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Performance-Related Specifications for Concrete Pavements

Volume I: Development of a Prototype Performance-Related Specification

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

This report is one volume of a three-volume set of reports presenting the results of a study to further the development of performance-related specifications for portland cement concrete pavement construction. Laboratory testing was conducted to fill several gaps in the knowledge of portland cement concrete. Drawing upon the results of the laboratory testing, the underlying theory of performance-related specifications was considerably extended, and a prototype performance-related specification was developed. Also, a computer program was developed for use with the specification to assist in simulation and in generating pay adjustments. This report will be of interest to engineers concerned with quality assurance, specifications, and construction of concrete pavements.

Sufficient copies of this report are being distributed to provide two copies to each FHWA regional office and three copies to each FHWA division office and each State highway agency. Direct distribution is being made to the division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), United States Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161.

Byron N. Lord

Byron N. Lord, Acting Director
Office of Engineering and Highway
Operations Research and Development

NOTICE

U.S. Department
of Transportation
Federal Highway Administration

Research and Development

Turner-Fairbank Highway
Research Center
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McLean, Virginia 22101-2296

ERRATA

"Performance-Related Specifications for Concrete Pavements"

Volume II: Appendixes A, B, & C
Publication No. FHWA-RD-93-043

Because of an oversight, recipients of the above referenced publication are asked to make the following pen-and-ink changes to their copies of the report:

Location	**Change**
Foreword, 1st para, 5th sentence	**Strike out** the words "contains two appendixes which"
Foreword, 2nd paragraph	**Change from** "one copy to each FHWA regional and division office and to each State highway agency." **to** "two copies to each FHWA regional office and three copies to each FHWA division office and each State highway agency."

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16. Abstract

This study continued the development of performance-related specifications (PRS) for concrete pavements. Drawing upon previous work, a prototype PRS was developed that considers the expected life-cycle cost of the as-constructed pavement as the overall measure of quality. The approach calls for measurement of *in situ* concrete properties and explicitly considers variability and multiple quality characteristics in the determination of pay adjustments. Extensive laboratory testing was conducted to determine material relationships needed in the prototype PRS, and a detailed test plan has been developed for the evaluation of construction variables (e.g., dowel misalignment) that significantly affect concrete pavement performance, but are not currently accounted for in the specification. A computer program, PaveSpec, has been developed for use with the specification in simulation and in generating pay adjustments.

This volume describes the development of the prototype PRS. Concrete strength, slab thickness, air content, and initial roughness are included in the specification as the key quality characteristics. Both cost models and distress prediction models are used to compute life-cycle costs. The difference between the life-cycle costs of the target, as-designed pavement and the actual, as-constructed pavement is used to determine the pay adjustment. Numerous examples on the use and sensitivity of the specification are presented. A summary of the laboratory testing results that were used in the specification is given, along with a test plan for the evaluation of quality characteristics not currently included in the specification.

This volume is the first in a series. The other volumes are:

FHWA-RD-93-043	Volume II:	Appendix A—Prototype Performance-Related Specification Appendix B—PaveSpec Users Guide Appendix C—Annotated Bibliography
FHWA-RD-93-044	Volume III:	Appendix D—Laboratory Testing Procedures and Testing Results Appendix E—Review of Recent Studies and Specifications

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

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

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

(Revised September 1993)

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LIST OF ACRONYMS AND ABBREVIATIONS

A	Air Content
AAMAS	Asphalt-Aggregate Mixture Analysis System
AASHTO	American Association of State Highway and Transportation Officials
A-C	As-Constructed
AC	Asphalt Concrete
ACI	American Concrete Institute
A-D	As-Designed
ADT	Average Daily Traffic
AEQ	Air-Entraining Quantity
AGE	Age of Specimens at Time of Testing
AGED	Age of Specimens at Time of Testing Deviation from Mean Value
AIR	Air Content
AIRD	Air Content Deviation from Mean Value
AQL	Acceptable Quality Level
ARR	Arrhenius Maturity
ASR	Alkali-Silica Reactivity
ASTM	American Society for Testing and Materials
CAG	Coarse Aggregate Geometry
CAGD	Coarse Aggregate Geometry Deviation from Mean Value
CAH	Coarse Aggregate Hardness
CAHD	Coarse Aggregate Hardness Deviation from Mean Value
CAM	Coarse Aggregate Maximum Size
CAP	Coarse Aggregate Volume Percentage
CAPD	Coarse Aggregate Volume Percentage Deviation from Mean Value
CAQ	Coarse Aggregate Quantity
CAT	Coarse Aggregate Type
CEP	Cement Volume Percentage
CEPD	Cement Volume Percentage Deviation from Mean Value
CEQ	Cement Quantity
CET	Cement Type
CH	Crushed-Hard
COV	Coefficient of Variation
CPF	Composite Pay Factor
CRCP	Continuously Reinforced Concrete Pavement
CS	Crushed-Soft
CSL	Consolidation Level
CSLD	Consolidation Level Deviation from Mean Value
DDE	Dynamic Data Exchange
DOT	Department of Transportation
E_c	Concrete Elastic Modulus
EMS	Error Mean Square
EP	Expected Pay Value
ESAL	Equivalent Single-Axle Load

LIST OF ACRONYMS AND ABBREVIATIONS (Continued)

f'_c	Concrete Compressive Strength
F-T	Freeze-Thaw
FAM	Fine Aggregate Fineness Modulus
FAP	Fine Aggregate Volume
FAQ	Fine Aggregate Quantity
FAT	Fine Aggregate Type
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
HRWR	High-Range Water Reducer
G	Gradation
GPR	Ground-Penetrating Radar
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
L	Void Spacing Factor
LCC	Life-Cycle Cost
LCC_{con}	Life-Cycle Cost of the As-Constructed Pavement
LCC_{des}	Life-Cycle Cost of the As-Designed Pavement
LTPP	Long-Term Pavement Performance
M&C	Materials & Construction
MESAL	Millions of ESAL's
MR	Concrete Modulus of Rupture (Flexural Strength)
MS	Mean Square
n	Number of Samples (= Number of Sublots)
NCHRP	National Cooperative Highway Research Program
NDT	Nondestructive Testing
NS	Nurse-Saul Maturity
OC	Operating Characteristic
PCC	Portland Cement Concrete
PD	Percent Defective
PDL	Percent Defective Length
PF	Pay Factor
PPA	Percent Pay Adjustment
PRH	Percent Relative Humidity
PRS	Performance-Related Specifications
PSI	Present Serviceability Index
PSR	Present Serviceability Rating
PV	Pulse Velocity
PWT	Percent Within Tolerance
Q	Quality Index
QC	Quality Characteristics
QCSP	Quality Characteristic and Sampling Parameters
R	Initial Roughness

LIST OF ACRONYMS AND ABBREVIATIONS (Continued)

R^2	Coefficient of Determination
Rep. RMS	Replicate Root Mean Square
Rep. MS	Replicate Mean Square
RH	Round-Hard
RMS	Root Mean Square
RQL	Rejectable Quality Level
RS	Round-Soft
S	Concrete Strength
S_{con}	Standard Deviation of the LCC of the As-Constructed Pavement
SEE	Standard Error of Estimate
SHA	State Highway Agency
SHRP	Strategic Highway Research Program
SSD	Saturated Surface Dry
ST	Concrete Splitting Tensile Strength
Std. Dev.	Standard Deviation
T	Slab Thickness
TRB	Transportation Research Board
UC	User Cost
WAP	Water Volume Percentage
WAQ	Water Quantity
WASHTO	Western Association of State Highway and Transportation Officials
WC	Water-Cement Ratio
WCD	Water-Cement Ratio Deviation from Mean Value
WIM	Weigh-In-Motion
WWF	Welded-Wire Fabric

CHAPTER 1. INTRODUCTION

BACKGROUND

In recent years, there has been a growing interest in the development of performance-related specifications (PRS) for highway pavement construction. PRS are specifications on key material and construction quality characteristics (e.g., strength, air content, dowel bar alignment) that have been shown to correlate significantly with the performance of the pavement. Through the use of PRS, a methodology is provided by which the quality of pavement construction can be related to the performance and costs of a given project. This ability can lead to the identification of an optimum level of construction quality and the rational determination of pay adjustments for a specific project.

