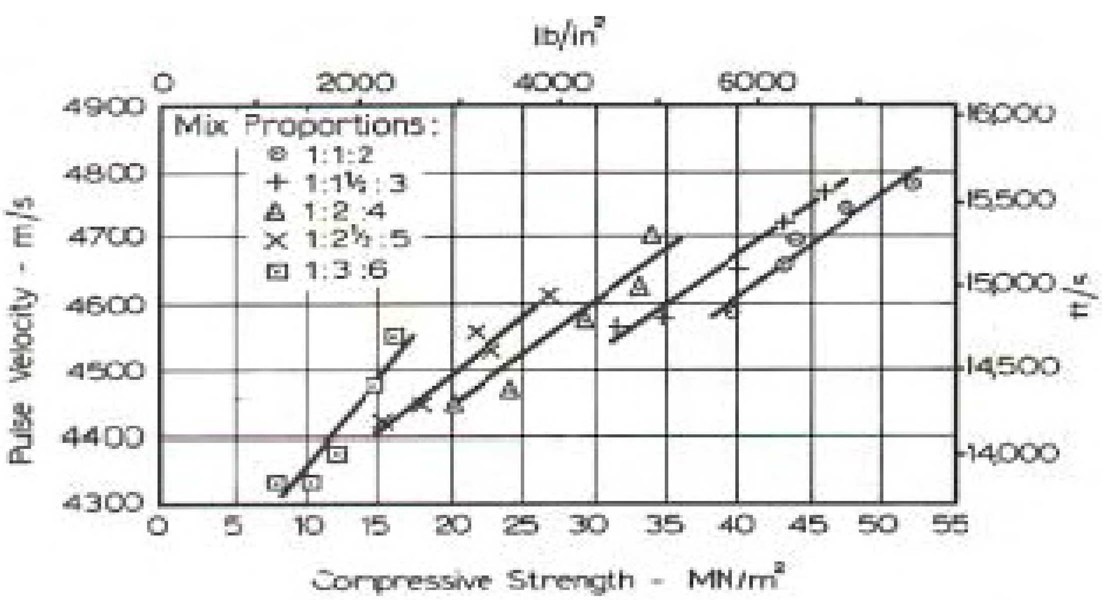


Figure 3.3- Relation between ultrasonic pulse velocity and compressive strength for concretes of different mix proportions

3.1.6.4. Determination of the modulus of elasticity and dynamic Poisson's ratio

The relationship between these elastic constants and the velocity of an ultrasonic pulse travelling in an isotropic elastic medium of infinite dimensions is given below:

$$E_d = \frac{\rho v^2 (1+v)(1-2v)}{(1-v)} \tag{3-3}$$

Where

E_d is the dynamic elastic modulus (MN/m²),
ν is the dynamic Poisson's ratio,
ρ is the density (kg/m³),
V is the pulse velocity (km/s).

If the values of ν and ρ are known, it is possible to use above equation (3-3) to determine the value of E_d in concrete samples for a wide range of shapes or sizes. This is because the pulse velocity is not significantly affected by the dimensions of the test specimen, except when one or more of the lateral dimensions is small relative to the wavelength of the pulse. Similarly ν could be determined if the values of ρ and E_d are known.

3.1.7. Advantages and Disadvantages

The pulse velocity method is an excellent means for investigating the uniformity of concrete. Advantages are:

- rapidly survey large areas and thick members,
- Simple, and the equipment is readily available,
- Portable and it is as easy to use on the construction site and as it is in the laboratory.

Testing procedures have been standardized by ASTM and other organizations. Because the pulse velocity is truly non destructive and several tests can be run in a short amount of time, this equipment is becoming more popular as a means for estimating early age concrete strength development.
A large number of variables can affect the relation between the strength properties of concrete and its pulse velocity; therefore, it is important that a correlation between pulse velocity and compressive strength be developed for project mixes prior to any measurements *in-situ* (2).
Disadvantages are:

- Require proper surface preparation,
- Time consuming as it takes only point measurements,
- Skill is required in the analysis of results as moisture variations and presence of metal reinforcement can affect results,
- The interpretation of ultrasonic test results based on published graphs and tables can be misleading. It is therefore necessary that correlation with the concrete be inspected is carried out. It works on single homogenous materials.

3.2. Surface Hardness Test

The rebound hammer has been used to estimate the in-situ compressive strength of concrete. It has also been used to assess the overall uniformity of concrete prior to undertaking more extensive destructive tests, such as coring. The rebound hammer is easy to use and provides a large number of readings in a short time. However, extreme care should be taken in evaluating the results. Frequent calibration of the hammer is also required to ensure the greatest accuracy. The rebound hammer test is basically a surface hardness tester with little apparent theoretical relationship between the strength of concrete and the rebound number of the hammer. However, within certain limits, empirical correlations have been established between compressive strength and the rebound number. In general, most investigators have found that the accuracy of the rebound hammer is between 60 and 70 percent (3).

3.2.1. Test equipment and procedure

A typical rebound hammer is shown in figure 3.4. The hammer weights about 4 lb (1.8 kg) and can be used in the field and laboratory. Figure 3.5 contains a schematic view of rebound hammer, showing its main components.
To perform a rebound test, release the plunger from its locked position by gently pushing the plunger against a hard surface and slowly allow the spring to push the body of the hammer away from the hard surface. This causes the plunger to extend from the hammer body, allowing the latch to engage the spring-loaded steel hammer and plunger rod (figure 3.5 (a)). Hold the plunger perpendicular to the concrete surface to be tested and slowly push the hammer toward the surface. As the hammer is pushed toward the concrete surface, the main spring connecting the hammer mass to the plunger is stretched (figure 3.5 (b)) .When the hammer is pushed to the limit, the latch is automatically released, and the energy stored in the spring propels the hammer mass toward the plunger tip (figure 3.5 (c)). The mass impacts the shoulder of the plunger rod and rebounds (figure 3.5 (d)). During rebound, the slide indicator travels with the hammer mass and records the rebound distance. A button on the side of the hammer is pushed to lock the plunger in the retracted position, and the rebound number is read from the scale. The rebound distance is indicated by a pointer on a scale graduated from 0 to 100; the rebound readings are termed “R-values”. These values give an indication of the concrete surface hardness with values increasing with the hardness of the concrete.

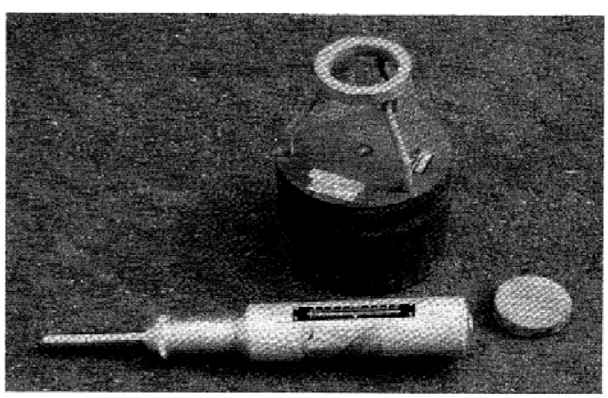


Figure 3.4-Rebound Hammer

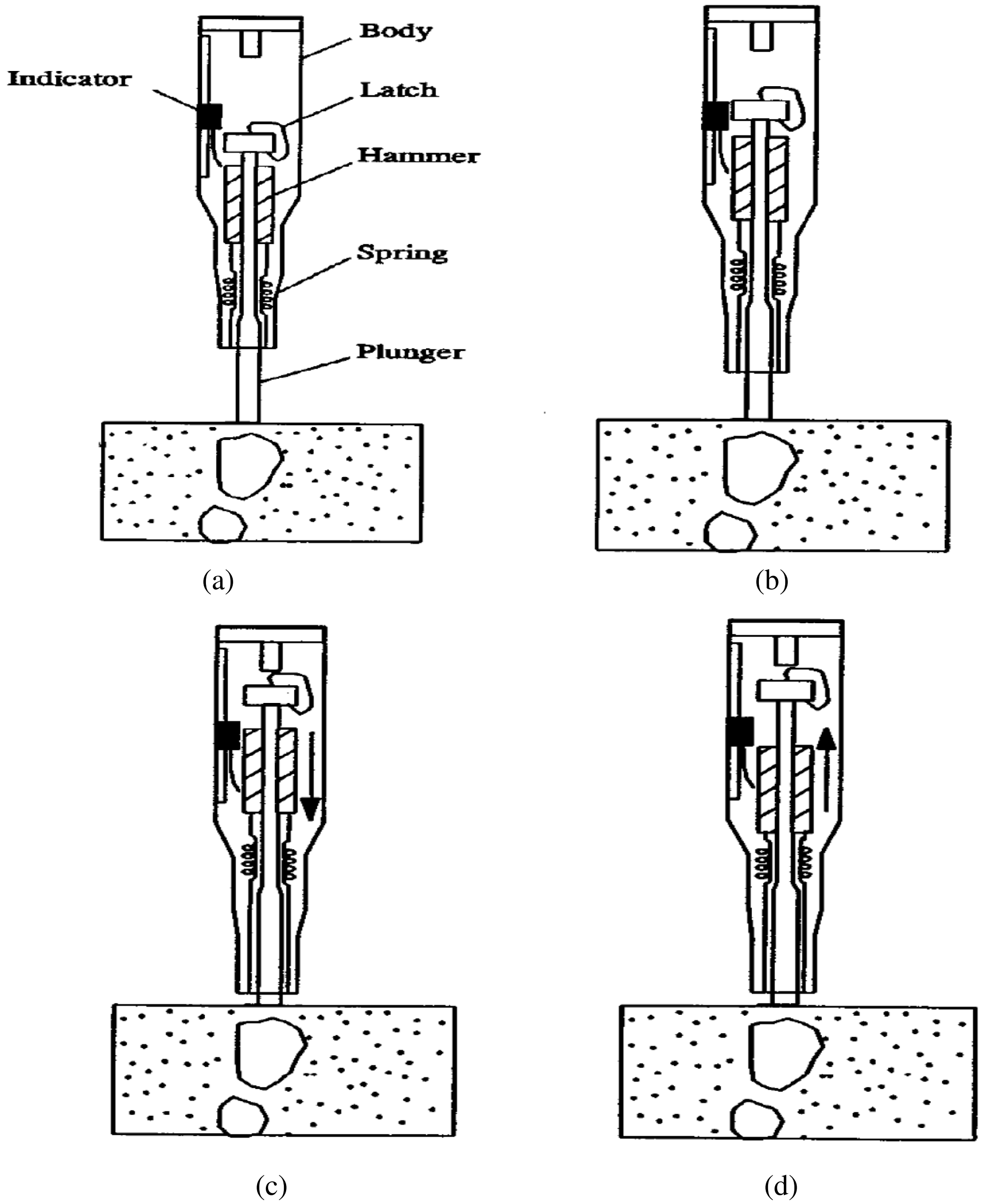


Figure 3.5-schematic of rebound hammer operation

Estimate the rebound number on the scale to the nearest whole number and record the rebound number. Take 10 readings from each test area. No two impact tests shall be closer than 25 mm (1 in). Examine the impression made on the surface after impact; if the impact crushes or breaks through a near-surface air void, disregard the reading and take another reading.

Discard readings differing from the average of 10 readings by more than 6 units and determine the average of the remaining reading. If more than 2 readings differ from the average by 6 units, discard the entire set of readings and determine rebound numbers at 10 new locations within the test area (4).

The test can be conducted horizontally, vertically upward or downward, or at any intermediate angle. Due to different effects of gravity on the rebound hammer mass as the angle is changed, the rebound number will be different for the same concrete, and requires a separate calibration or correction chart for each test angle.

3.2.2. Advantages and Disadvantages:

Advantages are:

- Provide a quick and inexpensive means of assessing the general quality of concrete and for locating areas of poor quality
- Taken large number of readings rapidly, so can scan large exposed areas in a few hours

Disadvantages are:

- Because the test only measures the rebound of a given mass on the concrete surface, the results reflect only the quality of the surface, not the entire depth,
- The results of the test are affected by the smoothness of the test surface, type of coarse aggregate, age of concrete being tested, moisture content, type of cement, and surface carbonation.

A brief explanation of how these factors affect the result of the hammer rebound test is given below.

Surface Smoothness:

Surface texture can have an important effect on the accuracy of test results. If a rebound test is performed on a rough-textured surface, the plunger tip causes excessive crushing of the cement paste, which will result in the reduction of the rebound number measured. To obtain more accurate results on rough surfaces, a carborundum stone should be used to grind the surface to a uniform smoothness. Past research has also shown that troweled surfaces or surfaces formed by metal forms yield rebound numbers 5 to 25 percent higher than surfaces cast against wooden forms (3). Troweled surfaces also give a higher scatter of test results, which lower confidence in the estimated strength results.

Age of Material Being Tested:

The rate of gain of surface hardness of concrete is rapid for the first 7 days, after which there is little or no gain in surface hardness. However, for properly cured concrete, there is a significant strength gain beyond 7 days, because cement continues to hydrate within the concrete and gain strength. When concrete over 28 days is to be tested, direct correlations need to be developed between the rebound numbers taken on the concrete and the compressive strength of cores taken from the concrete.

Caution should also be exercised when testing concrete less than 3 days old or concrete with expected compressive strengths less than 7 Mpa (1000 psi). The reason for this is that the rebound numbers will be too low for an accurate reading, and the rebound hammer will leave blemishes on the concrete surface when impacted.

Moisture Content:

The presence of surface moisture and the overall moisture content of the concrete have a profound effect on the results of the rebound hammer test. Well-cured, air-dried specimens that have been soaked in water and tested in the saturated surface-dry (SSD) condition generally show rebound numbers 5 points lower than air-dried specimens. When SSD specimens were left in a room at 21°C (70 ° F)and air-dried ,they gained 3 points in 3 days and 5 points in 7 days (3). To achieve the most accurate results for specimens where the actual moisture condition is unknown, the surface should be pre-saturated with water several hours prior to testing and use the correlation developed for SSD specimens.

Type of Cement:

The type of cement can have a significant effect on the rebound number. Concrete containing type 3 high-early strength cement can have higher rebound numbers at an early age than concrete made with type 1 cement.

Carbonation of Concrete Surface:

The rebound numbers for carbonated concrete can be up to 50 percent higher than those obtained on a non-carbonated concrete surface. The carbonation effects are more severe in older concretes where the carbonated layer can be several millimeters thick and in extreme cases up to 20 mm (3/4 in) thick. To achieve more accurate results, correction factors need to be established for specific concrete being tested.

Type of Coarse Aggregate:

For equal compressive strengths, concrete made with crushed limestone shows rebound numbers approximately 7 points higher than those for concretes made with gravel, representing approximately 7 Mpa (1000 psi) difference in compressive strength. The same type of coarse aggregate obtained from different sources can yield different concrete strength estimations. Correlation testing of materials is necessary.

Interpretation of test Results

A general correlation exists between the compressive strength of concrete and the hammer rebound number. However, there is a big disagreement among researchers concerning the accuracy of the hammer for estimating the compressive strength of concrete. Coefficients of variation for compressive strength of concrete can vary from 15 percent to over 30 percent for a wide variety of specimens. These large deviations can be reduced by developing a proper correlation curve for the hammer that takes into account the variables discussed earlier, instead of relying on the correlation curves provided by the manufacturer of the rebound hammer.
For a properly calibrated hammer the accuracy is between 15 and 20 percent for test specimens cast, cured, and tested under lab conditions. However, the accuracy of the rebound hammer for estimating in-situ compressive strength is approximately 30 to 40 percent.

3.2.3. Summary

The Schmidt hammer should not be regarded as a substitute for standard compression tests but as a method for determining the uniformity of concrete in structures, and comparing one concrete against another. Estimation of the strength of concrete by the rebound hammer within an accuracy of ± 15 to 20 percent may be possible only for specimens cast, cured, and tested under similar conditions as those from which the correlation curves are established.