The concept behind PRS is not new, and is actually reflected in other types of specifications. Indeed, the underlying purpose of both methods specifications (in which the owner agency specifies the materials and methods for doing the job) and end-result specifications (in which the contractor is given considerable freedom in performing a job, provided that the specified results are achieved) is to designate standards or specify test results that are expected to produce a long-lasting pavement; often, these methods or test values are based on those that have provided good performance in the past.

However, PRS goes beyond those specifications by having the ability to identify levels of quality associated with specifying different values for a given quality characteristic, whereas most current specifications are unable to identify the level of quality that is being produced. For example, in a PRS for a specified quality characteristic (say, strength), the performance of the pavement can be estimated using established relationships, and the level of quality determined in terms of desirable performance (acceptable quality) or undesirable performance (unacceptable quality). In this way, PRS permits the identification and specification of the optimum level of quality that provides the best balance between costs and performance for a given project. It has been estimated that such improvements in quality control procedures could produce long-term benefits to agencies and users measured in billions of dollars.[1]

PRS also provides a rational basis for which pay adjustments can be determined. The key pavement material and construction quality characteristics are measured at the time of construction on the *in situ* pavement and then used in established relationships to predict the performance of the pavement. If these results indicate an increase or decrease in the performance of the pavement, appropriate pay adjustments can be made—either to reward work that results in an increase in pavement life or to penalize work that results in a decrease in pavement life. The amount of the bonus (incentive) or penalty (disincentive) is based on the increase or decrease in future costs that will be incurred by the agency and potential users over the life of the project, assuming a given rehabilitation policy.

The development of PRS requires a thorough understanding of how material and construction quality characteristics affect the performance of a pavement. Unfortunately, the effects of all material and construction quality characteristics on performance are not well understood. At present, only a few agencies are using a limited PRS, and while several studies have been sponsored on the development of PRS for both asphalt concrete (AC) and portland cement concrete (PCC) pavements, there is a great need to fill many of the existing gaps so that a true, working PRS may be developed.

PROJECT OBJECTIVES AND SCOPE

The focus of this study is to continue the development of PRS for PCC pavement construction through the fulfillment of the following objectives:

1. Examine the results and recommendations of previous studies.
2. Develop a detailed laboratory testing plan and conduct the laboratory study to quantify the relationships among selected materials variables.
3. Develop fully the materials portion of PRS through the preparation of a prototype specification.
4. Develop a detailed plan for accelerated field tests to include experimental designs, construction details, and data collection and analysis procedures.

To address these objectives, the research team performed an extensive literature review. By far the most useful information on PRS development comes from the New Jersey Department of Transportation (DOT).[2] Other key studies include the work done under National Cooperative Highway Research Program (NCHRP) Project 10-26A (see reference 3) and a previous Federal Highway Administration (FHWA) study on PRS for PCC pavements (see reference 4). In addition, there are several other current State specifications that provide insight into PRS.

A comprehensive laboratory investigation of materials variables was also conducted. This study helped to fill several of the gaps in the materials area of the PRS.

Drawing upon the literature and the laboratory testing, the underlying theory of PRS was considerably extended, and a prototype performance-related specification was developed. This prototype specification considers the expected life-cycle cost (LCC) of the as-constructed pavement (that pavement actually constructed by the contractor) as the overall measure of quality, and explicitly considers variability and multiple quality characteristics in the development of pay schedules. The pay schedules are based on comparisons between the target as-designed pavement (that pavement specified by the designer) and the as-constructed pavement. A computer program, PaveSpec, has been developed for use with the specification in simulation and in generating pay schedules.

Recognizing that many distresses in concrete pavements are related to construction variables, detailed plans for the investigation of construction quality characteristics were developed. These field-testing plans, if implemented, could be used to identify the effect of various construction quality characteristics on concrete pavement performance. These variables could then be incorporated into the PRS.

To guide and direct the research team in dealing with many of the complex issues involved, an advisory panel consisting of experienced engineers and statisticians was assembled. This panel represented a broad cross-section of individuals knowledgeable in the areas of concrete pavement design and construction, concrete materials, concrete pavement performance, statistics, and quality assurance specifications. The advisory panel provided valuable input into the development of the specification and reviewed all pertinent project documentation. Members of the advisory panel include:

- Mr. Jim Duit, Duit Construction Co.
- Mr. Jim Grove, Iowa Department of Transportation.
- Mr. Clint Solberg, American Concrete Pavement Association.
- Mr. Garland Steele, Steele Engineering, Inc.
- Mr. Richard Weed, New Jersey Department of Transportation.

SEQUENCE OF REPORT

Chapter 2 of this report describes the approach proposed by the research team to continue the development of PRS and identifies key characteristics that should be part of a successful PRS. Chapter 3 describes the development of the model PRS, provides several detailed examples on the use of the model PRS, and describes the PaveSpec computer program used in the simulation of the specification. Chapter 4 reports on the results obtained from the laboratory testing studies and chapter 5 proposes testing plans for the evaluation of several construction and material quality characteristics that are not currently represented in the model specifications. Finally, chapter 6 summarizes the results of the overall study and provides recommended areas for future research.

Several appendixes are included in support of the final report. Appendix A contains the model PRS in its entirety. Appendix B is a users guide for the PaveSpec computer program that was developed under this project. Appendix C contains an annotated bibliography of selected literature regarding performance-related specifications and quality control issues. Appendix D provides a detailed description of the laboratory testing studies and presents all of the data obtained from those studies. Finally, appendix E summarizes some of the key work that has been conducted in the area of performance-related specifications.

CHAPTER 2. APPROACH TO PRS DEVELOPMENT

INTRODUCTION

The major goal of conducting research into PRS is to quantify the relationships between quality and subsequent performance. If this objective could be met, it would be possible to identify the optimum quality level that provides the best balance between cost and performance. This optimum quality level can be identified and specified through the use of a performance-related specification that adequately considers both cost and performance. The major concern is in identifying and specifying the quality that will give the most for the investment. The LCC approach to PRS development described in this report is believed to be a good start toward that goal.

SUMMARY OF PREVIOUS PRS RESEARCH

There have been several studies conducted on the development of PRS during the past decade. However, three separate research efforts in particular have laid the foundation for and continued the development of PRS. These three studies are:

- New Jersey DOT.[2] The most significant work conducted by far is that done by the New Jersey DOT under the direction of Mr. Richard Weed. This work has led to a sound, fundamental basis for PRS, and also has provided a reasonable PRS for one quality characteristic (concrete strength). This specification has been implemented in New Jersey and has reportedly resulted in greater concrete strengths on most paving projects. The key aspect of the specification is that it relates measured concrete strength to pavement performance through the use of a predictive model. Performance is related to the future LCC of the pavement so that rational pay adjustments can be computed for a pavement lot. The amount of the pay increase or decrease depends on the anticipated future costs that result from the higher or lower quality work.

- NCHRP Project 10-26A.[3] This project, conducted under the sponsorship of the National Cooperative Highway Research Program, investigated PRS for hot-mix asphalt concrete, although the general framework developed is applicable for all pavement types. This study emphasizes the use of materials and construction (M&C) variables that are performance-related and that can be controlled by the contractor. The use of "economic life" is suggested for comparing alternatives in this approach, which is defined as the time within the initial performance period at which the equivalent uniform annual cost has a minimum value. This concept is frequently used for equipment replacement analyses in industrial engineering applications.

- FHWA Report FHWA-RD-89-211.[4] This FHWA research study concentrated on the development of PRS for concrete pavements, using the same conceptual framework adopted under NCHRP project 10-26A. This study recommends the New Jersey DOT approach for acceptance plans, with materials and construction variables recommended for three key factors: 28-d compressive strength, slab thickness, and as-constructed surface profile. The AASHTO rigid pavement design model (see reference 5) and the COPES distress models (see reference 6) are recommended to predict the performance of both the as-designed and the as-constructed pavements. Initial construction costs, maintenance costs, overlay costs, and salvage value are included in the LCC analysis.

While these studies and approaches do have some inherent limitations, they serve as the basis for the work presented in this report. More detailed information on these studies is presented in appendix E.

OBJECTIVES OF A WORKING PRS

In the development of this work, it became apparent that the objectives of a working PRS must be identified in order to continue its development. One objective of a true PRS, for example, is to establish fair and equitable procedures for assigning pay factors to work that differs from the specified quality level. Such a specification should recognize the consequences of both substandard and high-quality work and should provide a strong incentive for the contractor to produce the desired level of quality. Previous work has concluded that the pay adjustments made to the contractor's bid should correspond to the present worth of the anticipated extra increase (or decrease) in costs resulting from defective (or superior) work, based on the legal principle of liquidated damages.[2,3,4] The difference in present worth costs provides a rational and defensible basis for developing fair and equitable pay adjustments.

The research team has identified 12 major characteristics of a working PRS for concrete pavements. These characteristics, adapted from reference 2, are:

1. The exact requirements must be clearly communicated to the contractor. The as-designed (target) pavement must be defined in terms of material and construction quality characteristics (defined as materials and construction variables under the control of the contractor that relate to performance) and other factors.

2. The contractor should be responsible for controlling the construction process while the agency should be responsible for judging the acceptability of the completed work.

3. The specification should be tailored to include the three major types of conventional concrete pavements (JPCP, JRCP, CRCP). The significant construction and performance differences between these pavements must be recognized and considered in the specification development. Research in this report is specifically related to JPCP and JRCP.

4. The specification should be driven by the key distress indicators that control the service life and subsequent future life-cycle costs of the pavement. Examples of distress indicators include several types of cracking, joint faulting, joint and crack spalling, initial roughness, scaling, and punchouts. Examples of future costs include maintenance, rehabilitation, and highway user costs.

5. The specifications should include all materials and construction quality characteristics that not only significantly affect the performance of the pavement, but that are also under the control of the contractor.

6. Both the mean and the variability of measured quality characteristics must be considered in the specification or it will be seriously deficient.

7. There must be a strong incentive to the contractor to produce the desired level of quality or better. This can be accomplished by means of an adjusted pay schedule that assesses pay reductions for deficient quality and grants pay increases for superior quality.

8. The acceptance plan must be practical for field implementation. This means rapid, reliable sampling and test methods that meet the required statistical criteria of unbiasedness and sufficiency must be used. The acceptance plan must include information on lot size, key quality characteristics upon which acceptance will be based, sample size, random sampling procedures, test methods, and rapid test result processing to compute the lot pay factor.

9. The specification must be fair and equitable in assigning pay factors for work that differs from the desired quality level. The pay adjustment should correspond to the present worth of the anticipated difference between the life-cycle costs of the as-designed and as-constructed pavements.

10. The specification should be realistic in defining the acceptable quality level (AQL) and rejectable quality level (RQL) values. The AQL is that level of quality, usually defined in terms of some minimal degree of deficiency, that the specifying agency is willing to accept at 100-percent payment.[2] The RQL is that level of quality that is so deficient that immediate repair, removal, or replacement is needed. The AQL should be set at a level that satisfies design requirements, but not so high that extraordinary construction methods or materials will be required. At the other extreme, the RQL should be set at a sufficiently low level of quality that the option to require removal and replacement at the contractor's expense is truly justified. Typically, the RQL should provide a pay factor between 70 and 80 percent.

11. The specification should pay 100 percent, on the average, when the quality is exactly at the AQL.

12. The specification should include *in situ* measurements of the newly placed concrete slab, not on samples taken from the plant or trucks.

NEEDED IMPROVEMENTS TO EXISTING SPECIFICATIONS

Current specifications for concrete pavement are either traditional method or end-result specifications, both of which have limitations. Traditional method specifications do not directly consider product variability or the effect of substandard or high-quality workmanship. End-result specifications have the following limitations:[7]

- The inability to identify or measure the essential performance-related characteristics of the pavement.
- The inability to quantify substantial compliance and to determine price adjustment factors that relate to reduced or enhanced value.
- Uncertainty as to the value to be gained from the costs of implementing statistically based, end-result specifications.
- End-result specifications may guarantee improved compliance and improved evidence of compliance, however, they do not guarantee improved performance.
- Improved performance depends on a better understanding of the relationship between the factors controlled during construction and the performance and worth of the finished product.

The last item is perhaps the key aspect of a PRS that sets it apart from an end-result specification.

Several end-result type specifications contain a pay schedule based upon one or more quality characteristics (e.g., thickness, strength, roughness) to reflect the effect of work quality that differs from the target. While these specifications are certainly a step in the right direction, the pay adjustments are all determined through the subjective judgment of the developers (with the exception of the NJ DOT specification for concrete strength). Furthermore, with the exception of roughness, most of these specifications only provide for negative pay adjustments. The provision of a subjectively determined negative pay adjustment is very likely unrealistic and may reduce the incentive to the contractor to produce higher quality work.

For those end-result specifications consisting of more than one quality characteristic, the final pay factor is often determined by multiplying, averaging, or applying weighting factors to individual pay factors, or by simply using the lowest of all individual pay factors. This can lead to a very unrealistic pay schedule and strong opposition from the construction industry, who perceive these specifications as being unfair.

Another limitation is the lack of consideration of key quality characteristics that significantly affect the performance of the pavement. It is believed that a PRS must be driven by the key quality characteristics that significantly affect the performance of the pavement and that are under the control of the contractor.

It should be pointed out that the existing specifications in use are "performance-related" to the extent that they do attempt to control factors that are believed to significantly affect pavement performance. However, the connection between the quality characteristics measured and expected pavement performance is subjective, so that the specifications are not able to rationally relate the level of a given quality characteristic (i.e., initial roughness) to future performance.

It is within this framework that the following plan for continued development of PRS for concrete pavements is proposed. The specifications and previous work were used as the basis for the continued development of PRS for concrete pavements. While these approaches do contain some limitations, they lay a strong foundation in certain areas. This is particularly true of the approach used by the New Jersey DOT, which serves as the cornerstone of the specification proposed herein.

To continue the development of a PRS for concrete pavements, it is believed that major improvements are needed in at least the following specific areas:

- Development of the underlying PRS theory and concepts for relating the key quality characteristics to pavement performance and to the future costs of the pavement.

- Development of a more realistic procedure for determining the pay adjustment when dealing with more than one quality characteristic. The pay adjustment should reflect the relative and interactive effects of multiple quality characteristics (e.g., thickness, strength, air content, and so on).

- Use of direct measurement of *in situ* concrete slab quality characteristics. This will provide the best estimate of as-constructed conditions for use in determining the required pay adjustments.

- Identification and inclusion of additional quality characteristics that are under the control of the contractor and that affect performance of the pavement. Currently, several key materials and construction quality characteristics are missing in existing specifications.

- Development and inclusion of more reliable, rapid, and meaningful test methods to measure each quality characteristic. This is a top priority item so that process changes can be made as soon as problems are identified.

- Generation of the operating characteristic curve for the specification to determine how it performs over a broad range of possible quality levels and paving conditions.

- Consideration of within-lot pavement variability in determining the pay adjustments. It is well known that increased variability of key factors along a pavement results in increased localized deterioration.

PROPOSED APPROACH

An innovative approach to the problems of considering both multiple quality characteristics and within-lot pavement variability has been developed. This approach uses a single overall quality characteristic for acceptance, yet it can include any number of construction and material quality characteristics. This overall quality characteristic is the future LCC of the as-constructed pavement lot. This approach makes it possible to develop a rational PRS where any number of quality characteristics and their variation can be included. Some of the major advantages of this approach are as follows:

- **One overall quality characteristic for acceptance.**

  The problem with current specifications is that to fairly evaluate the as-constructed pavement lot, it is essential to consider several quality characteristics that are strongly correlated with pavement performance. However, each of the selected quality characteristics has its own individual pay factor, which makes the rational determination of an overall pay factor impossible. Methods that have been used to determine an overall pay factor include averaging or weighting the individual pay factors, using the lowest pay factor, and multiplying the individual pay factors by one another to produce a new pay factor. Such procedures are typically applied because of the inability to rationally combine the different quality characteristics into a single acceptance plan.