References

(1) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2002 "*Guidebook on non-destructive testing of concrete structures*" TRAINING COURSE SERIES No. 17
(2) G.I.Crawford, September 1997,U.S. Department of Transportation, Federal Highway Administration, "*Guide to Nondestructive Testing of Concrete*".
*(3)*Malhotra,V.M.,Chapter 1*,"Surface Hardness Methods, Handbooks of Nondestructive Testing of Concrete",*CRC Press,Inc.,Boca Raton,FL,1991,pp. 1-17
*(4)*ASTM Designation C 805 -94*,"Standard Test Method For Rebound Number of Hardened Concrete","1995 Annual Book Of ASTM Standards",Volume 04.02,"Concrete And Concrete Aggregates",* American Society For Testing And Materials,Philadelphia ,PA.

Strength of Concrete

3.1. Penetration Resistance or Windsor Probe Test

4.1.1. Fundamental Principle

The Windsor probe, like the rebound hammer, is a hardness tester, and its inventors' claim that the penetration of the probe reflects the precise compressive strength in a localized area is not strictly true. However, the probe penetration does relate to some property of the concrete below the surface, and, within limits, it has been possible to develop empirical correlations between strength properties and the penetration of the probe (1). The Windsor probe is semi-destructive, but since in some literature they are classified as nondestructive technique (2).
The penetration technique essentially uses a powder-actuated gun or driver which fires a hardened alloy probe into the concrete. The Windsor probe testing system is the most widely known penetration resistance device available for both laboratory and in situ measurements.

Figure 4.1- Windsor probe system by James NDT Product (USA)

4.1.2. Equipment for Windsor Probe Test

The Windsor probe consists of a powder-actuated gun or driver, hardened alloy steel probes, loaded cartridges, a depth gauge for measuring the penetration of probes, and other related equipment. As the device looks like a firearm it may be necessary to obtain official approval for its use in some countries. The probes have a tip diameter of 6.3 mm, a length of 79.5 mm, and a conical point. Probes of 7.9 mm diameter are also available for the testing of concrete made with lightweight aggregates. The rear of the probe is threaded and screws into a probe driving head, which is 12.7 mm in diameter and fits snugly into the bore of the driver. The probe is driven into the concrete by the firing of a precision powder charge that develops energy of 79.5 m kg. For the testing of relatively low strength concrete, the power level can be reduced by pushing the driver head further into the barrel.

4.1.3. General Procedure for Windsor Probe Test

The area to be tested must have a brush finish or a smooth surface. To test structures with coarse finishes, the surface first must be ground smooth in the area of the test. Briefly, the powder-actuated driver is used to drive a probe into the concrete. If flat surfaces are to be tested a suitable locating template to provide 178 mm equilateral triangular pattern is used, and three

probes are driven into the concrete, one at each corner. A depth gauge measures the exposed lengths of the individual probes. The manufacturer also supplies a mechanical averaging device for measuring the average exposed length of the three probes fired in a triangular pattern. The mechanical averaging device consists of two triangular plates. The reference plate with three legs slips over the three probes and rests on the surface of the concrete. The other triangular plate rests against the tops of the three probes. The distance between the two plates, giving the mechanical average of exposed lengths of the three probes, is measured by a depth gauge inserted through a hole in the centre of the top plate. For testing structures with curved surfaces, three probes are driven individually using the single probe locating template. In either case, the measured average value of exposed probe length may then be used to estimate the compressive strength of concrete by means of appropriate correlation data.
The manufacturer of the Windsor probe test system has published tables relating the exposed length of the probe with the compressive strength of the concrete. For each exposed length value, different values for compressive strength are given, depending on the hardness of the aggregate as measured by the Mohs' scale of hardness. The tables provided by the manufacturer are based on empirical relationships established in his laboratory. However, investigations carried out by Gaynor, Arni, Mallotra, and several others indicate that the manufacturer's tables do not always give satisfactory results (1). Sometimes they considerably overestimate the actual strength and in other instances they underestimate the strength.
It is, therefore, imperative for each user of the probe to correlate probe test results with the type of concrete being used. Although the penetration resistance technique has been standardized the standard does not provide a procedure for developing a correlation. A practical procedure for developing such a relationship is outlined below.

(1) Prepare a number of 150 mm × 300 mm cylinders, or 150 mm^3 cubes, and companion 600 mm × 600 mm × 200 mm concrete slabs covering a strength range that is to be encountered on a job site. Use the same cement and the same type and size of aggregates as those to be used on the job. Cure the specimens under standard moist curing conditions, keeping the curing period the same as the specified control age in the field.
(2) Test three specimens in compression at the age specified, using standard testing procedure. Then fire three probes into the top surface of the slab at least 150 mm apart and at least 150 mm in from the edges. If any of the three probes fails to properly penetrate the slab, remove it and fire another. Make sure that at least three valid probe results are available. Measure the exposed probe lengths and average the three results.
(3) Repeat the above procedure for all test specimens.
(4) Plot the exposed probe length against the compressive strength, and fit a curve or line by the method of least squares. The 95% confidence limits for individual results may also be drawn on the graph. These limits will describe the interval within which the probability of a test result falling is 95%.

A typical correlation curve is shown in Fig.4.2, together with the 95% confidence limits for individual values. The correlation published by several investigators for concrete made with limestone, gravel, chert, and traprock aggregates are shown in Fig.4.3. Note that different relationships have been obtained for concrete with aggregates having similar Mohs' hardness numbers.

4.1.4. Applications of Windsor Probe Test

4.1.4.1. Formwork removal

The Windsor probe test has been used to estimate the early age strength of concrete in order to determine when formwork can be removed. The simplicity of the test is its greatest attraction. The depth of penetration of the probe, based on previously established criteria, allows a decision to be made on the time when the formwork can be stripped.

4.1.4.2. As a substitute for core testing

If the standard cylinder compression tests do not reach the specified values or the quality of the concrete is being questioned because of inadequate placing methods or curing problems, it may be necessary to establish the *in situ* compressive strength of the concrete. This need may also arise if an older structure is being investigated and an estimate of the compressive strength is required. In all those situations the usual option is to take a drill core sample since the specification will generally require a compressive strength to be achieved. It is claimed, however, that the Windsor probe test is superior to taking a core. With a core test, if ASTM C42 –87 is applied, the area from which the cores are taken needs to be soaked for 40 h before the sample is drilled. Also the sample often has to be transported to a testing laboratory which may be some distance from the structure being tested and can result in an appreciable delay before the test result is known. Swamy and Al-Hamed report that the Windsor probe estimated the wet cube strength to be better than small diameter cores for ages up to 28 days. For older concrete the cores estimated the strength better than the probe.

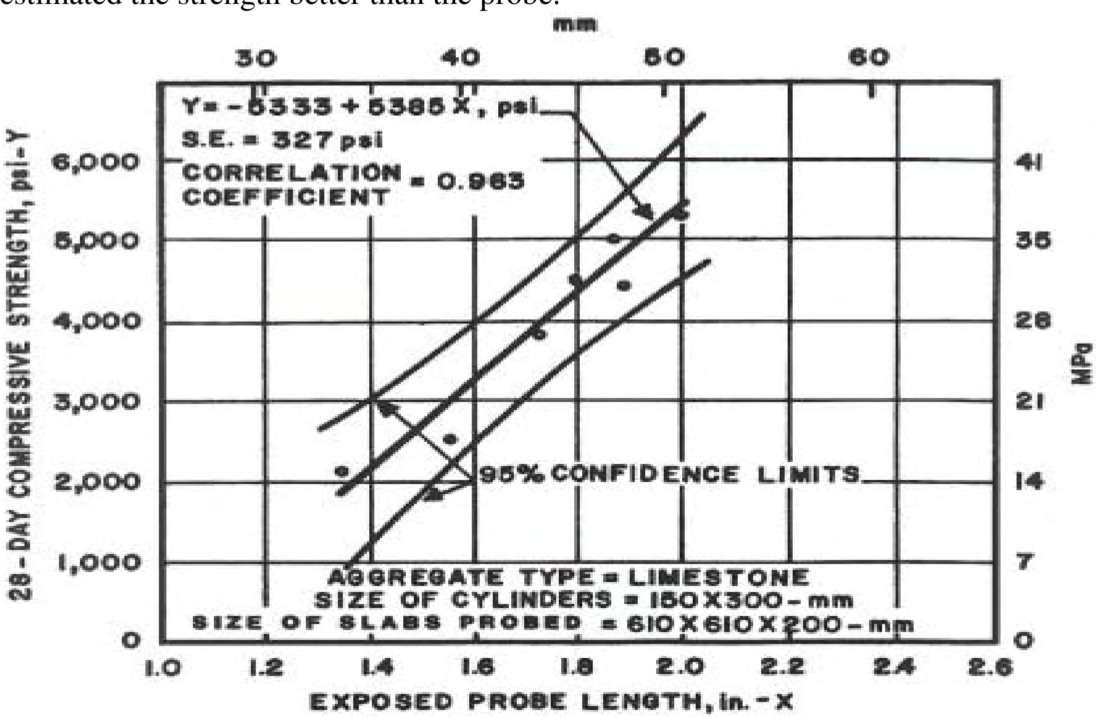


Figure 4.2- Relationship between exposed probe length and 28 day compressive strength of concrete

4.1.5. Factors Affecting Probe Test Results

The coarse aggregate hardness has a profound effect on the accuracy of the probe test for estimating the compressive strength. The equipment manufacturer has made an effort to account for this in the correlation tables by developing values based on the hardness of the aggregate. Several researchers have found varying concrete strength for aggregate with similar Moh's hardness numbers. This implies that other factors in addition to aggregate hardness affect the probe penetration. Mortar strength also has a large effect on the compressive strength at early ages. Apart from its hardness, the type and size of coarse aggregate will also have a significant effect on probe penetration. Other parameters that affect the correlation are mix proportions, moisture content of hardened concrete, curing conditions, surface conditions, degree of carbonation, and age of the concrete (3).

The penetration resistance test is generally considered non-destructive; however, the probe leaves a minor hole in the concrete for the depth of the probe penetration 25 to 63.5 mm (1-2.5 in). For more mature concrete a cone-shaped area of concrete may be heavily fractured around the probe. For exposed surfaces the probe would have to be removed and the surface patched. The test is considered non-destructive to the extent that concrete can be tested *in-situ*, and the strength integrity of the concrete is not affected significantly by the test.

4.1.6. Advantages and Limitations

The advantages are:

- The test is relatively quick and the result is achieved immediately provided an appropriate correlation curve ,
- The probe is simple to operate, requires little maintenance except cleaning the barrel and is not sensitive to operator technique,
- Access is only needed to one surface,
- The correlation with concrete strength is affected by a relatively small number of variables,
- The equipment is easy to use and does not require surface preparation prior to testing,
- It is good for determining in situ quality of concrete,
- The results are not subject to surface conditions, moisture content or ambient temperature.
- The test result is likely to represent the concrete at a depth of from 25 mm to 75 mm from the surface rather than just the property of the surface layer as in the Schmidt rebound test.

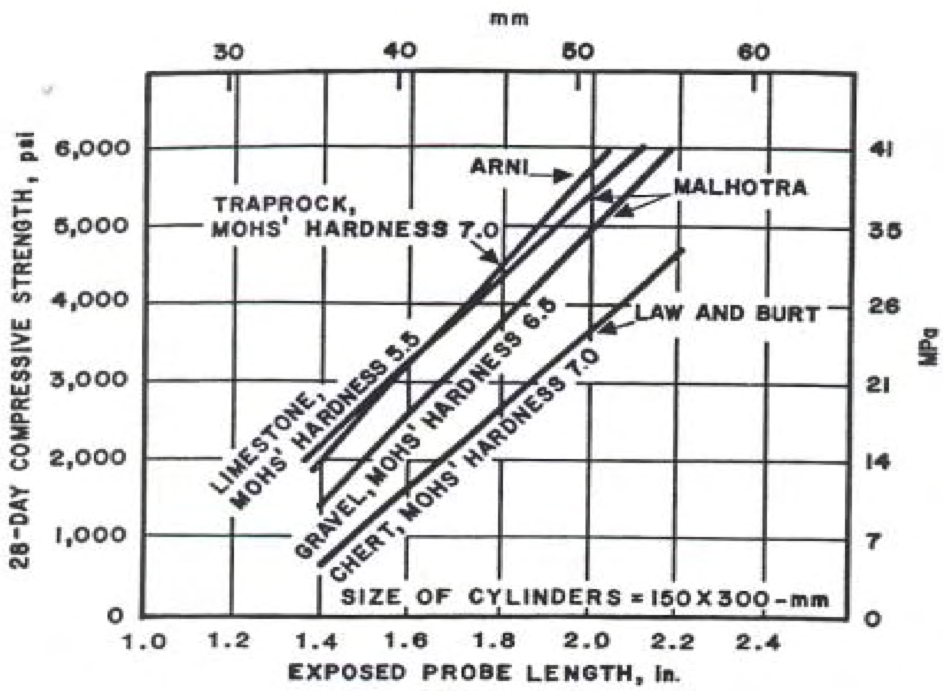


Figure 4.3- Relation between exposed probe length and compressive strength for different coarse aggregates.

The limitations are:

- The minimum acceptable distance from a test location to any edges of the concrete member or between two test locations is of the order of 150 mm to 200 mm,
- The minimum thickness of the member, which can be tested, is about three times the expected depth of probe penetration,
- The distance from reinforcement can also have an effect on the depth of probe penetration especially when the distance is less than about 100 mm,
- The test is limited to <40 Mpa and if two different powder levels are used in an investigation to accommodate a larger range of concrete strengths, the correlation procedure becomes complicated,
- The test leaves an 8 mm hole in the concrete where the probe penetrated and, in older concrete, the area around the point of penetration is heavily fractured,
- On an exposed face the probes have to be removed and the damaged area repaired,
- It slightly damages small area,
- Calibration by manufacturers does not give precise prediction of strength for concrete older than 5 years and where surface is affected by carbonation or cracking.
- Calibration based on cover is necessary for improved evaluation.

References

(1) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2002 "*Guidebook on non-destructive testing of concrete structures"* TRAINING COURSE SERIES No. 17
*(2)*IAEA, International Atomic Agency, Vienna 2005 *"non-destructive testing for plant life assessment "*Training Course Series No.26
(3) ASTM Designation C 805-94," *Standard Test Method for Rebound Number of Hardened Concrete* ","*1995 Annual Book of ASTM Standard*" Volume 04.02 "*Concrete and Concrete Aggregates*" American Society for Testing and Materials, Philadelphia, P.A

4.2. Pullout Test

4.2.1. Test Equipment and Procedure

A Pullout test, by using a dynamometer and a reaction bearing ring, measures the force required to pullout from concrete a specially shaped insert whose enlarged end has been cast into the concrete.
A force is applied to the insert by a loading ram that is seated on a bearing ring and is concentric with the insert shaft. The bearing ring transmits the reaction force to the concrete. As the insert is pulled out, a conical-shaped fragment of concrete is extracted from the concrete.
In the pullout test, a 25 mm (1 in) diameter steel disc on a conical shaped stem is embedded at 25 mm (1 in) below the surface of the concrete during casting. A pull bolt is screwed into the stem of the disk and pulled by hydraulic force against a surface mounted reaction ring. The disk is loaded to failure by means of a hand operated portable hydraulic jack and the total force is measured on a gauge attached to the jack.
The pullout test can be used during construction to estimate the in-place strength of concrete to help determine whether construction activities such as form removal, application of post-tensioning, early opening to traffic, or termination of cold weather protection can proceed. Because compressive strength is usually required to evaluate structural safety, the ultimate pullout force measured during the in-place test is converted to an equivalent compressive strength by means of a previously established correlation relationship.