  The proposed approach considers the LCC of the as-constructed pavement as the overall quality characteristic. This allows the use of as many quality characteristics as desired, provided that reliable models exist to predict the effect of these characteristics on pavement performance. Since the most accurate estimation of the pavement quality is a function of the combined values of all quality characteristics, the use of LCC produces the true and interactive effect of such combinations of quality characteristic values. Therefore, tradeoffs between different quality characteristics are easily considered in the determination of the pay factor (e.g., an increase in slab strength may offset a slight deficiency in slab thickness).

- **PRS Driven By Key Distress Indicators.**

  A key aspect of PRS is the direct consideration of the key distress indicators that affect the service life and thus cost of the pavement. These key distress indicators may include, for example, several types of cracking, joint and crack spalling, punchouts, initial smoothness, and initial surface friction.

- **Consideration of variability in the acceptance plans.**

  Variability of any pavement property is perhaps the most important aspect related to pavement performance (and thus to LCC). A pavement having higher variability in any given quality characteristic, such as air content or dowel alignment, leads to increased deterioration and increased LCC. Overall pavement variability can be classified into two main categories:

  1. Within-lot material and construction variability that are mainly the sole responsibility of the contractor. There should be procedures for detecting, evaluating, quantifying, and estimating the effects of these variability sources, with acceptance based in part on the amount of product variability.

  2. Sampling, testing, performance prediction error, and any other error sources over which the contractor has no control. In addition to the requirements discussed for material and construction variability, these variability contributions should be minimized through sound procedures for sampling, testing, and performance prediction. The importance of estimating sampling and testing errors is to make allowances in the PRS for these sources of variability.

  The procedures developed herein are based on the concept that a certain amount of variability is not only inherent in the process, but is also acceptable.

- **Unique pay factor relation for each project.**

  Since each project is unique in its actual as-designed characteristics, it follows that a unique pay factor relationship should exist for that project. For example, a thick, doweled slab with a stabilized base performs differently than a thin, nondoweled slab. It is also probable that the as-designed target values of the quality characteristics will be different from one project to another. Thus, it is important to include a clear definition of the target as-designed pavement within the specification itself. Furthermore, the variability of quality characteristics is often a function of the mean.

- **Use of *in situ* sampling.**

  Since the pay factor is based on the LCC of the as-constructed pavement, direct sampling from the completed pavement is required. For example, sampling from concrete trucks is not believed to be an accurate representation of the actual, *in situ* properties of the concrete slab.

DISTRESS INDICATORS THAT DRIVE THE PRS

An underlying basis for PRS is the direct consideration of the key distress indicators that affect the service life of the pavement. A comprehensive listing of concrete pavement distress indicators assembled under this study is shown in table 1.

Table 1. Comprehensive listing of concrete pavement distress indicators related primarily to causes within the slab.

1. Transverse cracking due to repeated loading (strength/thickness).
2. Transverse cracking due to inadequate or late sawing.
3. Transverse cracking due to drying shrinkage of PCC.
4. Longitudinal cracking due to inadequate or late sawing or improper joint forming.
5. Longitudinal cracking due to improper tiebar design.
6. Transverse crack deterioration in JRCP due to inadequate steel, corroded steel, or locked joints.
7. Transverse joint spalling/scaling due to inadequate air void system.
8. Transverse joint spalling due to D-cracking or reactive aggregate.
9. Transverse joint spalling due to dowel misalignment.
10. Transverse joint spalling due to incompressibles in joint.
11. Transverse/longitudinal joint spalling due to early or improper sawing.
12. Longitudinal spalling due to improper tiebar placement.
13. Longitudinal spalling due to keyway failure from improper forming.
14. Transverse joint faulting due to mix deficiency (small aggregate).
15. Transverse joint faulting due to base erosion (caused by base, not slab).
16. Transverse joint faulting due to inadequate load transfer design.
17. Scaling due to improper finishing techniques.
18. Edge slump due to improper slump of PCC mix.
19. Inadequate (non-uniform) cross slope of pavement.
20. Corner breaks from loss of support, poor load transfer, or poor drainage.
21. Initial smoothness.
22. Surface friction loss from improper tining.
23. Spalling due to steel near surface.
24. Overall roughness (Present Serviceability Index) of pavement.
25. Popouts.
26. Edge punchouts in CRCP.
27. Reflection cracking from subbase.
28. Surface irregularities due to rain.
29. Joint spalling, blowups, bridge pushing, or other pressure-related damage due to inadequate joint sealant installation.

Note that these indicators are related almost exclusively to the concrete slab. Distress indicators that are related to other pavement components (such as the subgrade, base, and shoulders) are not listed, although these ultimately should be included in a more comprehensive PRS.

The significance of the distress indicators listed in table 1 were rated by the expert panel members, FHWA representatives, and research team members. Each distress was rated in terms of its significance to concrete pavement performance, assuming that the distress had occurred. Based on the results of that rating, several key distress indicators were identified that significantly affect concrete pavement performance and that are under the control of the contractor, as described in the following sections.

Materials-Related Distress Indicators

Two concrete pavement distress indicators were identified that are related to properties of the concrete mix. These indicators are:

- Transverse cracking due to repeated loading and thermal curling stresses caused primarily by inadequate concrete strength and insufficient slab thickness.
- Transverse joint spalling caused by an inadequate air void system.

Construction-Related Distress Indicators

Numerous pavement distress indicators were identified that are related to concrete pavement construction activities. These distress indicators, which are not typically included in existing specifications, include:

- Transverse cracking caused by inadequate or late joint sawing.
- Longitudinal cracking caused by inadequate or late joint sawing or improper joint forming.
- Transverse joint spalling caused by dowel misalignment.
- Transverse/longitudinal joint spalling caused by early or improper sawing.
- Longitudinal joint spalling caused by improper tiebar placement.
- Initial roughness built in during construction.
- Surface friction loss caused by improper tining or surface texturing.
- Spalling caused by steel too near surface.

- Transverse joint spalling caused by improper densification of concrete surrounding the dowel bars.

- Punchouts in CRCP caused by improper steel placement or consolidation.

- Problems such as joint spalling, blowups, and bridge pushing problems caused by improper installation of joint sealant.

- Reflection cracking in slab caused by cracks in the underlying base course.

- Surface scaling caused by inadequate curing procedures.

Discussion of Distress Indicators

The distress indicators listed above are considered to be those that control the life of a concrete pavement, are under the control of the contractor, and (with the exception of faulting) are not related to other components, such as the subgrade, base, or shoulder of the overall pavement/subgrade structure. It is probable that additional distress indicators may be identified as the work on PRS evolves and especially as direct consideration of other components are included. The approach presented in this study allows any number of distress indicators to be included.

Distresses caused by poor or inadequate design were not considered, since these are not under the control of the contractor. For example, excessive joint spacing can cause transverse slab cracking to occur. Similarly, the mechanism responsible for the development of transverse joint faulting, a major distress of jointed concrete pavements, was not judged to be under the control of the paving contractor and, therefore, that distress type was not included. PCC durability problems caused by D-cracking and reactive aggregate, although considered to be very significant, were also judged not to be under the control of the paving contractor in most situations, since the agency generally approves materials prior to construction. Finally, the base or subbase layers, subgrade, shoulders, and any subdrainage system are not addressed here, although they certainly could be considered in future PRS work since the PRS theory included herein is expandable to any other pavement component.

SIGNIFICANT QUALITY CHARACTERISTICS

Each of the key distress indicators identified above can be related to one or more variables that affect its development. These variables, called quality characteristics, are under the direct control of the contractor, can be measured during construction, and can be used to estimate the future performance and costs of the pavement.

A listing of the various distress indicators and their corresponding measured materials and construction quality characteristics is provided in table 2. For each of the quality characteristics listed in table 2, a standard test must be conducted during construction to measure the value of the quality characteristic. Ideally, these tests should be rapid, repeatable, and suitable for use in the field. Future work on PRS must focus on the identification and development of such rapid field tests.

Table 2. Distress indicators and corresponding measured quality characteristics.