4.2.2. Applications

The pullout test has been adopted as a standard test method in many parts of the world, including North America, and has been used successfully on numerous large construction projects. Primary use of the system has been in either controlling formwork removal or the time of post – tensioning, or determining the minimum amount of curing needed in cold weather concerting.

4.2.3. Advantages and Disadvantages

Advantages are:

- It provides a direct measure of the in situ strength of concrete.
- The method is relatively simple and testing can be done in the field in a matter of minutes.

Disadvantages are:

- Minor damage to the concrete surface must be repaired,
- The standard pullout tests have to be planned in advance, and unlike other in situ tests, cannot be performed at random after the concrete has hardened.

Reference

(1) G.I.Crawford, September 1997,U.S. Department of Transportation, Federal Highway Administration, ”*Guide to Nondestructive Testing of Concrete”*.

4.3. Break-Off Test

4.3.1. Introduction

Out of the many currently available NDT methods, only the Break-Off test and the Pullout tests measure a direct strength parameter (1). The Break-Off test consists of breaking off an in-place cylindrical concrete specimen at a failure plane parallel to the finished surface of the concrete. The cylindrical specimen is formed either by inserting a plastic sleeve into fresh concrete or by drilling a core after the concrete has hardened. The Break-Off stress at failure can then be related to the compressive strength or flexural strength of concrete using a predetermined relationship which relates the concrete strength to the Break-Off strength for a particular concrete mix. The Break-Off test is not very widely used in North America. The primary factor in limiting the widespread use of this method is the lack of necessary technical data and experience in North America. Initial work at the Canada center for Minerals and Energy Technology (CANMET) in the early 1980s indicated a lack of reproducibility in results of this test method (1).

4.3.2. Test Equipment and Procedure

The Break-Off tester (fig.4.4), consists of a load cell, a manometer, and a manual hydraulic pump capable of breaking a cylindrical concrete specimen 55 mm (2.17 in) diameter and 70 mm (2.76 in) long ,as shown in figure 4.5 . The load cell has two measuring ranges: low range setting for low-strength concrete up to approximately 20 Mpa (3000 Psi) and high range setting for higher strength concrete up to about 62 Mpa (9000 Psi). The manufacturer also provides a calibrator for calibration and adjustment of the Break-Off tester.

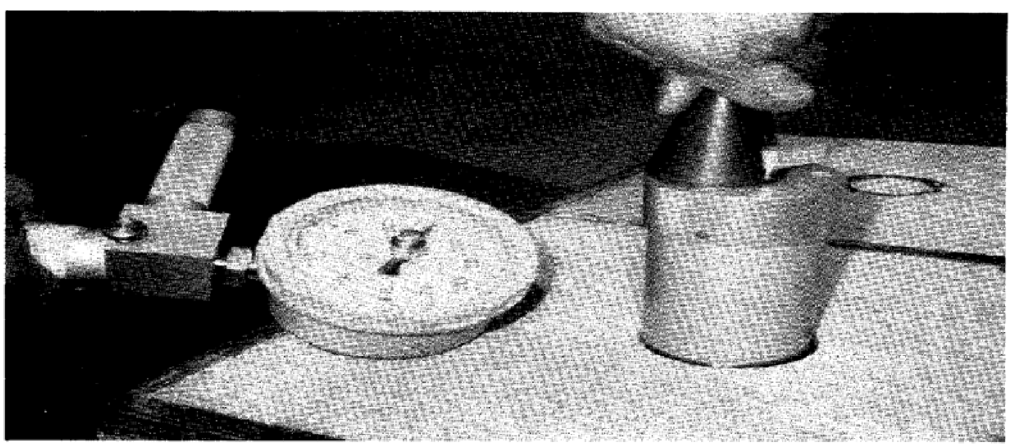


Figure 4.4- Break-Off test equipment

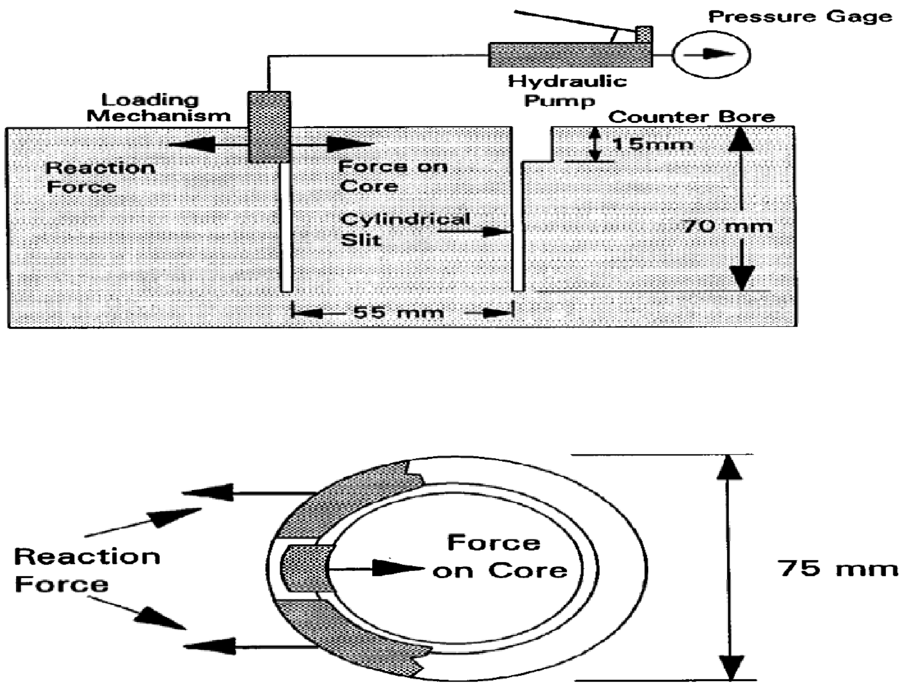


Figure 4.5- schematic of cylindrical slit and application of load for Break-Off test

4.3.3. Applications

The Break-Off method can be used both as quality control and quality assurance tools. The most practical use of the Break-Off test equipment is for determining the time for safe form removal and the release time for transferring the force in prestressed or post-tensioned members.

4.3.4. Advantages and disadvantages

The advantages are:

- Ability to measure in-place compressive strength
- Safe, simple to use
- Test is quickly performed , requires only one exposed surface
- Reproducible to an acceptable degree of accuracy and correlates well with the compressive strength of concrete.

The disadvantages are:

- The damage to the concrete member that requires patching

4.4. Tensile Bond Strength (Pull-off) Test

4.4.1. Introduction

The rehabilitation of concrete commonly requires the removal of deteriorated concrete and repair with a patch material and/or an overlay. To ensure long service of the rehabilitated concrete, it is imperative that the repair materials are well bonded to the underlying concrete. Proper surface preparation of the substrate is an important factor for the success of any repair. The tensile bond strength (pull-off) test is quick, simple and accurate method for determining how well the repair material is bonded to the underlying concrete.

4.4.2. Test Equipment and Procedure

Test equipment required evaluating the tensile bond (pull-off) strength of a patch or an overlay to underlying concrete in a repair area consists of: (1) a dynamometer to measure the tensile load applied to a metal pipe cap or disc bonded with epoxy to the repaired surface, (2) 50 mm (2 in) diameter metal disc with threaded pull bolts, and (3) an electric core drill fitted with a carbide-tipped or diamond core drill capable of producing a cored disc 50 mm (2 in) in diameter. Figure 4.6 shows a commercially available tensile bond tester.

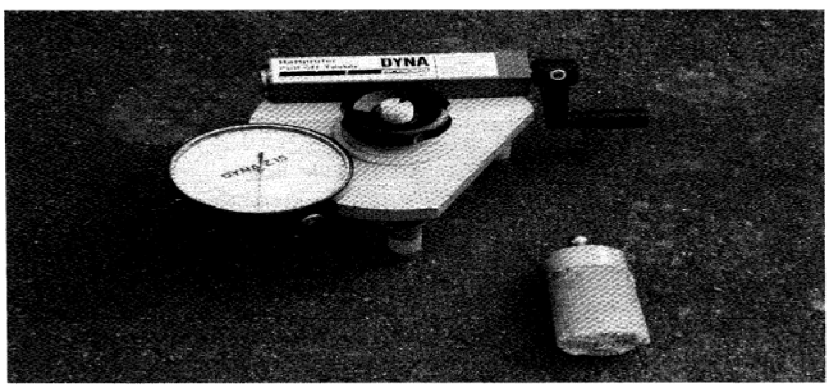


Figure 4.6- Commercially available tensile bond strength tester

4.4.3. Compressive Strength of Concrete

The pull-off test, when used to predict the in-place compressive strength of concrete, involves bonding a metal disc to the surface of the concrete with a rapid-set epoxy adhesive. Before performing the test, the surface of the concrete to be tested should be abraded to remove any laitance and ensure a good bond between the metal disc and the concrete surface. Drill a partial depth core into the concrete. The core bit should produce a cored disc 2 inches in diameter. Sometimes adequate results have been obtained by bonding the metal directly to the cleaned surface without coring first. A metal disc with a threaded pull bolt is then bonded with a rapid-set epoxy to the top of the unbroken core.

After the epoxy has cured, approximately one hour at 22 ˚C (72 ˚F), place an appropriate loading device similar to the ACI503 R device, or a commercially available device. Use the loading device to apply a tensile force sufficient to pull the core out in tension. The total load applied divided by the cross-sectional area of the core is a direct measurement of the tensile bond strength. The load should be applied at the approximate rate of 0.4 KN (100 lb) every 5 seconds. Calibration graphs, based on pull-off tests and cube/cylinder compressive tests, provide a reliable estimate of equivalent cube/cylinder strengths.

4.4.4. Tensile Strength of Concrete

One of the biggest disadvantages in concrete is its brittle nature and its inability to resist cracking due to direct tensile forces. Direct tensile strength of concrete ranges from 7 to 11 percent of its compressive strength. However, laboratory tests for direct tension are seldom carried out because of difficulties in mounting the specimens and secondary stresses induced by the holding device. The direct tensile/compressive strength ratio is 10 to 11 percent for low strength, 8 to 9 percent for medium strength, and 7 percent for high strength concrete.

4.4.5. Applications

- The test is important because it is performed in-situ and can be reliably used as a quality control tool,
- useful for assessing the best procedure to be used for surface preparation for patches or concrete overlays,
- Determining whether a bonding agent is required and the effect of the bonding agent on the bond strength,
- To estimate the expected service life of overlays by measuring the degradation of bond strength with time.

Reference

(1) Naik, T.R. Chapter 4"*The Break-Off Test Method*", "*Handbook on Nondestructive Testing of Concrete* "CRC Press, Inc., Boca Raton, Fl, 1991, pp .83-100.

4.5. Maturity Test

4.5.1. Introduction

The maturity concept is a useful technique for estimating the strength gain of concrete at early ages, generally less than 14 days old. The method accounts for the combined effects of temperature and time on concrete strength development. An increase in the curing temperature can speed up the hydration process which will increase the strength development. Maturity is a function of the product of curing time and internal concrete temperature. It is then assumed that a given mix at equal maturities will have the same strength, independent of the curing time and temperature histories (1).

4.5.2. Test Equipment and Procedure

It is essential that proper curing procedures be used to apply the maturity method for estimating strength development. If this is not the case, then strength estimates based upon the maturity method are meaningless. Application of the maturity method involves two steps: (1) laboratory calibration, (2) actual measurement of time-temperature history of concrete placed in a structure. Because laboratory testing establishes the strength-maturity relationship for a particular mix, it must be performed prior to any field work.
In the field, the time-temperature history of concrete placed in a structure must be collected in order to determine in-place maturity. This in-place maturity is then used in conjunction with the strength-maturity relationship to estimate the in-place strength. Careful consideration should be given in selecting appropriate locations for the temperature sensors.

4.5.3. Maturity Test Equipment

In order to determine concrete maturity, a temperature-time record of the in-place concrete must be kept. The most basic method of measuring concrete maturity would be to measure and record the in-place concrete temperature with a thermometer and measure the elapsed time with a watch. This method is very labor intensive, and is not economical or practical.
Several maturity devices are now available which continuously measure concrete temperature and calculate maturity at least once every hour. The meters can also display the maturity value digitally at any point in time. Some maturity meters can be set up to use either the Nurse-Saul or the Arrhenius equation. The choice of equation depends on the range of ambient temperature to which concrete will be exposed during curing. Depending on the meter being used, four to sixteen different locations can be monitored simultaneously.
Nurse-Saul equation is:

$$M=\sum_0^t (T - T_o)\Delta t \qquad (4\text{-}1)$$

Where,

M=maturity at age T_o
T= average temperature of the concrete during time interval Δt
T_o=datum temperature

The Arrhenius equation states that: “the rate of a chemical reaction is proportional to a rate constant K, whose relationship to absolute temperature T, the gas constant R, and the activation energy E is given in the equation:

$$K = A^{\left(-\frac{E}{RT}\right)} \tag{4-2}$$

Where,
K= rate constant
A=constant
E= activation energy
R=gas constant
T= absolute temperature

The constant “A” depends on whether the reaction is uni- or bi- molecular. The activation energy E depends on the properties of the cement, water/cement ratio, and aggregates in the concrete mixture. The maturity equation becomes:

$$Te = \sum_0^t e^{\left(-\frac{E}{R}\right)\left(\frac{1}{273+T}-\frac{1}{273+Tr}\right)\Delta t} \tag{4-3}$$

Where,
T= average temperature of concrete during the time interval Δt, C
Tr= reference temperature, C
E= activation energy, J/mol
For T ≥ 20˚ C E= 33500 J/mol
For T ≤ 20˚ C E= 33500+1470 (20-T) J/mol
R= universal gas constant, 8.3144 J/ (mol K)

4.5.4. Applications

The maturity method has numerous applications in concrete construction:

- It has been used successfully to estimate in-place strength of concrete to assure critical construction operations. Such as form removal or the application of prestressing or post-tensioning force.
- To determine when traffic can be turned on to new pavement construction or the opportune time to saw joints in concrete pavement,
- Some of the more advanced maturity techniques, such as the Computer Interactive Maturity System (CIMS) can be used for quality control and concrete mix verification.

4.5.5. Advantages and Disadvantages

- Useful, easily implemented, accurate means of estimating *in-situ* concrete strength.
- Quality assurance costs can be reduced because the number of test cylinders is reduced by using the maturity concept.

Reference

(1) Carino, N.J., Chapter 5”The Maturity Method, Handbook on Nondestructive Testing of Concrete”, CRC Press, Inc., Boca Raton ,FL,1991, pp.101-146

5. Hidden Flaws

5.1. Infrared thermography

5.1.1. Fundamental Principles

According to the fundamental Law of Planck all objects above absolute zero emit infrared radiation. This radiation only becomes visible to the human eye when the temperature is above about 500˚C. Infrared monitoring equipment has been developed which can detect infrared emission and visualize it as a visible image. The sensitive range of the detector lies between 2 and 14 microns. The 2-5.6 micron range is generally used to visualize temperature between 40˚C and 2000˚C and the 8-14 micron range is used for temperature between -20˚C and ambient temperatures (1).
The thermograms taken with an infrared camera measure the temperature distribution at the surface of the object at the time of the test. It is important to take into consideration that this temperature distribution is the result of a dynamic process. Taking a thermogram of this object at an earlier or later time may result in a very different temperature distribution. This is especially true when the object has been heated or cooled.
The detectability of any internal structure such as voids, delaminations or layer thicknesses depends on the physical properties (heat capacity, heat conductivity, density, and emissivity) of the materials of the test object. Naturally any interior 'structure' has an effect on the temperature distribution on the surface. If the temperature changes on the surface there is a delay before the effect of this change occurs below where a defect such as a void occurs. The longer the time delays before the temperature changes, the greater the depth of the defect below the surface. Generally anything deeper than 10 cm will only show after a long period of time (>1 hr) after the temperature change has occurred.
Since the infrared system measures surface temperatures only, the temperatures measured are influenced by three factors: (1) subsurface configuration, (2) surface condition; and (3) environment. As an NDT technique for inspecting concrete, the effect of the subsurface configuration is usually most interesting. All the information revealed by the infrared system relies on the principle that heat cannot be stopped from flowing from warmer to cooler areas, it can only be slowed down by the insulating effects of the material through which it is flowing. Various types of construction materials have different insulating abilities or thermal conductivities. In addition, differing types of concrete defects have different thermal conductivity values. For example, an air void has a lower thermal conductivity compared with the surrounding concrete. Hence the surface of a section of concrete containing an air void could be expected to have a slightly different temperature from a section of concrete without an air void.
As we know, there are three ways of transferring thermal energy from a warmer to a cooler region: (1) conduction; (2) convection; and (3) radiation. Sound concrete should have the least resistance to conduction of heat, and the convection effects should be negligible. The surface appearance, as revealed by the infrared system, should show a uniform temperature over the whole surface examined. However, poor quality concrete contains anomalies such as voids and low density areas which decrease the thermal conductivity of the concrete by reducing the energy conduction properties without substantially increasing the convection effects.
In order to have heat energy flow, there must be a heat source. Since concrete testing can involve large areas, the heat source should be both low cost and able to give the concrete surface an even distribution of heat. The sun fulfils both these requirements. Allowing the sun to warm the

surface of the concrete areas under test will normally supply the required energy. During night-time hours, the process may be reversed with the warm ground acting as the heat source. For concrete areas not accessible to sunlight, an alternative is to use the heat storage ability of the earth to draw heat from the concrete under test. The important point is that in order to use infrared thermography, heat must be flowing through the concrete. It does not matter in which direction it flows.

The second important factor to consider when using infrared thermography to measure temperature differentials due to anomalies is the surface condition of the test area. The surface condition has a profound effect upon the ability of the surface to transfer energy by radiation. This ability of a material to radiate energy is measured by the emissivity of the material, which is defined as the ability of the material to radiate energy compared with a perfect blackbody radiator. A blackbody is a hypothetical radiation source, which radiates the maximum energy theoretically possible at a given temperature. The emissivity of a blackbody equals 1.0. The emissivity of a material is strictly a surface property. The emissivity value is higher for rough surfaces and lower for smooth surfaces. For example, rough concrete may have an emissivity of 0.95 while shiny metal may have an emissivity of only 0.05. In practical terms, this means that when using thermographic methods to scan large areas of concrete, the engineer must be aware of differing surface textures caused by such things as broom textured spots, rubber tire tracks, oil spots, or loose sand and dirt on the surface.

The final factor affecting temperature measurement of a concrete surface is the environmental system that surrounds that surface. Some of the factors that affect surface temperature measurements are:

Solar Radiation: testing should be performed during times of the day or night when the solar radiation or lack of solar radiation would produce the most rapid heating or cooling of the concrete surface.

Cloud Cover: clouds will reflect infrared radiation, thereby slowing the heat transfer process to the sky. Therefore, night-time testing should be performed during times of little or no cloud cover in order to allow the most efficient transfer of energy out of the concrete.

Ambient Temperature: This should have a negligible effect on the accuracy of the testing since one important consideration is the rapid heating or cooling of the concrete surface. This parameter will affect the length of time (i.e. the window) during which high contrast temperature measurements can be made. It is also important to consider if water is present. Testing while ground temperatures are less that 0˚C should be avoided since ice can form, thereby filling subsurface voids.

Wind Speed: High gusts of wind have a definite cooling effect and reduce surface temperatures. Measurements should be taken at wind speeds of less than 15 mph (25 km/h).

Surface Moisture: Moisture tends to disperse the surface heat and mask the temperature differences and thus the subsurface anomalies. Tests should not be performed while the concrete surface is covered with standing water or snow.

Once the proper conditions are established for examination, a relatively large area should be selected for calibration purposes. This should encompass both good and bad concrete areas (i.e. areas with voids, delaminations, cracks, or powdery concrete). Each type of anomaly will display a unique temperature pattern depending on the conditions present. If, for example, the examination is performed at night, most anomalies will be between 0.1° and 5°C cooler than the surrounding solid concrete depending on configuration. A daylight survey will show reversed results, i.e. damaged areas will be warmer than the surrounding sound concrete.

5.1.2. Equipment for Infrared Thermographic method

In principle, in order to test concrete for subsurface anomalies, all that is really needed is a sensitive contact thermometer. However, even for a small test area, thousands of readings would have to be made simultaneously in order to outline the anomaly precisely. Since this is not practical, high resolution infrared thermographic cameras are used to inspect large areas of concrete efficiently and quickly. This type of equipment allows large areas to be covered and the resulting data can be displayed as pictures with areas of differing temperatures designated by differing grey tones in a black and white image or by various colors on a color image. A wide variety of auxiliary equipment can be used to facilitate data recording and interpretation.
There are two types of infrared cameras available:

(1) Focal Plane Array (FPA) cameras – where there are a large number of active elements (256x256 or larger). Cooling is done by Stirling engines in a few minutes so the system can be used independent of liquid nitrogen supply. The newer cameras use newer sensor materials such as PtSi. Uncooled infrared cameras based on the bolometer principle are available with sensitive arrays but have not reached the sensitivity of the cooled detectors. For transient experiments frame rates (the number of frames taken per second) up to 60 Hz are standard, higher rates are available from special manufacturers. High quality data can be stored by writing direct digital storage up to 16 bit resolution, avoiding the degradation of data by digital/analogue conversion and storage of the images on video tapes in video format.
(2) Single active element scanner cameras where the image is mechanically scanned by a Single detector.

A complete infrared camera and analysis system can be divided into four main subsystems. The first is the infrared camera, which normally can be used with interchangeable lenses. It is similar in appearance to a portable video camera. The camera's optical system is transparent either to short wave infrared radiation with wavelengths in the range of 3 to 5.6 µm or to medium wave infrared radiation with wavelengths in the range of 8 to 12 µm.
Typically the infrared camera's highly sensitive detector is cooled by liquid nitrogen to a temperature of –196˚C and can detect temperature variations as small as 0.1°C. Alternate methods of cooling the infrared detectors are available which use either compressed gases or electric cooling. These last two cooling methods may not give the same resolution since they cannot bring the detector temperatures as low as liquid nitrogen. In addition, compressed gas cylinders may present safety problems during storage or handling.
The second major component of the infrared scanning system is a real time microprocessor coupled to a display monitor. With this component, cooler items being scanned are normally represented by darker grey tones, while warmer areas are represented by lighter grey tones. In order to make the images easier to understand for those unfamiliar with interpreting grey-tone images they might be transferred into false color images. This transformation assigns different colors (8, 16, or 256) to the temperature range displayed. The color palette used for transformation can be created as one wishes. It is important to remember that the color assigned to a temperature has no physical meaning.
The third major component of the infrared scanning system is the data acquisition and analysis equipment. It is composed of an analogue to digital converter, a computer with a high resolution color monitor, and data storage and analysis software. The computer allows the transfer of

instrumentation videotape or live images of infrared scenes to single frame computer images. The images can then be stored individually and later retrieved for enhancement and individual analysis. The use of the computer allows the engineer in-charge of testing to set specific analysis standards based upon destructive sample tests, such as cores, and apply them uniformly to the entire area of concrete. Standard, off-the-shelf image analysis programs may be used, or custom written software may be developed.
The fourth major component consists of various types of image recording and retrieving devices. These are used to record both visual and thermal images. They may be composed of instrumentation video tape recorders, still frame film cameras with both instant and 35 mm or larger formats, or computer printed images.
All of the above equipment may be carried into the field or parts of it may be left in the laboratory for additional use. If all of the equipment is transported to the field to allow simultaneous data acquisition and analysis, it is prudent to use an automotive van to set up and transport the equipment. This van should include power supplies for the equipment, either batteries or inverter, or a small gasoline driven generator. The van should also include a method to elevate the scanner head and accompanying video camera to allow scanning of the widest area possible depending on the system optics used.
Several manufacturers produce infrared thermographic equipment. Each manufacturer's equipment has its own strengths and weaknesses. These variations are in a constant state of change as each manufacturer alters and improves his equipment. Therefore, equipment comparisons should be made before purchase. Recently the three main manufacturers of thermography equipment — AGEMA, Inframetrics and FLIR — have merged.

5.1.3. General Procedure for Infrared Thermographic Method

In order to perform an infrared thermographic inspection, a temperature gradient and thus a flow of heat must be established in the structure. Assume that it is desired to test an open concrete bridge deck surface. The day preceding the inspection should be dry with plenty of sunshine. The inspection may begin two to three hours after sunrise or sunset, both times being of rapid heat transfer.
The deck should be cleaned of all debris. Traffic control should be established to prevent accidents and to prevent traffic vehicles from stopping or standing on the pavement to be tested. It will be assumed that the infrared scanner be mounted on a mobile van along with other peripheral equipment, such as recorders for data storage and a computer for assistance in data analysis. The scanner head and either a regular film-type camera or a standard video camera should be aligned to view the same sections to be tested.
The next step is to locate a section of concrete deck and establish, by coring, that it is sound concrete. Scan the reference area and set the equipment controls so that an adequate temperature image is viewed and recorded.
Next, locate a section of concrete deck known to be defective by containing a void, delamination, or powdery material. Scan this reference area and again make sure that the equipment settings allow viewing of both the sound and defective reference areas in the same image with the widest contrast possible. These settings will normally produce a sensitivity scale such that full scale represents no more than 5°.
If a black and white monitor is used, better contrast images will normally be produced when the following convention is used: black is defective concrete and white is sound material. If a color

monitor or computer enhanced screen is used, three colors are normally used to designate definite sound areas, definite defective areas, and indeterminate areas.
As has been mentioned, when tests are performed during daylight hours, the defective concrete areas will appear warmer, while during tests performed after dark, defective areas will appear cooler.
Once the controls are set and traffic control is in place, the van may move forward as rapidly as images can be collected, normally 1 to 10 miles (1.6 to 16 km) per hour. If it is desired to mark the pavement, white or metallic paint may be used to outline the defective deck areas. At other times, a videotape may be used to document the defective areas, or a scale drawing may be drawn with reference to bridge deck reference points. Production rates of up to 130 m^2/day have been attained.
During long testing sessions, re-inspection of the reference areas should be performed approximately every 2 h, with more calibration retests scheduled during the early and later periods of the session when the testing “window” may be opening or closing.
For inside areas where the sun cannot be used for its heating effect, it may be possible to use the same techniques except for using the ground as a heat sink. The equipment should be set up in a similar fashion as that described above, except that the infrared scanner's sensitivity will have to be increased. This may be accomplished by setting the full scale so it represents 2°C and/or using computer enhancement techniques to bring out detail and to improve image contrast.
Once data are collected and analyzed, the results should be plotted on scale drawings of the area inspected. Defective areas should be clearly marked so that any trend can be observed.
Computer enhancements can have varying effects on the accuracy and efficiency of the inspection systems. Image contrast enhancements can improve the accuracy of the analysis by bringing out fine details, while automatic plotting and area analysis software can improve the efficiency in preparing the finished report.
When inspecting areas where shadows occur, such as pavements near buildings, it is preferable to perform the inspection after sunset since during daylight hours the shadows move and can result in confusing test results.

5.1.4. Advantages and Limitations of Infrared Thermography

Thermographic testing techniques for determining concrete subsurface voids, delaminations, and other anomalies have advantages over destructive tests like coring and other NDT techniques such as radioactive/nuclear, electrical/magnetic, and acoustic and radar techniques.

Advantages are:

- major concrete areas need not be destroyed during testing,
- Only small calibration corings are used,
- major savings in time, labor, equipment, traffic control, and scheduling problems,
- when aesthetics is important, no disfiguring occurs on the concrete to be tested,
- Rapid set up and take down ,when vandalism is possible,
- no concrete dust and debris are generated that could cause environmental problems,
- infrared thermographic equipment is safe as it emits no radiation,

- It only records thermal radiation, which is naturally emitted from the concrete, as well as from all other objects. It is similar in function to an ordinary thermometer, only much more efficient.
- it is an area testing technique, while the other NDT methods are mostly either point or line testing methods,
- Infrared thermography is capable of forming a two dimensional image of the test surface showing the extent of subsurface anomalies.
- portable and permanent records can be made,
- Testing can be done without direct access to surface and large areas can be rapidly inspected using infrared cameras,

Disadvantages are:

- The depth or thickness of a void cannot be determined, although its outer dimensions are evident. It cannot be determined if a subsurface void is near the surface or farther down at the level of the reinforcing bars.
- Equipments are expensive and require highly skillful and experienced operator.
- It is very sensitive to thermal interference from other heat sources. Moisture on the surfaces can also mask temperature differences.

5.2. Visual inspection

5.2.1. Introduction

Visual inspection refers to an NDT method which uses eyes, either aided or non-aided to detect, locate and assess discontinuities or defects that appear on the surface of material under test (Fig. 5.1). It is considered as the oldest and cheapest NDT method. It is also considered as one of the most important NDT method and applicable at all stages of construction or manufacturing sequence. In inspection of any engineering component, if visual inspection alone is found to be sufficient to reveal the required information necessary for decision making, then other NDT methods may no longer considered necessary.

Figure 5.1 - Visual inspection of an object

Visual inspection is normally performed by using naked eyes. Its effectiveness may be improved with the aid of special tools. Tools include fiberscopes, borescopes, magnifying glasses and mirrors. In both cases, inspections are limited only to areas that can be directly seen by the eyes. However, with the availability of more sophisticated equipment known as borescope, visual inspection can be extended to cover remote areas that under normal circumstances cannot be reached by naked eyes.

Although considered as the simplest method of NDT, such an inspection must be carried out by personnel with an adequate vision. Knowledge and experience related to components are also necessary to allow him to make correct assessment regarding the status of the components (2).

5.2.2. Advantages and Disadvantages

Advantages are:

- Cheapest NDT method,
- Applicable at all stages of construction or manufacturing,
- Do not require extensive training,
- Capable of giving instantaneous results,

Limitations:

- Limited to only surface inspection,
- Require good lighting,
- Require good eyesight.

5.3. Half-Cell Electrical Potential Method

5.3.1. Fundamental Principle

The method of half-cell potential measurements normally involves measuring the potential of an embedded reinforcing bar relative to a reference half-cell placed on the concrete surface. The half-cell is usually a copper/copper sulphate or silver/silver chloride cell but other combinations are used. The concrete functions as an electrolyte and the risk of corrosion of the reinforcement in the immediate region of the test location may be related empirically to the measured potential difference. In some circumstances, useful measurements can be obtained between two half-cells on the concrete surface. ASTM C876 - 91 gives a Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.