DISTRESS INDICATOR	MEASURED QUALITY CHARACTERISTIC(S)
Transverse cracking caused by loading and thermal curling	• Flexural strength • Slab thickness
Transverse joint spalling	• Air-void system • Timing of joint sawing • Dowel bar alignment • Improper densification of concrete surrounding dowel bars
Longitudinal joint spalling	• Air-void system • Timing of joint sawing • Depth/alignment of tiebars
Random transverse cracking	• Timing of joint sawing • Depth of joint sawing
Random longitudinal cracking	• Timing of joint sawing • Depth of joint sawing
Surface roughness	• Initial surface profile
Low surface friction	• Initial surface friction
Scaling/spalling throughout slab	• Depth of reinforcement
Punchouts and crack spalling	• Depth of reinforcement (CRCP only)
Transverse joint spalling, blowups, and bridge pushing problems	• Improper joint sealant installation

LIFE-CYCLE COST AS THE OVERALL QUALITY MEASURE

One of the major findings of this work is that the LCC of the as-constructed pavement can be used as the overall quality characteristic to be controlled. The LCC of the as-constructed pavement can be related to several of the distress indicators previously identified, which, in turn, are a function of various quality characteristics that are measured either directly or indirectly during construction. It follows, then, that the quality characteristics can be measured at the time of construction and used to estimate the LCC of the as-constructed pavement lot. At this time, four significant *in situ* quality characteristics are included in this specification:

1. Concrete strength, S.
2. Slab thickness, T.
3. Air content, A.
4. Initial roughness, R.

Any number of other characteristics (such as dowel bar placement or depth to reinforcement) could be included as long as there exist predictive models to estimate their effect on pavement performance and, subsequently, on the LCC of the as-constructed pavement.

Estimating Life-Cycle Costs

In order to apply LCC as the overall measure of quality, it must be estimated for both the as-designed and the as-constructed pavements. On this estimation rests the overall validity of the PRS. The LCC computations should ideally include the stream of all future costs related to the pavement over the design analysis period. These costs include future maintenance costs, rehabilitation costs, user costs, and the salvage value at the end of the analysis period.

The pay factor is calculated by considering the difference between the target as-designed pavement LCC (denoted by LCC_{des}) and the as-constructed pavement LCC (denoted by LCC_{con}) in conjunction with the contract bid price. A lower lot LCC_{con} indicates an increase in quality, while a higher lot LCC_{con} indicates a decrease in quality.

Target As-Designed Pavement

Performance-related specifications require a very clear definition of the target as-designed pavement that the contractor is expected to construct in order to receive full pay. This is because the ultimate pay that the contractor will receive for a lot will be a direct function of the difference between the LCC of the as-designed and the as-constructed pavement lot. Therefore, the inputs for these LCC predictions are very important and must be clearly defined. The inputs required are those used in the prediction models for the distress indicators and for the subsequent LCC calculations. The input variables are divided into two groups:

- Quality characteristics. Quality characteristics include material and construction variables that are under the control of the contractor and that are used for acceptance by the agency. For the initial version of the PRS, these include the means and standard deviations of concrete strength (S), slab thickness (T), entrained air content (A), and initial roughness (R). These variables may differ between the as-designed and as-constructed pavements. Of course, these four variables are not all of the quality characteristics that must be controlled on a concrete pavement construction project; these are only the ones that have been selected for this initial PRS development. Other variables, such as aggregate gradations, dowel alignment, and steel placement, must be controlled on a conventional acceptance/rejection basis.

- Constant variables. Constant variables are all other variables required to make the LCC predictions, such as traffic loadings, climatic factors, joint sealant type, shoulder type, effective *k*-value, dowel bar diameter, unit costs for rehabilitation, and so on. These constant variables are exactly the same for the as-designed and the as-constructed pavements.

Regardless of the design procedure used to develop the design, the designer must specify the means and standard deviations of each of the quality characteristics and the means of the constant variables for use in the LCC predictions.

Why specify the target means of the quality characteristics to be achieved in construction? A critical point that must be understood is that these mean values are those expected to be achieved on average by the contractor for each pavement lot in order to receive 100 percent pay. These means are used in the LCC_{des} calculations. These mean and standard deviation values define the target values that the contractor must attain to achieve 100 percent pay. Of course, there can be trade-offs between these quality characteristics that will still result in 100 percent pay, but these target values are the clearly defined goal for the contractor to achieve for each quality characteristic.

Why include the standard deviation of each quality characteristic? The standard deviations are values used in the as-designed LCC calculations and represent acceptable quality levels for which the agency is willing to pay 100 percent. These standard deviations are point-to-point variations (including testing errors) for strength, thickness, and air content in the slab, and variations between longitudinal profiles for initial roughness.

In the pavement design process, the designer selects various inputs to develop the pavement design. When using the 1986 AASHTO Guide, mean values of all inputs are required by the procedure, including concrete strength, slab support, traffic, initial serviceability, and so on.[5] The Guide also requires an overall standard deviation, which consists of an estimate of the uncertainty in the prediction of future traffic loadings and the error associated with the prediction of performance. The variation of quality characteristics, such as slab thickness, concrete strength, and initial serviceability (due to initial roughness) are included in the estimation of the overall standard deviation. The design output is the mean slab thickness required for a selected level of design reliability to be obtained during construction. The mean concrete strength and initial serviceability (or roughness) is expected to be obtained during construction to achieve the given level of design reliability.

Thus, when using the AASHTO Guide for pavement design, the mean values used for inputs and the output thickness are specified in the PRS as the mean quality characteristics for the as-designed pavement. It would be adding even more conservatism to the design by stipulating that, say, 90 percent of the concrete strength or the slab thickness attained during construction be above the mean value. Such a requirement is not what the designer assumed in the design process, and would add considerable cost to construction. In other words, the resulting design thickness value obtained at, say, a 95-percent reliability level already has variability built into it. Thus, the mean values should be specified as the target values to be achieved by the contractor in order to obtain full pay.

What about pavements that are designed using other procedures where it is not clear whether mean or conservative input values should be used? This is an area of concern, especially since it is not clear for some design procedures what type of safety factors have been included in the design thickness. In this case, the designer may want to stipulate that, say, 90 percent of the design thickness or strength is above a given level. In this case, the designer must estimate the target means and standard deviations of each quality characteristic required in the PRS. For example, if a 254-mm (10-in) thick pavement is obtained from a given design method and the designer wants, say, 90 percent of the pavement to be greater than 254 mm (10 in), the designer must specify a mean target thickness of something greater than 254 mm (10 in) for use in the PRS. This can be accomplished by applying statistical theory to determine the appropriate target mean value. While the same concept can be applied to strength, the designer usually specifies mean target values for entrained air or initial roughness, which are the values required in the prototype PRS.

The above discussion does not preclude the likely desire of the contractor to provide quality characteristics that have somewhat higher quality levels than the targets. This would provide some level of confidence for the contractor to achieve at least 100 percent pay, and may very well provide additional value during construction that would warrant an incentive. In fact, the ability to achieve an added bonus for exceptional quality of construction should give significant incentive to the contractor.

As-Constructed Pavement

The as-constructed pavement lot is defined as that pavement actually constructed by the contractor. The lot is divided into sublots having approximately equal surface areas. The minimum length of a sublot is 0.16 km (0.1 mi) to allow for the measurement of pavement roughness. A minimum of three sublots per lot is required to apply the PRS acceptance plan. Each sublot is sampled *in situ* for thickness, strength, air content, and roughness after the concrete slab has been placed.

Air content can be measured in each sublot in the plastic concrete behind the paver or in the hardened concrete through a linear traverse of cores after the slab has sufficiently cured. Thickness and strength (either compressive or splitting tensile) can be determined in each sublot from cores. The agency can set a minimum time for strength coring, such as 72 h, so as to provide a more rapid turnaround of strength information to the contractor. Various adjustments are made to the compressive or splitting tensile strength from the core to convert it to an equivalent 28-d, third-point loading flexural strength value for use in the PRS prediction models.

Adjustments for curing are made using the maturity concept, whereas adjustments for converting from compressive or splitting tensile strength to flexural strength are made using laboratory-derived correlations developed for the specific project materials. Procedures to make these conversions are provided in chapter 4.

Roughness is measured over the entire length of each sublot and prorated to a standard unit of m/km (in/mi). A minimum of two tests per sublot is generally recommended.