5.3.2. Equipment for Half-Cell Electrical Potential Method

The testing apparatus consists of the following (Fig. 5.2):

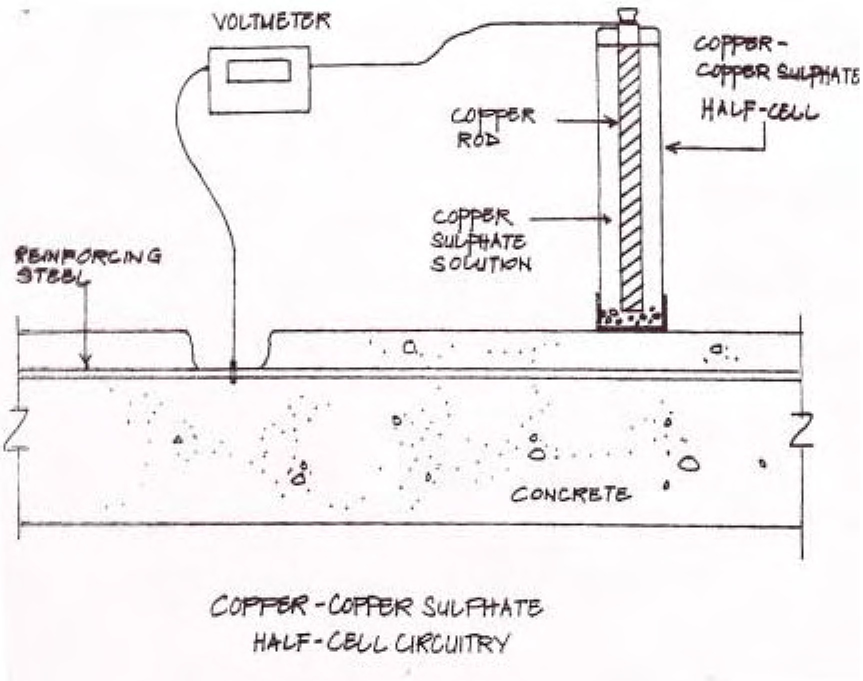


Figure 5.2- A copper-copper sulphate half-cell

Half-cell: The cell consists of a rigid tube or container composed of dielectric material that is non-reactive with copper or copper sulphate, a porous wooden or plastic plug that remains wet by capillary action, and a copper rod that is immersed within the tube in a saturated solution of copper sulphate. The solution is prepared using reagent grade copper sulphate dissolved to saturation in distilled or deionized water.
The rigid tube should have an inside diameter of not less than 25 mm; the diameter of the porous tube should not be less than 13 mm; the diameter of the immersed copper rod should not be less than 6 mm and its length should be at least 50 mm.
Present criteria based on the half-cell reaction of Cu → Cu^{++} + 2e indicate that the potential of the saturated copper-copper sulphate half-cell as referenced to the hydrogen electrode is -0.316 V at 72˚F (22.2˚C). The cell has a temperature coefficient of about 0.0005V more negative per ˚F for the temperature range from 32 to 120˚F (0 to 49˚C).

Electrical junction device: An electrical junction device is used to provide a low electrical resistance liquid bridge between the surface of the concrete and the half-cell. It consists of a sponge or several sponges pre-wetted with a low electrical resistance contact solution. The sponge can be folded around and attached to the tip of the half-cell so that it provides electrical continuity between the porous plug and the concrete member.

Electrical contact solution: In order to standardize the potential drop through the concrete portion of the circuit, an electrical contact solution is used to wet the electrical junction device. One solution, which is used, is a mixture of 95 mL of wetting agent or a liquid household detergent thoroughly mixed with 19 L of potable water. At temperatures less than 10˚C approximately 15% by volume of either isopropyl or denatured alcohol must be added to prevent clouding of the electrical contact solution, since clouding may inhibit penetration of water into the concrete to be tested.

Voltmeter: The voltmeter should be battery operated and have ± 3% end of scale accuracy at the voltage ranges in use. The input impedance should be not less than 10 MW when operated at a full scale of 100 mV. The divisions on the scale used should be such that a potential of 0.02 V or less can be read without interpolation.

Electrical lead wires*:* The electrical lead wire should be such that its electrical resistance for the length used does not disturb the electrical circuit by more than 0.0001 V.
This has been accomplished by using no more than a total of 150 m of at least AWG No. 24 wire. The wire should be suitably coated with direct burial type of insulation.

5.3.3. General Procedure for Half-Cell Electrical Potential Method

Measurements are made in either a grid or random pattern. The spacing between measurements is generally chosen such that adjacent readings are less than 150 mV with the minimum spacing so that there is at least 100 mV between readings. An area with greater than 150 mV indicates an area of high corrosion activity. A direct electrical connection is made to the reinforcing steel with a compression clamp or by brazing or welding a protruding rod. To get a low electrical resistance connection, the rod should be scraped or brushed before connecting it to the reinforcing bar. It may be necessary to drill into the concrete to expose a reinforcing bar. The bar is connected to the positive terminal of the voltmeter. One end of the lead wire is connected to the half-cell and the other end to the negative terminal of the voltmeter. Under some circumstances the concrete surface has to be pre-wetted with a wetting agent. This is necessary if the half-cell reading fluctuates with time when it is placed in contact with the concrete. If fluctuation occurs either the whole concrete surface is made wet with the wetting agent or only the spots where the half-cell is to be placed. The electrical half-cell potentials are recorded to the nearest 0.01 V correcting for temperature if the temperature is outside the range 22.2 ± 5.5°C.
Measurements can be presented either with an equipotential contour map which provides a graphical delineation of areas in the member where corrosion activity may be occurring or with a cumulative frequency diagram which provides an indication of the magnitude of affected area of the concrete member.

Equipotential contour map*:* On a suitably scaled plan view of the member the locations of the half-cell potential values are plotted and contours of equal potential drawn through the points of equal or interpolated equal values. The maximum contour interval should be 0.10 V. An example is shown in Fig.5.3.

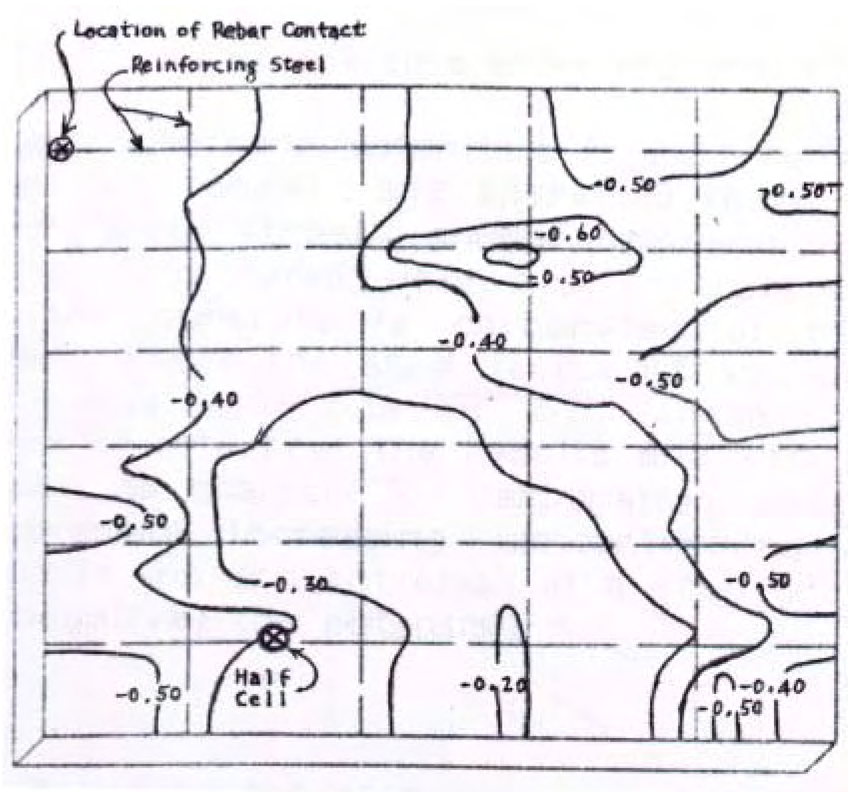


Figure 5.3- Equipotential contour map

Cumulative frequency distribution: The distribution of the measured half-cell potentials for the concrete member are plotted on normal probability paper by arranging and consecutively numbering all the half-cell potentials in a ranking from least negative potential to greatest negative potential. The plotting position of each numbered half-cell potential is determined by using the following equation.

$$fx = \frac{r}{\Sigma n + 1} x100 \qquad (5\text{-}1)$$

where,

fx plotting position of total observations for the observed value, %

r rank of individual half-cell potential,

Σn total number of observations.

The ordinate of the probability paper should be labeled "Half-cell potential (millivolts, CSE)" where CSE is the designation for copper-copper sulphate electrode. The abscissa is labeled "Cumulative frequency (%)". Two horizontal parallel lines are then drawn intersecting the –200mv and –350mv values on the ordinate across the chart, respectively. After the half-cell potentials are plotted, a line is drawn through the values. The potential risks of corrosion based on potential difference readings are shown in Table 5.1.

Table 5.1- Risk of Corrosion against Potential Difference Readings

Potential difference levels (mv)	Chance of re-bar being corroded
less than –500	visible evidence of corrosion
-350 to -500	95%
-200 to -350	50%
More than -200	5%

However, half-cell electrode (Figure 5.4) potentials in part reflect the chemistry of the electrode environment and therefore there are factors which can complicate these simple assumptions. For example, interpretation is complicated when concrete is saturated with water, where the concrete is carbonated at the depth of the reinforcing steel, where the steel is coated and under many other conditions. In those situations an experienced corrosion engineer may be required to interpret the results and additional testing may be required such as analysis for carbonation, metallic coatings and halides. For example, increasing concentrations of chloride can reduce the ferrous ion concentration at a steel anode thus lowering (making more negative) the potential.

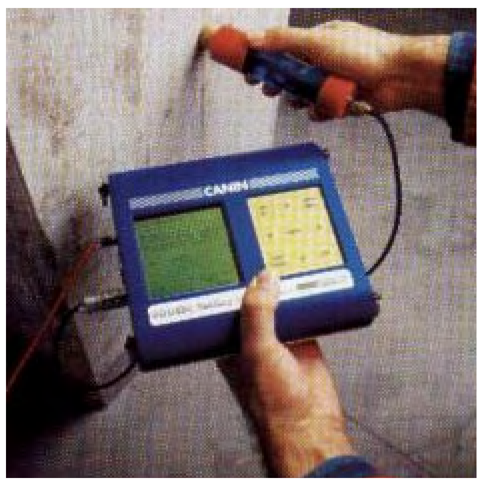


Figure 5.4- Half-Cell Potential instrument for corrosion measurements by Proceq (Switzerland)

5.3.4. Range and Limitations of Half-Cell Electrical Potential

The method has the advantage of being simple with equipment also simple. This allows an almost non-destructive survey to be made to produce isopotential contour maps of the surface of the concrete member. Zones of varying degrees of corrosion risk may be identified from these maps.

The limitation of the method is that the method cannot indicate the actual corrosion rate.

It may require drilling a small hole to enable electrical contact with the reinforcement in the member under examination, and surface preparation may also be required. It is important to recognize that the use and interpretation of the results obtained from the test require an

experienced operator who will be aware of other limitations such as the effect of protective or decorative coatings applied to the concrete.
The Half-Cell potential instruments are used to determine corrosion in reinforcement bars based on the anomalies in the electrical field generated by the instrument on the surface of the concrete structure. The main drawback of the electrical methods is the assumption that the resistivity of each layer is constant and varies slightly with depth, which is far from reality (2).

References

(1) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2002 *"Guidebook on non-destructive testing of concrete structures"* TRAINING COURSE SERIES No. 17
(2) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2005," *Non-destructive testing for plant life assessment"* TRAINING COURSE SERIES No. 26

5.4. Betatron PXB - 7.5 MeV

5.4.1. General Description

The Portable X-ray Betatron (PXB) produces X-ray beams with an energy level of 7.5 MeV. With such high energy, the X-rays can penetrate thick concrete and steel, and reveal flaws inside the concrete structure by high quality X-ray images. The radiation levels outside the main beam are low. It is suitable for both in-lab and in-situ operations.

5.4.2. Applications

The Betatron (Figure 5.5) is typical being used for:

- Mapping of the reinforcement (size, depth, position, configuration and condition)
- Studying the homogeneity of the concrete (voids)

Figure 5.5- Testing with the Betatron

5.4.3. Performance and advantages (1)

- It is possible to fulfill the Nuclear Energy Agency requirements: x-ray detect ability of 20 mm porosity in 1000 mm thick concrete.
- It is possible to detect from 5% - 20% loss of thickness in cables and reinforcement depending on the direction of exposure.
- The depth placement of reinforcing bars can be determined by means of image processing if the nominal diameter of the bar is known.
- It is possible to determine the approximate depth of a void by calculating a void density factor.

Reference

(1) High Energy X-ray Radiography for Examining Reinforced Concrete, Force technology

6. Modulus of Pavement Layers

6.1. Introduction

The premature failure of highway pavements from substandard construction practices and materials is a major expense in terms of money, labor, and natural resources, and improved techniques are needed to mitigate this problem. Knowing the structural characterization of subgrade and base materials used in pavement systems is essential for developing better design and construction procedures. Standard guides for the design of pavement structures incorporate the correlation between resilient modulus and more traditional soil parameters such as stiffness, density, moisture content and material type. Making accurate assessments of the structural condition of roads during construction helps tremendously in locating weak areas prone to localized failure and correcting them prior to completion of the pavement. Knowledge of these failure-prone zones greatly facilitates maintenance and rehabilitation operations.

After construction, it is generally assumed that pavements perform up to design standards. However, non-uniformity or variability in the structural characteristics of various pavement components and poor construction monitoring may lead to the formation of localized areas of premature distress in the form of rutting, cracking or other types of distress. Under repeated traffic loading and severe environmental conditions, these areas tend to deteriorate rapidly, leading to poor service conditions and necessitating early maintenance and rehabilitation. Recent studies have shown that the most effective method for controlling the premature failure of pavement is through proper inspection and in-situ testing of construction materials during construction (1).

Nondestructive testing (NDT) of the subgrade and base layers along the length of a project during and directly after construction aids in identifying localized problem areas where the stiffness of these materials deviates from the desired values. Dynamic response and pavement parameters, such as layer thickness, stiffness, modulus, moisture content, and density can be determined from NDT data. After calculating the variability in the characteristics of the subgrade and base material, potential problem areas can be identified and remedial measures taken during the construction process.

6.1.1. Humboldt Stiffness Gauge

The Humboldt Stiffness Gauge (HSG) provides a simple, quick and accurate means of directly measuring stiffness of the upper lift of material. The stiffness of the subgrade and base is directly influenced by the degree of compaction, the moisture content of fine-grained material in these layers and the type of soil in the subgrade.

The HSG measures impedance at the soil surface by generating vibrations at 100 and 200 Hz that impart a very small change in the applied load (2). The stiffness of the pavement material in resisting this load is determined at each frequency and the average is displayed on the Stiffness Gauge display window. The entire process takes about one minute. It has been found that, at low frequencies, the impedance at the surface is stiffness controlled. If a Poisson's ratio is assumed and knowing the HSG's physical dimensions, shear and elastic modulus can be derived for the base and subgrade. The HGS weighs about 10 kg, is 28 cm in diameter, 25.4 cm tall and rests on the soil surface via a ring-shaped foot, as shown in Figure 6.1.

Figure 6.1- Humbolt Stiffness Gauge

Small deflections generated by the HSG are given the symbol δ, which is proportional to the outside radius of the ring foot (R), the elastic modulus (E), the shear modulus (G) and the Poisson's ratio (ν) of the soil. The stiffness of the layer being tested is the ratio of the force to the displacement: K= P/δ. The HSG generates soil stress levels commonly experienced by the base and subgrade (1 92 Pa or 4 psi).
The stiffness value obtained at each location, which was directly displayed in the Humboldt Stiffness Gauge display window, was recorded in MN/m.
Stiffness values were computed with the Humboldt Stiffness Device using the following equation:

$$K = P / \delta \approx 1.77\ RE / \left(1 - v^2\right) \tag{6-1}$$

Where,
K = stiffness (lb/ in)
P = Load (lb)
δ = Deflection

After calculating the stiffness (K), knowing the radius (R), and assuming Poisson's ratio (ν) = 0.4, the modulus (E) can be calculated with Equation 6-2. Since the influence zone for the Stiffness Gauge is limited to a 6-inch depth, the modulus of compacted subgrade and base materials must be calculated from data obtained on the surface of those layers.

$$E = \frac{K\left(1 - v^2\right)}{1.77R} \tag{6-2}$$

Where,
E = Modulus
K = stiffness
R = radius of the HSG ring = 2.25 inches
ν = Poisson's ratio = 0.4
P = load in pounds

The HSG was placed firmly on the soil surface, which itself required little or no preparation. A 60% minimum contact area between the HSG foot and soil was required. On particularly hard or rough surfaces, less than 1/4 inch of moist sand or local fines was used to ensure adequate contact between the HSG and the surface, and to provide a uniform surface for the HGS. Once firm contact had been established, readings were taken by pressing the "Measure" button. Each stiffness reading took about one minute.

6.1.2. German Plate Load Test

German Plate Load testing is a procedure in which the sequential loading and unloading of soil is done by means of a load plate through a pressure application device (3). Settlement of the plate is measured as the load is applied and released.

The Plate Load equipment consists of a load plate, a pressure application device with an oil pump, a single action hydraulic press, and a high-pressure hose. The load plates are made of steel of at least grade ST 52.0, and the bottom of the plate must be flat. A load application offset device (counter weight) producing a 10 KN load or greater is necessary to provide the required reaction: heavy trucks are most often used for this purpose.

The settlement measurement device used with the German Plate Load test consisted of a dial gauge conforming to DIN 878. With a scale gradation value of 0.01 mm and a minimum measurement range of 10 mm.

Settlement measured on the subgrade and base at each loading and unloading sequence was utilized for calculation purposes. Stiffness (lb/in), modulus (psi), and unit load layer deflection were calculated from the raw data using Equations 6-3 to 6-8 for both the first sequence of loading and also for the second sequence of loading.

The subgrade and the composite stiffness of the entire base layer, which includes the depth of the subgrade required to support the applied load test, were calculated using the equation

$$K = \frac{P}{\delta} = \frac{\pi R E_3}{2(1-\nu^2)} \qquad (6\text{-}3)$$

The subgrade modulus for the Plate Load Test device was computed using:

$$E = \frac{2K(1-V^2)}{\pi R} \qquad (6\text{-}4)$$

Equations 6-5 to 6-7, which were used to evaluate the modulus of a two-layer system of base and subgrade directly under the center of the loading plate, were obtained from the concept of Odemark and Boussinesq(4), which is also known generally as the "method of equivalent thickness". This method consists of transforming a system of $\boldsymbol{n}$ layers of different layer moduli into a single layer of equivalent stiffness where all layers have the same modulus. When calculating the base modulus for the German Plate Load Test and the Falling Weight Deflectometer, the influence of the applied load, which extends to a great depth into the subgrade layer, makes it necessary to adopt the method of equivalent thickness to calculate the modulus of the base layer.

For a two-layer system of base and subgrade, the deflection Do, 2, located directly under the center of the load plate on the top of the base, was approximated using equation 6-5.

$$\text{Do}, 2 = 2(1 - \nu^2)\frac{qa}{E_2E_3}[E_3 + Fb(E_2 - E_3)] \quad (6\text{-}5)$$

Where,

Do = deflection (inches)
q = pressure (psi)
a = radius (inches)
E_2 = Base modulus (psi)
E_3 = Subgrade modulus (psi)
Fb = Boussinesq Deflection Factor, calculated using Equation 6-6.

$$\text{Fb} = [\sqrt{(1 + [\frac{he}{a}]^2)} - (\frac{he}{a})][1+\{(\frac{he}{a}) \div (2(1-\nu^2)\sqrt{1 + (\frac{he}{a})^2})\}] \quad (6\text{-}6)$$

Where,
he = equivalent thickness of subgrade to replace base in inches in order to maintain the stiffness equivalent to that of the base, determined using Equation 6-7.

$$he = h_2(\frac{E_2}{E_3})^{1/3} \quad (6\text{-}7)$$

Where,

h_2= thickness of base (inches)
E_2 = base modulus
E_3 = subgrade modulus

The unit load layer deflection on the subgrade and base material along the centerline at each station was calculated using Equation 6-8:

$$\text{Unit deflection} = \frac{L}{\frac{P}{A}} \quad (6\text{-}8)$$

L is the deflection measured at the site
P is the load applied in pounds (Ib.)

A is the area of the circular steel plate in inches squared (in^2)

6.1.2.1. Test procedure

Before beginning the Load Plate test, the area of ground selected for testing was made as flat and level as possible and loose particles were removed. The plate had to properly rest on the surface with no cavities below the plate. The area of contact between the plate and the soil surface had to be more than 60%.

Each level of load was sustained for an equal time increment. A change in load between loading stages was completed in less than one minute. In the load relief stage, the load was removed from the plate in three stages of 50%, 25% and 0% of the maximum applied load. A second loading cycle was applied only after the complete load removal from the earlier loading sequence. This comprised one load application cycle. Settlement measurements for each load increment and load relief cycle were taken using the dial gauge.

Drawbacks of the German Plate Test include the lengthy time required to complete each test. The deflections being measured, which include material creep, are static and do not accurately represent the response of the pavement structure to moving vehicles.

6.1.3. Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is a nondestructive testing device widely used for pavement testing, research and construction monitoring (Figure 6.2). Many test programs have been established to monitor subgrade construction and pavement performance by using the Falling Weight Deflectometer as the primary tool for assessing changes in layer properties and stiffness.

The Falling Weight Deflectometer (FWD) delivers a transient force impulse to the pavement layers by raising a weight to the desired height on a guide system and dropping it onto the 300-mm diameter circular footplate. By varying the mass of the weight or the drop height or both, the impulse load on the layer surface can be varied between 30 kN. and 110 kN for standard FWDs (such as that used by ODOT), and between 30 kN. and 250 kN for heavy-duty FWDs. Four to nine sensors measure the deflection of the layer surface induced by the applied impulse load. The first sensor is mounted at the center of the footplate, while the remaining sensors are positioned at various radial distances up to 2.5 meters from the load center. All recorded peak deflections are displayed on the FWD monitor and stored for subsequent downloading.

Deflections measured at the center of the load plate were used to calculate modulus and stiffness. Deflection variation between test points within a section may be quite large; ranging from 15 percent to more than 60 percent. This variation reflects changes in layer thickness, material properties, moisture and temperature conditions, sub-grade support, and contact pressure under the load plate (20).

(a) Trailer-mounted FWD

(b) Truck-mounted FWD

(c) Portable light weight FWD

Figure 6.2- Falling Weight Deflectometers

Portable light weight FWD has been developed and used in Europe has interest in many DOTs. Its applications are:

- Rapid stiffness testing of bases and subgrades but discrete measurement of bearing capacity of granular layers.
- Alternative to Nuclear density gauges

6.1.3.1 Advantages and Disadvantages:

Advantages are:

- Deflections can be converted to stiffness
- Low cost
- Portable

The limitations are:

- Depth of influence unknown
- Better software required for multilayer analysis

6.1.3.2. Test Procedure

The FWD device was positioned at the test point. The footplate and seven sensors spaced 0, 8, 12, 18, 24, 36, and 60 inches away from the center of the loaded area were then lowered onto the layer being tested, as shown in Figure 6.3. Pressure was applied by dropping the desired weight from a selected height. After the data had been recorded, the device was moved to the next site. A typical test cycle requires about one minute to complete.

Figure 6.3 -FWD Sensors

6.1.3.3. Backcalculation of Pavement Layer Moduli

A simplified method for calculating pavement layer moduli and thicknesses directly from FWD deflection basin was developed by Noureldin (5). In this method (BACKCAL), layer moduli are estimated using FWD sensors that deflect exactly the same as the interfaces between pavement

layers. The central sensor is at the first interface. Sensors used for moduli calculation are also used for calculating estimated layer thicknesses (5). Pavement layer moduli and thicknesses determined by this method were validated in a number of other research and field studies (6-8). All computations using this method are made with a spreadsheet that allows analysis of data for every FWD testing point. Because this method does not require thickness information and its simplicity, it provides a useful tool in analyzing FWD deflection data at the network level and for those situations in which thickness information is not available. This method was also proven to be successful for project level evaluation and for investigating sensitivity of pavement layers to stress levels, temperature and moisture levels (5).
The main advantage of this technique is that thickness data is not required for the backcalculation process and hence it provides a useful tool in analyzing FWD deflection data particularly at the network level.
BACKAL computations are conducted using the following equations:

$$E_{\text{subgarde}} = \frac{2149}{r_x \ D_x} \times \frac{Actual\ FWD\ Load\ (pounds)}{9000}$$

$r_x D_x$ = largest deflection radii multiplication (i.e. r8D8, r12D12, r18D18, r24D24, r36D36, r48D48 and r60D60). Radii and deflection units are in inches and mils, respectively.

Subgrade modulus obtained using this equation matches exactly with that obtained using the 1993 AASHO Guide algorithm (9), if the same sensor used to calculate that modulus is picked. To estimate the subgrade resilient modulus, MR, values obtained using the above equation is divided by 3 as prescribed in the 1993 AASHTO Guide (9). Pavement support layer (base and subbase) moduli are estimated employing the same equation and using measurements of sensors located between the sensor used for subgrade modulus computation and the sensor underneath the loading center.

Overall Pavement Modulus, E_P, Ksi

$$Ep = \frac{716 - \frac{2149}{r_x}}{D_0 - D_x} \times \frac{Actual\ FWD\ Load\ (pounds)}{9000}$$

Ep = Pavement Modulus (combined for pavement layers on top of the subgrade) in Ksi

r_x and D_x are the same as for subgrade, (i.e. the values associated with maximum $r_x D_x$) and D_0 is the center deflection in mils.

The above equation can also be used to calculate the surface layer modulus only. In this case D_8 (the closest sensor located at 8” from loading center) is designated as D_x. as follows;

Surface Modulus, $E_{Surface}$, Ksi

$$\text{Ep} = \frac{716 - \frac{2149}{8}}{D_0 \quad - D_8} \times \frac{Actual\ FWD\ Load\ (pounds)}{9000}$$

When thickness data is known or the surface layer is thin (lower than 4") the following equation of equivalent thickness is preferred in calculating the surface modulus.

$$\text{E surface} = \left(\frac{\sqrt[3]{EpTp} - \sqrt[3]{E\ support - T\ support}}{T\ surface} \right)^{3}$$

E surface= Surface Modulus in Ksi
E p & E support are pavement & support moduli and Tp, T support & T surface are the layer thicknesses in inches.
Layer moduli backcalculation is conducted for FWD data before any temperature correction. Backcalculated asphalt concrete layer modulus only is then normalized to a standard temperature (usually 68° F).

Temperature Corrected E Surface **=** E Surface **/** Correction Factor

Correction Factor $= (1.0000008)^{314432 - T^3}$
T = mean temperature of asphalt concrete layer, °F, measured at the mid – depth of that layer or calculated using air and surface temperature data collected by the FWD.

Total Thickness, T_x, inches

$$Tx = 0.5\left[\frac{Do - Dx}{Dx\left(\frac{r_x}{3} - 1\right)}\right]^{1/3} \times (4r_x{}^{2} - 36)^{1/2}$$

r_x and D_x are the same as defined above, (i.e. the values associated with maximum r_xD_x) and D_0 is the center deflection in mils.

Surface Thickness, $T_{Surface,}$, inches

$$\text{T surface} = 23.2379 \left[\frac{D0 - D12}{3D12}\right]^{2}$$

D_0 and D_{12} are (the center deflection and the deflection of the sensor located at radii of 12 inches) in mils.

Layer Coefficients and Structural Numbers

AASHTO layer coefficients and structural numbers are calculated employing backcalculated moduli and using the following equations reported by Noureldin (5) and based on the 1993 AASHTO Guide (9);

Surface Layer Coefficient, a_1

$$a_1 = \left(\frac{Temp.Corrected\ Surface\ Modulus\ ,Ksi}{11\times10^3}\right)^{1/3}$$

Support Layer Coefficient, a_2

$$a_2 = \left(\frac{support\ modulus, Ksi}{11\times10^3}\right)^{1/3}$$

Structural Numbers are calculated by multiplying the layer coefficient of a specific layer by its thickness.

Use of Backcalculated Moduli Values in Mechanistic Empirical Pavement Analysis

Backcalculated moduli and thickness values can be employed to calculate stresses and strains at specific locations within the pavement system. Computer software such as ELSYM 5, CHEVRON or BISAR can be used for that purpose (10-17).

Pavement remaining life to failure in ESALs due to fatigue cracking and permanent deformations (rutting) can be calculated employing these stresses, strains and moduli (10-17) as follows;

Remaining Life to Failure in Fatigue Cracking

Log ESALs = a - b log ε_t - c log E_{HMA}

Remaining Life to Failure in Permanent Deformation

Log ESALs= d - e log ε_c

ε_t = maximum tensile strain within hot mix asphalt, HMA, layer (microstrain)

ε_c = compressive strain on the top of subgrade layer (microstrain) or unbound granular layer (base or subbase).

ESALs = number of 18 kips (80 KN) single axle load repetitions to an acceptable degree of cracking or an acceptable rut depth.

E_{HMA} = HMA stiffness modulus, MPa (1MPa=145 psi)

a, b, c, d, e = material coefficients (material coefficients suggested by some procedures are given in Tables 6.1 and 6.2) .

Table 6.1: Material Coefficients for Fatigue Cracking Analysis

Procedure	Reference	a	b	c
ILLI-PAVE	22	12.699	3	0
Finn ,et al	21	15.536	3.291	0.854

Table 6.2: Material Coefficients for Permanent Deformation (Rutting) Analysis

Procedure	Reference	d	e
Nottingham	23	15.5	3.57
Shell	24	17	4
Asphalt Institute	25	18	4.477
Chevron	26	18	4.4843

Remaining life in ESALs can also be estimated employing the 1993 AASHTO design equation, using backcalculated moduli values and setting a specific serviceability range of $\Delta PSI = 1.7$ i.e. (4.2-2.5);

$$\text{Log ESALs} = 9.36\log(SN+1) - 0.2 + \frac{\log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32\log(MR) - 8.07$$

M_R = Subgrade resilient modulus in psi (which can be obtained from dividing backcalculated subgrade modulus by 3).
SN= Total pavement structural number which can also be obtained via backcalculation analysis.

6.1.4. Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) is a quick, simple, automated field test method for evaluating the in-situ stiffness of existing highway pavements (Figure 6.4). The greatest advantage associated with the DCP is its ability to penetrate into underlying layers and accurately locate zones of weakness within the pavement structure. It measures the strength and stiffness of unstabilized base and subgrade layers. The unit has software for storing DCP data.

Figure 6.4-Dynamic cone penetrometer

The DCP drives the penetrometer rod into the ground using constant energy for each blow, and the penetration index determined with the DCP is calculated as a running depth of penetration per blow. After determining the penetration index, Equations 9 and 10 were used to calculate CBR and the subgrade and base modulus (Mr). From these equations two modulus values were obtained. The upper limit value was calculated by adding 0.075 and the lower value was obtained by subtracting 0.075, as shown in equation 6-9.

$$\text{Log (CBR)} = 2.200 - 0.71\ (\log \text{PI})^{1.5} \pm 0.075 \qquad (6\text{-}9)$$

Where PI = DCP Penetration Index (mm/blow)

$$\text{Mr} = 1.2\ \text{CBR} \qquad (6\text{-}10)$$

6.1.4.1. Test Procedure

The Dynamic Cone Penetrometer (DCP), shown in Figure 6.4, generates sufficient energy to drive a rod up to 1.2 m into the pavement structure by striking the head of the rod with an 8-kilogram weight falling a distance of 574.0 mm. The rate of penetration is continuously monitored with depth. Measuring the stiffness of each layer gives a clear profile of the underlying support layers. While the resistance to a driven rod may not be indicative of the actual load-carrying capacity of the layers, weaknesses within the layered structure can be quickly identified. When the DCP rate of penetration exceeds established criteria, a zone of weakness is indicated. Testing the subgrade to a depth of 1.2 m requires about five minutes.