Consideration of Within-Lot Variation

Variation exists within the lot for each of the quality characteristics that are being measured. The effect of this variation is considered during the estimation of the LCC for the lot. The lot is divided into sublots of approximately equal area and two or more samples of each quality characteristic are taken and averaged for each sublot. The mean sample values for air content, thickness, strength, and roughness from each sublot are input into predictive models to estimate key distress indicators (e.g., spalling, cracking, faulting, and serviceability) over the design analysis period. Then, the rehabilitation policy is applied and the future costs are estimated. The present worth LCC is calculated for each sublot. This process is performed for each pavement sublot, and is performed for each year over the design analysis period. The percent defective sublots based on the mean and standard deviation of the as-constructed sublots is then computed. The percent defective is then used to determine the pay factor.

The PRS acceptance procedure outlined above considers both the mean and standard deviation of each of the significant characteristics in ultimately determining the pay factor for a lot. Thus, the contractor must be concerned with both the mean and variance of any quality characteristic, since both will affect the pay factor. The capability of considering variability is extremely important and represents a major improvement in the determination of an overall pay factor.

Rehabilitation Policy

The rehabilitation policy used to calculate both the LCC_{des} and LCC_{con} must be specified by the agency. Three options are included in the prototype PRS, with the potential for many others to be added.

- Policy (A). Individual sublots are independently rehabilitated annually through full-depth repairs for joint spalling and slab replacements for slab cracking. Hot-mix asphalt concrete (AC) overlays are assumed to be placed when any of the distress indicators exceed a user-defined trigger value (e.g., cracking exceeds 42 deteriorated transverse cracks/km [67/mi], PSR value is less than 3.0, or joint spalling exceeds 47 deteriorated joints/km [75/mi]). In other words, each sublot is rehabilitated as an independent section of pavement.

- Policy (B). Individual sublots are independently rehabilitated annually through full-depth repairs of joint spalling and slab replacements of slab cracking. However, the entire lot is overlaid with an AC overlay when the overall lot reaches a critical level of distress (based on user-defined trigger values).

- Policy (C). Individual sublots are independently rehabilitated annually through full-depth repairs of joint spalling and slab replacements of slab cracking. However, the entire lot is overlaid with an AC overlay when a user-defined percentage of sublots (i.e., 15 percent) has developed a pre-selected amount of distress.

Rehabilitation policies B and C are believed to be the most realistic in terms of representing what actually happens over a long section of pavement.

DIVISION OF RESPONSIBILITY

The contractor controls the construction process and the agency is responsible for judging the acceptability of the completed work. This philosophy is in keeping with the end-result specification concept. Figure 1 shows the overall concept of a PRS for concrete pavement construction in terms of the general testing responsibilities and the acceptance procedures.

ACCEPTANCE PLAN

The acceptance plan, a critical part of the PRS, defines the methods for taking measurements for the purpose of determining the acceptability and pay adjustment of the lot. The acceptance plan uses a variable sampling plan in which the variability of the measured values of the quality characteristics is considered. Acceptance (and the subsequent pay adjustment) is based on the estimated lot percent defective. The acceptance plan defines the methods of sampling, testing, measurement, computing percent defective, and making the pay adjustment. Definitions of factors included in the acceptance plan are given below.

- **Quality Characteristics:** Inherent characteristics of the pavement that significantly affect the performance of the pavement. The prototype specification developed under this work includes concrete strength, slab thickness, entrained air content, and initial roughness as quality characteristics.

- **Lot:** A discrete quantity of constructed pavement to which an acceptance procedure is applied. A lot is equal to 1 day's production or less. The lot consists of a pavement one or more traffic lanes wide (but does not include shoulders).

- **Sublot:** A portion of a lot. The lot is divided into sublots of approximately equal surface area. This specification requires that sublots are uniquely defined for all sampling in that one or more samples of all quality characteristics are taken from each defined sublot. The minimum length of a sublot is 0.16 km (0.1 mi) so that roughness can be measured.

- **As-Designed Pavement:** The pavement as defined by the engineer. The desired quality level of the pavement must be clearly defined by specifying the means and standard deviations of the quality characteristics.

PART I—GENERAL TESTING RESPONSIBILITIES

SHA — Source Approval

- Cement
- Aggregates
- Additives (air entrainment, accelerators, fly ash, etc.)
- Joint Sealant
- Batch Plant

SHA — Acceptance

- PCC strength
- PCC slab thickness
- PCC air system
- Initial smoothness
- Dowel alignment
- Trans./long. joint-forming
- Tiebar placement
- Initial surface friction
- Steel depth
- Densification of concrete surrounding the dowel bars
- Installation of joint sealant

Contractor — Quality (Process) Control

- Strength
- Thickness
- Air Content
- Slump
- Unit Weight
- Aggregate Gradation
- Aggregate Moisture Control
- Concrete Temperature

Figure 1. General division of responsibility for concrete pavement PRS.

PART II—ACCEPTANCE OF MATERIALS AND CONSTRUCTION

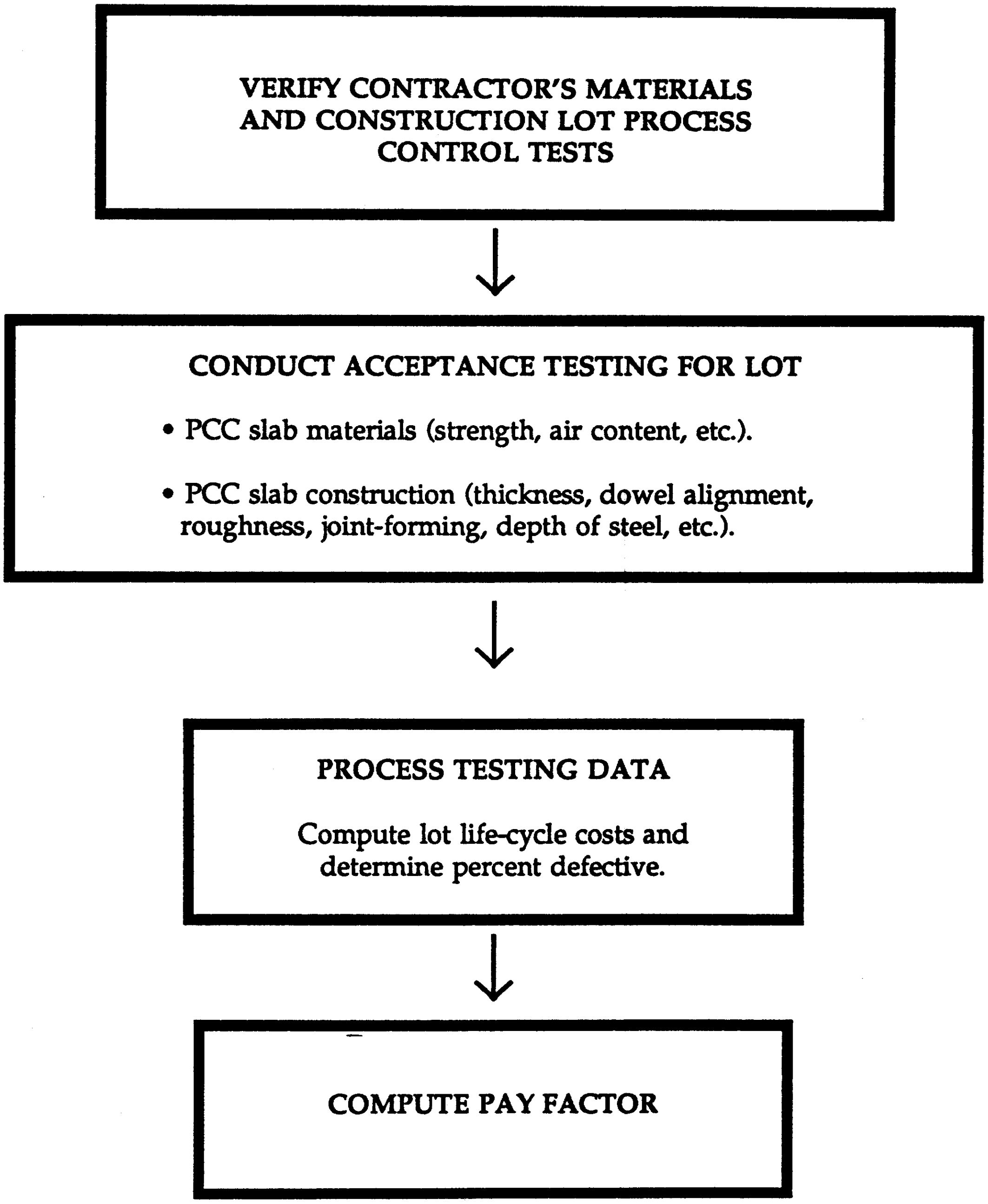


Figure 1. General division of responsibility for concrete pavement PRS (continued).

- **As-Constructed Pavement:** The *in situ* concrete pavement lot as constructed by the contractor.

- **Life-Cycle Cost (LCC):** The total cost of a lot over the pavement's analysis period. LCC in this specification consists of the estimated future rehabilitation costs over the analysis period and is expressed in terms of present worth through the use of a specified discount rate. The initial construction cost is not included in the LCC since it is identical for both the as-designed and the as-constructed pavements.

- **Percent Defective (PD):** The percentage of the lot falling above the mean target as-designed life-cycle cost (LCC_{des}) value.

- **Pay Factor:** The pay adjustment is expressed in percent of the bid price that the contractor is paid for the construction of a lot of concrete pavement. It is calculated as follows:

$$\text{Pay Factor} = 100 \,(\, BID + DIFF \,) \,/\, BID \tag{1}$$

where:

BID = Contractor's bid price for the lot, \$
$DIFF$ = $LCC_{des} - LCC_{con}$
LCC_{des} = As-designed life-cycle cost for lot, \$
LCC_{con} = As-constructed life-cycle cost for lot, \$

- **Sample size:** The sample size is defined as the number of samples (n) of LCC_{con} determined per lot. When more than one sample of the quality characteristics is taken per sublot, they are averaged within the sublot. The LCC of the sublot is determined from the mean values. With this approach, the sample size is equal to the number of sublots. The minimum number of sublots is three.

- **Stratified random sampling procedure:** Stratified random sampling procedures are used to determine locations to be used for testing. This is conducted by dividing the lot into "n" number of equal area sublots, from which one or more random samples are obtained for each quality characteristic.

- ***In situ* test methods:** Ideally, all samples are taken from the *in situ* pavement. This is believed to provide a better estimate of the actual, as-constructed quality of the pavement.

- **Computation procedures to determine the pay adjustment:** This includes computing the as-designed LCC_{des}, the as-constructed LCC for each sublot, the mean lot life-cycle cost (LCC_{con}) and standard deviation (S_{con}) for the as-constructed pavement, the lot quality index, and the lot percent defective. The pay factor is computed from the percent defective.

- **Operating characteristic curve:** A plot of the probability that a lot will be accepted (for 100 percent or better pay factor) as a function of the lot quality in terms of percent defective.

- **Retesting procedures:** Retesting provisions allow for additional testing when initial testing results are suspect. Retesting can be called for by either the agency or the contractor.

DETERMINING PAY ADJUSTMENTS

Percent Defective

The LCC is computed for the as-designed pavement through simulation using the target values (means and standard deviations) for strength, thickness, air content, and roughness. The LCC_{des} is estimated by simulating at least 100 lots. The agency uses this LCC_{des} target value to compare with the LCC computed from the as-constructed lot (LCC_{con}).

The percent defective of the as-constructed lot is defined as the proportion of the lot having an LCC_{con} greater than the LCC_{des}. The percent defective is calculated as follows:

$$Q = (LCC_{des} - LCC_{con}) / S_{con} \tag{2}$$

where:

LCC_{des}	=	As-designed target life-cycle cost, $
LCC_{con}	=	As-constructed life-cycle cost of lot, $
S_{con}	=	Standard deviation of LCC between as-constructed sublots, $

The percent defective as-constructed LCC can be determined using Q and "n" in standard tables for estimating lot percent defective, such as those found in table C of AASHTO R 9.[8] Table 3 contains those standard tables reproduced from AASHTO R 9.[8] The sample size (n) for use in the table is equal to the number of sublots.

Pay Adjustment (Pay Factor)

The contractor is paid based upon the achieved quality of the as-constructed pavement lot. The payment to the contractor is adjusted when the constructed pavement lot quality level varies from the as-designed pavement quality level. When the contractor constructs a lot that has a LCC_{con} that exactly equals the LCC_{des}, there is no need for a pay adjustment and 100 percent of the lot bid price will be paid to the contractor. When there is a difference between these values, the contractor's bid price will be adjusted according to the difference. That is, if LCC_{con} is less than LCC_{des}, there is a positive pay adjustment (incentive); if LCC_{con} is greater than LCC_{des}, there is a negative pay adjustment (disincentive).

Table 3. Estimation of lot percent defective based on quality index and sample size.[8]