6.1.4.2. Applications

- Quality assurance testing of subgrade and embankment materials
- Alternative to Nuclear density gauges

6.1.4.3. Advantages and limitations

The advantages are (18)

- Cheap/portable/simple
- Related to CBR and stiffness

The limitations are

- Slow, labor intensive
- Point specific
- Problems with granular materials
- Rod friction should be accounted for in clays

Barriers to implementations are

- No specifications (MnDOT)
- Influence of layer moisture content

6.2. Conclusion

It is difficult to directly compare results of the FWD, German Plate Load Test and Humboldt Stiffness Gauge because they are measuring to different depths, they utilize different technologies to induce load and measure in-situ response, and different equations are used to convert surface deformation to layer modulus, particularly on two layered pavement structures. Data obtained in a study indicate strongly that the devices do give similar magnitudes of stiffness and modulus, and similar trends in the data with regard to relative stiffness of the in-situ layers (19).

The types of response being measured with these devices include: dynamic response to heavy loads dropped on the surface with the FWD, static response generated as load is gradually increased during the German Plate Load Test, and dynamic response to small excitations generated by the Humboldt Stiffness Gauge which limits its depth of effectiveness. Dynamic loads typically reflect higher material stiffness than static loads, and the measurement of stiffness to a greater depth in a non-uniform pavement structure will certainly increase variability within the measurements.

The Humboldt Stiffness Gauge is an effective tool for monitoring the integrity of individual material lifts as they are constructed, since the measurements are limited to that lift. Conversely, the FWD and German Plate Load Test are effective in measuring the total composite stiffness of in-situ pavement structures. The FWD has a definite advantage over the German Plate Load Test in being faster, less labor intensive and able to provide much better coverage within a given period of time. If specific areas of the pavement are identified with the FWD as having unusually low stiffness, the Dynamic Cone Penetrometer can be used to identify the cause(s) of low stiffness and locate specific layers within the structure which will likely cause premature distress. Engineers can then assess the cost and benefits of correcting the problem early to extend the service life of the pavement, and avoid higher maintenance costs and public inconvenience later.

References

1. Sargand, Shad M., Wasniak, Daniel L., Masada, Teruhisa and Beegle, David. (January 2000). "*Evaluation of Initial Subgrade Variability on the Ohio SHRP Test Roads"*.
2. Humboldt Mfg. Co. (June 1999). "*Report Estimating Dry Density from Soil Stiffness and Moisture Content*".
3. Verlag Beuth. (June 1990). "Plate Load Test, DIN 181 34 (Construction Industry Standards Committee)".
4. Kenis, William J., and Weijun Wang. (June 1997). "*Analysis of Pavement Structural Variability*," FHWA-RD-97-072.
5. Noureldin, A. S., "*New Scenario for Backcalculation of Layer Moduli of Flexible Pavements*", TRR 1384, Washington, D.C., 1993.
6. Noureldin, A. S., "*Influence of Stress Levels and Seasonal Variations on In-Situ Pavement Layer Properties*", TRR 1448, Washington, D.C., 1994.
7. Noureldin, A. S., Sharaf, E., Arafa, A., and Al-Sugair, "*Rational Selection of Factors of Safety in Reliability Based Design of Flexible Pavements*", TRR 1540, Washington, D.C., 1996.
8. Noureldin, A. S., "*Seasonal Deflections and In Situ Moduli Patterns of Polymer Modified Versus Unmodified Asphalt Pavements*", Proceedings, Mechanical Tests for Bituminous Materials, Lyon, France, March 1997 RILEM, PP 583-590.
9. AASHTO, "*AASHTO Guide for Design of Pavement Structures*", American Association of state Highway and Transportation Officials, 1993.
10. Finn, F., C.L. Saraf, R.Kulkarni, K. Nair, W. smith, and A. Abdullah, NCHRP Report 291 : Development of Pavement Structural Subsystem., TRB, National Research council , Washington , D.C., 1986.
11. Thompson, M.R. and R.P. Elliot. ILLI-PAVE Based Response Algorithm for Design of Conventional Flexible Pavements. In Transportation Research Record 1043, TRB, National Research Council, Washington, D.C., 1985.
12. Huang, Y., H. Pavement Analysis and Design, Prentice Hall, Englewood Cliffs, N.J., 1993.
13. Shell Pavement Design Manual-Asphalt Pavements and Overlays for Road Traffic , Shell International Petroleum Company, Ltd., London, England, 1978 .
14. Research and Development of the Asphalt Institute Manual (MS-1), 9th ed. Report 82-2, Asphalt Institute, 1982.
15. Pierce, L.M., N.C. Jackson, and J.P. Mahoney, Development and Implementation of a Mechanistic, Empirically Based Overlay Design Procedure for Flexible Pavements In Transportation Research Record 1388, TRB, National Research Council, Washington, D.C. , 1992 .

16. Richter, C.A., and J.B. Rauhut, SHRP Plans for Nondestructive Deflection Testing in the Development of Pavement Performance Prediction Models" ASTM, STP 1026 A.J. Bush III and G.Y. Baladi, ASTM, Philadelphia, 1989.
17. Yoder E., and M. Witczack, Principles of Pavement Design, 2nd ed., John Wiley & Sons, Inc., New York, 1975.
18.International Symposium on Non-Destructive Testing for Design Evaluation and Construction Inspection ,"*Promising Existing and Emerging Technologies and Techniques*", A. Wimsatt, S. Hurlebaus, T. Scullion &E. Fernando ,Texas Transportation Institute College Station, TX, 77843
19. Ohio Research Institute for Transportation and the Environment, June 2001, "*Evaluation of Soil Stiffness Via Non-Destructive Testing* "Final Report
20**.** LTPP Manual for Falling Weight Deflectometer Measurements Operational Field Guidelines Version 3.1 August 2000

7. Seismic Pavement Analyzer

7.1. Measurement Procedure

Diagnosis of distress precursors is based on measuring mechanical properties and thicknesses of each of the pavement system layers. The Seismic Pavement Analyzer (SPA) lowers transducers and sources to the pavement and digitally records surface deformations induced by a large pneumatic hammer which generates low-frequency vibrations, and a small pneumatic hammer which generates high-frequency vibrations (Figures 7.1 and 7.2).

This transducer frame is mounted on a trailer that can be towed behind a vehicle and is similar in size and concept to a Falling Weight Deflectometer (FWD). The SPA differs from the FWD in that more and higher frequency transducers are used, and more sophisticated interpretation techniques are applied.

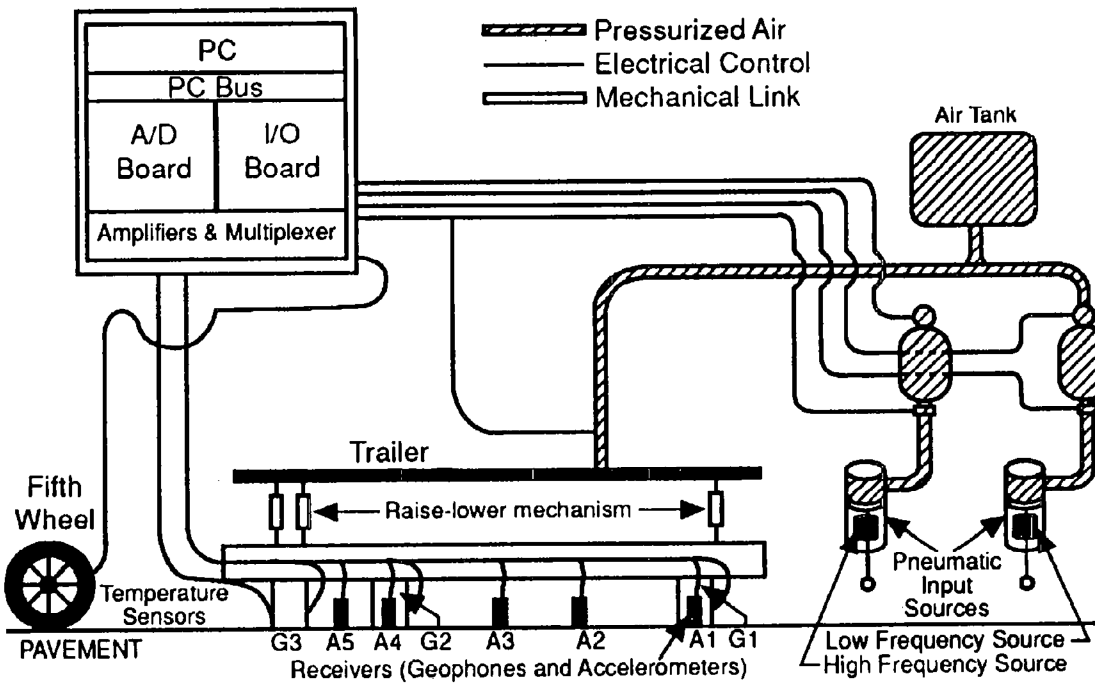


Figure 7.1 - Schematic of Seismic Pavement Analyzer

The SPA is controlled by an operator at a computer connected to the trailer by a cable. The computer may be run from the cab of the truck towing the SPA or from various locations around the SPA.

All measurements are spot measurements; that is, the device has to be towed and situated at a specific point before measurements can be made. A complete testing cycle at one point takes less than one minute. A complete testing cycle includes situating at the site, lowering the sources and receivers, making measurements, and withdrawing the equipment. During this one minute, most of the data reduction is also executed.

Figure 7.2 - Portable Seismic Pavement Analyzer

Nontechnical factors affecting the performance of the SPA are summarized in table 7.1. Safe operation of the device requires traffic control. The level of traffic control necessary is equivalent to that needed to operate an FWD. The skill level of the operator depends on the operation mode of the device. The SPA has two major levels of operation, operation mode and research mode. A conscientious technician with a high school diploma or a degree from a two-year technical college is needed for the operation mode. It is estimated that one or two weeks of training through videotape and the assistance of a maintenance engineer is also necessary. A research engineer with a background in pavements and wave propagation should operate the SPA in research mode.

The appropriate spacing of measurements depends on the intended use. For maintenance, a procedure similar to that of the FWD can be used. However, for high-precision diagnostics, tests should be carried out every 0.3 m to 30 m, depending on the nature of distress. The lower limit of 0.3-m spacing is suitable for precision mapping of delaminated areas or loss of support under portland cement concrete. The upper limit of 30 m is suitable for determining the general variation in the condition of pavement. For rigid pavements, test spacing depends on the joint spacing. Typically the two joints and at least the middle of the slab should be tested. For research purposes, the frequency of measurement should be based on the goals of the research.

An extensive field study (Nazarian et al., 1991) has determined the effects of temperature on the results of different tests. A study concluded that testing rigid pavements at ambient temperatures in excess of 35°C is not feasible (1). For flexible pavements, the temperature should not exceed 50°C. At such high temperatures, the asphalt concrete layer is too viscous and coupling of energy to it is difficult. To minimize the effects of fluctuation in the moisture level due to precipitation, the equipment should not be used until one day after significant precipitation.

The cost of operating the device is estimated at 20 cents per point, plus $10 per hour. This estimate is based upon the cost of operating the FWD as reported by one of the states (1).

Table 7.1 - Nontechnical factors affecting the performance of Seismic Pavement Analyzer

Item	Remark
Measurement Speed	One minute per point
Traffic Control required	Similar to that used for FWD testing
Skill Level of Operator	Operation Mode: A qualified technician Research Mode: A research engineer
Frequency of measurement	Routine Maintenance: Similar to the procedure used with FWD Diagnostics: Every 0.3 to 30 m depending on the project Research: Determined case by case
Necessary Ambient condition	Concrete: Ambient temperature not to exceed 35°C Asphalt : Ambient temperature not to exceed 50°C
Operating Cost per measurement	20 cents per point, plus $10 per hour

7.1.1. Data Analysis

The SPA collects three levels of data.
1) Raw data: the waveforms generated by hammer impacts and collected by the transducers.
2) Processed data: pavement layer properties derived from the raw data through established theoretical models.
3) Interpreted data: the diagnoses of pavement distress precursors from data processed through models.

These models will be improved and upgraded as further field data is available. Processed data will be archived, with the interpretations, so that the user or manufacturer can test and upgrade the interpretation models.

The raw data (waveforms) collected from the hammer impacts are processed immediately and are not saved for archival unless specifically requested. Each of the eight vibration sensors records three impacts. The storage requirements for saving these raw data are large (up to 0.4 megabytes per sample). The SPA can save these data for troubleshooting or research on enhanced processing techniques.

Processed data are the result of calculations performed on the raw data and are independent estimates of the physical properties of the pavement system. These calculated properties are archived for all measurements. Table 7.2 lists the pavement properties estimated from the raw data. Young's modulus is estimated from compression velocity measurements in the AC or PCC and from mechanical impedance in the base. The shear modulus is estimated from surface wave velocity dispersion. Thicknesses are estimated with the impact echo in the paving layer and with surface wave dispersion in the pavement and base. Damping is estimated from the impulse-response method.

Table 7.2 - Pavement properties estimated by the Seismic Pavement Analyzer

Pavement component	Parameter measured				
	Young's Modulus	Shear Modulus	Thickness	Damping	Other
Paving layer	yes	yes	yes	no	temperature
Base	yes	yes	yes*	no	
subgrade	no	yes	no	yes	

*Thickness estimate of base depends on shear modulus contrast with subgrade.

7.2. Description of Measurement Technologies

7.2.1. Impulse-Response (IR) Method

Two parameters are obtained with the IR method-the shear modulus of subgrade and the damping ratio of the system. These two parameters characterize the existence of several distress precursors. In general, the modulus of subgrade can be used to delineate between good and poor support. The damping ratio can distinguish between the loss of support or weak support. The two parameters are extracted from the flexibility spectrum measured in the field. An extensive theoretical and field study (Reddy, 1992) shows that except for thin layers (less than 75 mm) and soft paving layers (i.e., flexible pavements), the modulus obtained by the IR method is a good representation of the shear modulus of subgrade, and the stiffness of the paving layers would influence the results insignificantly. In other cases, the properties of the pavement layers (AC and base) affect the outcome in such a manner that the modulus obtained from the IR test should be considered an overall modulus.

The IR tests use the low-frequency source and geophone G1 (figure 7.1). The pavement is impacted to couple stress wave energy in the surface layer. At the interface of the surface layer and the base layer, a portion of this energy is transmitted to the bottom layers, and the remainder is reflected back into the surface layer. The imparted energy is measured with a load cell. The response of the pavement, in terms of particle velocity, is monitored with the geophone and then numerically converted to displacement. The load and displacement time histories are simultaneously recorded and are transformed to the frequency domain using a Fast-Fourier Transform algorithm. The ratio of the displacement and load (termed flexibility) at each frequency is then determined.

For analysis purposes, the pavement is modeled as a single-degree-of-freedom (SDOF) system. Three parameters are required to describe such a system-natural frequency, damping ratio, and gain factor. The last two can be replaced by the static amplitude and the peak amplitude. These three parameters are collectively called the modal parameters of the system. The natural frequency and gain factor are used to determine the modulus of subgrade. The damping ratio is used directly.

To determine the modal parameters, a curve is fitted to the flexibility spectrum according to an elaborate curve-fitting algorithm that uses the coherence function as a weighing function (Richardson and Formenti, 1982). The poles, zeros, and gain factor obtained from the curve-fitting are easily converted to modal parameters. From these parameters, the modulus of subgrade is determined. The shear modulus of subgrade, G, is calculated from (Dobry and Gazetas, 1986)

$$G = (1 - \nu) / [2L\ Ao\ Is\ Sz] \qquad (7\text{-}1)$$

Where,
ν = Poisson's ratio of subgrade
L = length of slab, and
Ao = static flexibility of slab (flexibility at f = 0).

The shape factor, Sz, has been developed by Dobry and Gazetas (1986). The value of Sz is equal to 0.80 for a long flexible pavement.
Is (Reddy, 1992) is a parameter which considers the effect of an increase in flexibility near the edges and corners of a slab. Parameter Is is a function of the length and width of the slab, as well as the coordinates of the impact point relative to one corner. Depending on the size of the slab and the point of impact, the value of Is can be as high as 6.
The damping ratio, which typically varies between 0 to 100 percent, is an indicator of the degree of the slab's resistance to movement. A slab that is in contact with the subgrade or contains a water-saturated void demonstrates a highly damped behavior and has a damping ratio of greater than 70 percent. A slab containing an edge void would demonstrate a damping ratio in the order of 10 to 40 percent. A loss of support located in the middle of the slab will have a damping of 30 to 60 percent.

7.2.2. Spectral-Analysis-of-Surface-Waves (SASW) Method

The SASW method uses the Raleigh wave (R-wave) to determine the stiffness profile and layer thickness of thin concrete layers. The SASW system includes an impact device, two receiving transducers, and a two-channel waveform analyzer. The characteristics of the impact device and the relative positioning of the transducers are determined by the stiffness and thickness of the layers.
The R-wave produced by impact contains a range of frequencies, or components of different wavelengths. This range depends on the contact time of impact; the shorter the contact time, the broader the range of frequencies or wavelengths. The velocity of the individual frequency components is called phase velocity. For the component frequency of the impacts, a plot of phase velocity versus wavelength is obtained. This curve is used to calculate the stiffness profile of the test object. The experimental results are compared with theoretical curves until the results match (2).
The main drawbacks of SASW are the limitation on the maximum layer thickness of the two media, and the matching of theoretical and experimental data.
The Spectral-Analysis-of-Surface-Waves (SASW) method was mainly developed by Nazarian and Stokoe (1989). SASW is a seismic method that can determine shear modulus profiles of pavement sections nondestructively.
The key point in the SASW method is the measurement of the dispersive nature of surface waves. A complete investigation of a site with the SASW method consists of collecting data, determining the experimental dispersion curve, and determining the stiffness profile (inversion process).

The set-up used for the SASW tests is depicted in Figure 7.3. All accelerometers and geophones are active. The transfer function and coherence function between pairs of receivers are determined during the data collection.

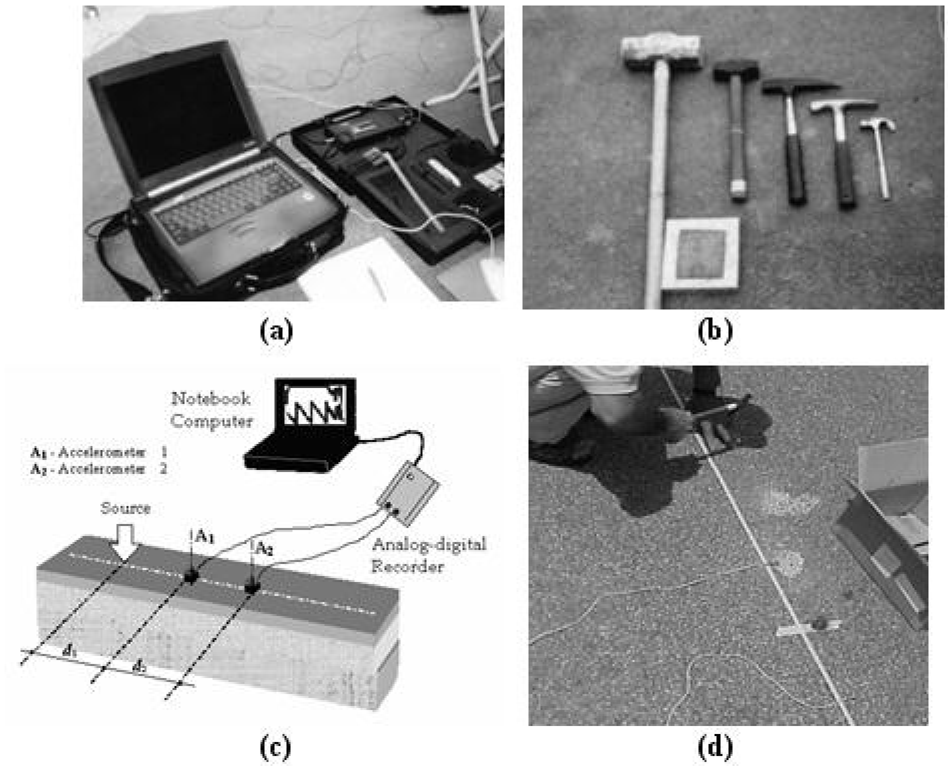


Figure7.3- (a) SASW equipments, (b) impact sources, (c) SASW experimental set up, and (d) SASW test conducted in field

A computer algorithm utilizes the phase information of the cross power spectra and the coherence functions from several receivers spacing to determine a representative dispersion curve in an automated fashion (Nazarian and Desai, 1993).
The last step is to determine the elastic modulus of different layers, given the dispersion curve. A recently developed automated inversion process (Yuan and Nazarian, 1993) determines the stiffness profile of the pavement section (Fugure 7.4).

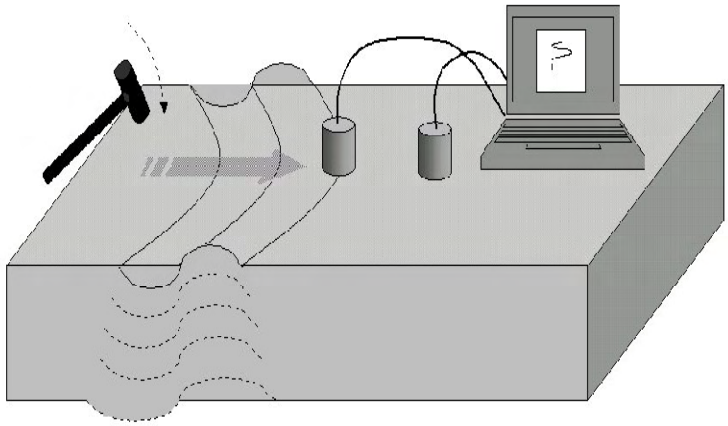


Figure 7.4 - Principle of testing with the SASW kit

7.2.3. Ultrasonic-Surface-Wave Method

The ultrasonic-surface-wave method is an offshoot of the SASW method. The major distinction between these two methods is that in the ultrasonic-surface-wave method the properties of the top paving layer can be easily and directly determined without a complex inversion algorithm. To implement the method, the high-frequency source and accelerometers A1 and A2 (Figure 7.1) are utilized.

Up to a wavelength approximately equal to the thickness of the uppermost layer, the velocity of propagation is independent of wavelength. Therefore, if one simply generates high frequency (short-wavelength) waves, and if one assumes that the properties of the uppermost layer are uniforms, the shear modulus of the top layer, G, can be determined from

$$G = \rho\ [(1.13 - 0.16)\ \nu\ V_{ph}]^2 \qquad (7\text{-}2)$$

Where,

Vph = velocity of surface waves
p = mass density
ν = Poisson's ratio

The thickness of the surface layer can be estimated by determining the wavelength above which the surface wave velocity is constant.

The methodology can be simplified even further. If one assumes that the properties of the uppermost layer are uniform, the modulus of the top layer, G, can be determined from

$$G = \rho\ [(1.13 - 0.16\nu)\ (m/360D)]^2 \qquad (7\text{-}3)$$

Parameter m (deg/Hz) is the least-squares fit slope of the phase of the transfer function in the high-frequency range.

7.2.4. Ultrasonic Compression Wave Velocity Measurement

Once the compression wave velocity of a material is known, its Young's modulus can be readily determined. The same set-up used to perform the SASW tests can be used to measure compression wave velocity of the upper layer of pavement.
Miller and Pursey (1955) found that when the surface of a medium is impacted, the generated stress waves propagate mostly with Rayleigh wave energy and, to a lesser extent, with shear and compression wave energy. As such, the body wave energy present in a seismic record generated using the set-up shown in figure 7.1 is very small; for all practical purposes it does not contaminate the SASW results. However, compression waves travel faster than any other type of seismic wave and are detected first on seismic records.
An automated technique for determining the arrival of compression waves has been developed. Times of first arrival of compression waves are measured by triggering on an amplitude range within a time window (Willis and Toksoz, 1983).

7.2.5. Impact-Echo Method

The Impact-Echo (IE) method (Figure 7.5) is a highly developed acoustic technique for detecting the presence of flaws and estimating their location in solid material. The equipment is also used to estimate the thickness of e.g. pavements and slabs.
Intensive research work on different applications of IE makes it a functional method for a variety of concrete problems. The IE tests rely on reflection of compression waves from the bottom of the structural member or from any hidden discontinuity. An instrumented hammer or an impactor is used as a source to generate compression waves which are sensed by a receiver after being reflected (3).

7.2.5.1. Applications

- Measurement of thickness of concrete elements
- Location of voids
- Location of cracks and crack depth measurement
- Detection of delamination caused by reinforcement corrosion
- Comparative surveys of concrete quality
- Qualitative surveys of bond strength between concrete layers.

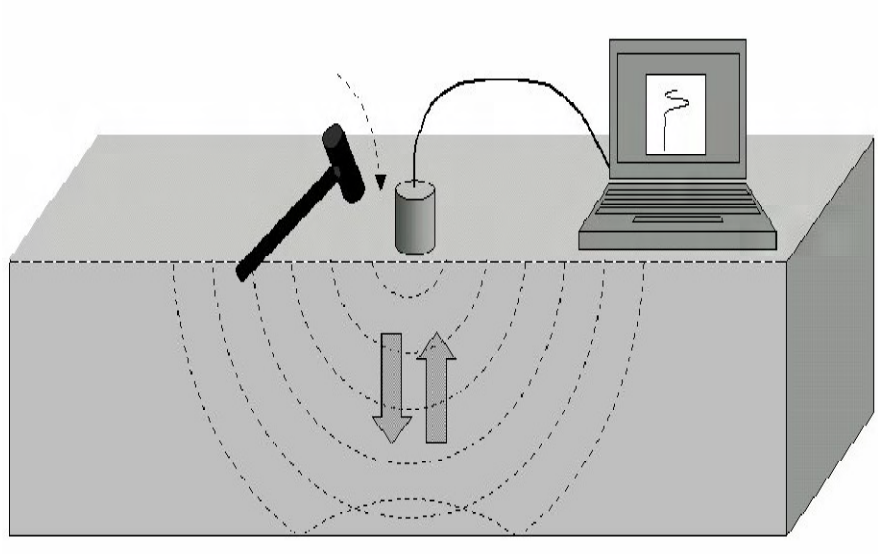


Figure 7.5 - Principle of testing with the Impact Echo

7.3. Precision of Measurements

The precision of the measurements was determined by conducting tests at the same locations between five and ten times. These tests were conducted at almost all sites tested. To determine the precision, it was assumed that the operator would not use any judgement at all; that is, the data were collected but never inspected during the tests. The coefficients of variation of the reduced data for Texas and Georgia are reported as the indication of precision in table 7.3.
In general, the precision reported for tests in Georgia is better than that obtained from tests in Texas. This improvement is the direct result of software changes made after tests in Texas. These precision levels are a function of acceptable levels of distress and the number of years before maintenance. In all cases, the precision levels reported in table 7.3 are much less than those necessary for small amounts of acceptable distress (less than 5 percent) with a three-year lead time for scheduling maintenance. Therefore, the precision reported in table 7.3 is quite adequate for maintenance purposes.

Table 7.3 - Precision associated with each measurement technique

Measurement technique	Precision percent	
	Texas	Georgia
Ultrasonic Surface Wave	5	5
Ultrasonic Body Wave	17	9
Impulse Response	8	6
Impact Echo	21	9

7.4. Advantages and Disadvantages of SPA

Advantages are (4):

- Reduces number of destructive tests required for determining pavement layer properties
- Results can be obtained within two minutes, since the data is analyzed on site

Limitations are (4):

- The testing is discrete by nature (i.e. the testing measures properties at a single point per test, and it takes two minutes per test)
- Not suitable for rapid 100% coverage testing
- unsuitability for testing composite pavements (5)
- unproven equipment reliability,
- and need for high skills relevant to data reduction and analysis

7.5. Conclusions

1. The new SPA nondestructive testing device is useful for maintenance activities.
2. The SPA can easily, accurately, and repeatably collects and reduces information about the condition of pavements.
3. The SPA meets or exceeds the specifications for accuracy and precision developed to determine its usefulness for maintenance.
4. The SPA is field-worthy and rugged, and can handle different climatic conditions.
5. The final versions of the software and hardware function well and accurately determine a wide range of pavement conditions.
6. The SPA is ready for commercialization.

References

(1) Soheil Nazarian, Mark R. Baker, Kevin Crain, Center for Geotechnical and Highway Materials Research, The University of Texas at El Paso ,El Paso, Texas 79968, "*Development and Testing of a Seismic Pavement Analyzer*"

(2) INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 2005," *Non-destructive testing for plant life assessment"* TRAINING COURSE SERIES No. 26

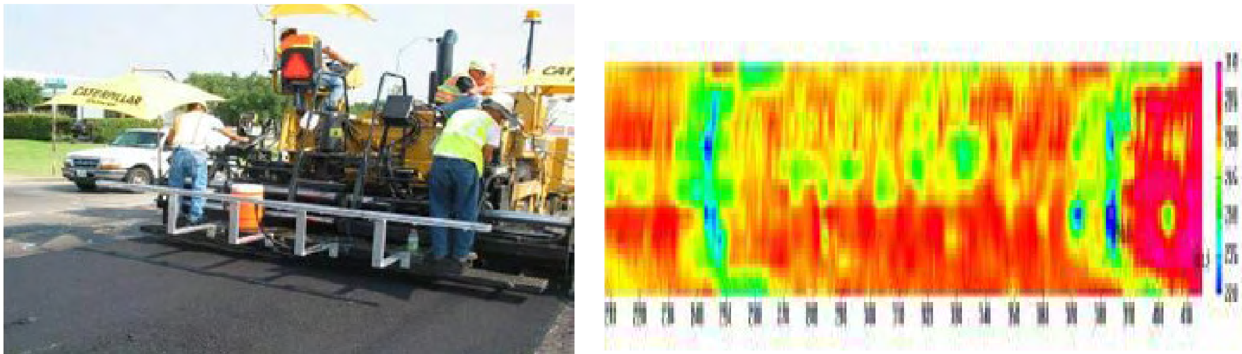
(3) Concrete Inspection and Analysis, Non-Destructive Testing (NDT) and Examination (NDE),Force Technology

(4) A. Wimsatt, S. Hurlebaus, T. Scullion &E. Fernando, Texas Transportation Institute College Station, TX, 77843, International Symposium on Nondestructive Testing for Design Evaluation and Construction Inspection, "*Promising Existing and Emerging Technologies & Techniques*"

(5) National Cooperative Highway Research Program ,RESEARCH RESULTS DIGEST December 2002—Number 271, *Determination of Insitu Material Properties of Asphalt Concrete Pavement Layers from Nondestructive Tests"*

8. HMA Temperature

Temperature measurement of the HMA mat during construction using infra-red cameras is very useful to investigate temperature uniformity of new HMA layers, detect thermal segregation, create a permanent log of paving operations, and locate and establish duration of paver stops. Figure 7.6 shows the HMA infra-red measurement set-up and and example of data representation. This system is currently in use by Washington and Texas DOT and NCAT.



(a) (b)

Figure 7.6 – HMA Infra-red Measurements: (a) Test Set up and (b) Example

Advantages

- Segregation of hot mix a continuing problem
- Newer lower cost camera systems widely available
- Automated system with 100% coverage
- Cameras and guns available

Limitations

- Equipment not widely available

Barriers to Implementation

- Unknown targets given the variability of PG gradations and mix types
- Not currently included in specifications