Variability—Unknown Procedure | **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
0.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00	50.00
0.01	49.72	49.67	49.64	49.63	49.63	49.62	49.62	49.62	49.61	49.61	49.60	49.60	49.60
0.02	49.45	49.33	49.29	49.27	49.25	49.24	49.24	49.23	49.22	49.21	49.21	49.21	49.20
0.03	49.17	49.00	48.93	48.90	48.88	48.86	48.85	48.85	48.83	48.82	48.81	48.81	48.81
0.04	48.90	48.67	48.58	48.53	48.50	48.49	48.47	48.46	48.44	48.43	48.42	48.41	48.41
0.05	48.62	48.33	48.22	48.16	48.13	48.11	48.09	48.08	48.05	48.04	48.02	48.02	48.01
0.06	48.35	48.00	47.86	47.80	47.75	47.73	47.71	47.70	47.66	47.64	47.63	47.62	47.61
0.07	48.07	47.67	47.51	47.43	47.38	47.35	47.33	47.31	47.27	47.25	47.24	47.22	47.22
0.08	47.79	47.33	47.15	47.06	47.01	46.97	46.95	46.93	46.88	46.86	46.84	46.83	46.82
0.09	47.52	47.00	46.80	46.70	46.63	46.59	46.57	46.54	46.49	46.47	46.45	46.43	46.42
0.10	47.24	46.67	46.44	46.33	46.26	46.22	46.18	46.16	46.10	46.08	46.05	46.04	46.03
0.11	46.96	46.33	46.09	45.96	45.89	45.84	45.80	45.78	45.71	45.69	45.66	45.64	45.63
0.12	46.69	46.00	45.73	45.60	45.51	45.46	45.42	45.60	45.33	45.29	45.27	45.25	45.24
0.13	46.41	45.67	45.38	45.23	45.14	45.08	45.04	45.01	44.94	44.90	44.88	44.86	44.84
0.14	46.13	45.33	45.02	44.86	44.77	44.71	44.66	44.63	44.55	44.51	44.48	44.46	44.45
0.15	45.85	45.00	44.67	44.50	44.40	44.33	44.29	44.25	44.16	44.13	44.09	44.07	44.05
0.16	45.58	44.67	44.31	44.13	44.03	43.96	43.91	43.87	43.78	43.74	43.70	43.68	43.66
0.17	45.30	44.33	43.96	43.77	43.65	43.58	43.53	43.49	43.39	43.35	43.31	43.29	43.27
0.18	45.02	44.00	43.60	43.40	43.28	43.21	43.15	43.11	43.01	42.96	42.92	42.89	42.88
0.19	44.74	43.67	43.25	43.04	42.91	42.83	42.77	42.73	42.62	42.57	42.53	42.50	42.48
0.20	44.46	43.33	42.90	42.60	42.54	42.46	42.40	42.35	42.24	42.19	42.15	42.11	42.09
0.21	44.18	43.00	42.54	42.31	42.17	42.08	42.02	41.97	41.85	41.80	41.76	41.73	41.70
0.22	43.90	42.67	42.19	41.95	41.80	41.71	41.64	41.60	41.47	41.42	41.37	41.34	41.31
0.23	43.62	42.33	41.84	41.59	41.44	41.34	41.27	41.22	41.09	41.03	40.98	40.95	40.93
0.24	43.34	42.00	41.48	41.22	41.07	40.97	40.89	40.84	40.71	40.65	40.60	40.56	40.54
0.25	43.05	41.67	41.13	40.86	40.70	40.59	40.52	40.47	40.33	40.27	40.22	40.18	40.15
0.26	42.77	41.33	40.78	40.50	40.33	40.22	40.15	40.09	39.95	39.89	39.82	39.79	39.77
0.27	42.49	41.00	40.43	40.14	39.97	39.85	39.77	39.72	39.57	39.50	39.45	39.41	39.38
0.28	42.20	40.67	40.08	39.78	39.60	39.48	39.40	39.34	39.19	39.12	39.07	39.03	39.00
0.29	41.92	40.33	39.72	39.42	39.23	39.11	39.03	38.97	38.81	38.75	38.69	38.65	38.62
0.30	41.63	40.00	39.37	39.06	38.87	38.75	38.66	38.60	38.44	38.37	38.31	38.26	38.24
0.31	41.35	39.67	39.02	38.70	38.50	38.38	38.29	38.23	38.06	37.99	37.93	37.89	37.86
0.32	41.06	39.33	38.67	38.34	38.14	38.01	37.92	37.86	37.69	37.61	37.55	37.51	37.48
0.33	40.77	39.00	38.32	37.98	37.78	37.65	37.55	37.49	37.31	37.24	37.18	37.13	37.10
0.34	40.49	38.67	37.97	37.62	37.42	37.28	37.19	37.12	36.94	36.87	36.80	36.75	36.72
0.35	40.20	38.33	37.62	37.27	37.05	36.92	36.82	36.75	34.57	36.49	36.43	36.38	36.35
0.36	39.91	38.00	37.28	36.91	36.69	36.55	36.46	36.38	36.20	36.12	36.05	36.01	35.97
0.37	39.62	37.67	36.93	36.55	36.33	36.19	36.09	36.02	35.83	35.75	35.68	35.63	35.60
0.38	39.33	37.33	36.58	36.20	35.98	35.83	35.73	35.65	35.46	35.38	35.31	35.26	35.23
0.39	39.03	37.00	36.23	35.84	35.62	35.47	35.37	35.29	35.10	35.01	34.94	34.89	34.86
0.40	36.74	36.67	35.88	35.49	35.26	35.11	35.00	34.93	34.73	34.65	34.58	34.52	34.49
0.41	38.45	36.33	35.54	35.14	34.90	34.75	34.64	34.57	34.37	34.28	34.21	34.16	34.12
0.42	38.15	36.00	35.19	34.79	34.55	34.39	34.29	34.21	34.00	33.92	33.85	33.79	33.76
0.43	37.85	35.67	34.85	34.43	34.19	34.04	33.93	33.85	33.64	33.56	33.48	33.43	33.39
0.44	37.56	35.33	34.50	34.08	33.84	33.68	33.57	33.49	33.28	33.20	33.12	33.07	33.03
0.45	37.26	35.00	34.16	33.73	33.49	33.33	33.21	33.13	32.92	32.84	32.76	32.71	32.69
0.46	36.96	34.67	33.81	33.38	33.13	32.97	32.86	32.78	32.57	32.48	32.40	32.35	32.31
0.47	36.66	34.33	33.47	33.04	32.78	32.62	32.51	32.42	32.21	32.12	32.04	31.99	31.95

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure | **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
0.48	36.35	34.00	33.12	32.69	32.43	32.27	32.15	• 32.07	31.85	31.77	31.69	31.63	31.60
0.49	36.05	33.67	32.78	32.34	32.08	31.92	31.80	31.72	31.50	31.41	31.33	31.28	31.24
0.50	35.75	33.33	32.44	32.00	31.74	31.57	31.45	31.37	31.15	31.06	30.98	30.93	30.89
0.51	35.44	33.00	32.10	31.65	31.39	31.22	31.10	31.02	30.00	30.71	30.63	30.57	30.54
0.52	35.13	32.67	31.76	31.31	31.04	30.87	30.76	30.67	30.45	30.34	30.28	30.23	30.19
0.53	34.82	32.33	31.42	30.96	30.70	30.53	30.41	30.32	30.10	30.01	29.93	29.88	29.84
0.54	34.51	32.00	31.08	30.62	30.36	30.18	30.07	29.90	29.76	29.67	29.59	29.53	29.49
0.55	34.20	31.67	30.74	30.28	30.01	29.84	29.72	29.64	29.41	29.32	29.24	29.19	29.15
0.56	33.88	31.33	30.40	29.94	29.67	29.50	29.38	29.29	29.07	28.98	28.90	28.85	28.81
0.57	33.57	31.00	30.06	29.60	29.33	29.16	29.04	28.95	28.73	26.64	28.56	26.51	28.47
0.58	33.25	30.67	29.73	29.26	28.99	28.82	28.70	28.61	28.39	28.30	28.22	28.17	28.13
0.59	32.93	30.33	29.39	28.93	28.66	28.48	28.36	28.28	28.05	27.96	27.89	27.83	27.79
0.60	32.61	30.00	29.05	28.59	28.32	28.15	28.03	27.94	27.72	27.63	27.55	27.50	27.64
0.61	32.28	29.67	28.72	28.25	27.98	27.81	27.69	27.60	27.38	27.30	27.22	27.16	27.13
0.62	31.96	29.33	28.39	27.92	27.65	27.48	27.36	27.27	27.05	26.96	26.89	26.83	26.80
0.63	31.63	29.00	28.05	27.59	27.32	27.15	29.03	26.94	26.72	26.63	26.56	26.50	26.47
0.64	31.30	28.67	27.72	27.26	26.99	26.82	26.70	26.61	26.39	26.31	26.23	26.18	26.14
0.65	30.97	28.13	27.39	26.92	26.66	26.49	26.37	26.28	26.07	25.98	25.90	25.05	25.82
0.66	30.63	28.00	27.06	26.60	26.33	26.16	26.04	25.96	25.74	25.66	25.59	25.53	25.49
0.67	30.30	27.67	26.73	26.27	26.00	25.83	25.72	25.63	25.42	25.33	25.26	25.21	25.17
0.68	29.96	27.33	26.40	25.94	25.68	25.51	25.39	25.31	25.10	25.01	24.94	24.89	24.86
0.69	29.61	27.00	26.07	25.61	25.35	25.19	25.07	24.99	24.78	24.69	24.62	24.57	24.54
0.70	29.27	26.67	25.74	25.29	25.03	24.86	24.75	24.67	21.64	24.38	24.31	24.26	24.23
0.71	28.92	26.33	25.41	24.96	24.71	24.54	24.43	24.35	24.15	24.06	23.99	23.95	23.91
0.72	28.57	26.00	25.09	24.64	24.39	24.23	24.11	24.03	23.83	23.75	23.68	23.64	23.60
0.73	28.22	25.67	24.76	24.32	24.07	23.91	23.80	23.72	23.52	23.44	23.37	23.33	23.30
0.74	27.86	25.33	24.44	24.00	23.75	23.59	23.49	23.41	23.21	23.13	23.07	23.02	22.99
0.75	27.50	25.00	24.11	23.68	23.44	23.28	23.17	23.10	22.90	22.83	22.76	22.72	22.49
0.76	27.13	24.67	23.79	23.37	23.12	22.97	22.86	22.79	22.60	22.52	22.46	22.42	22.39
0.77	26.76	24.33	23.47	23.05	22.81	22.66	22.56	22.48	22.30	22.22	22.16	22.12	22.09
0.78	26.39	24.00	23.15	22.74	22.50	22.35	22.25	22.18	21.99	21.92	21.86	21.82	21.79
0.79	26.02	23.67	22.83	22.42	22.19	22.04	21.94	21.87	21.70	21.62	21.57	21.53	21.50
0.80	25.64	23.33	22.51	22.11	21.88	21.74	21.46	21.57	21.40	21.33	21.27	21.23	21.21
0.81	25.25	23.00	22.19	21.80	21.58	21.44	21.34	21.27	21.10	21.04	20.98	20.94	20.92
0.82	24.86	22.67	21.87	21.49	21.27	21.14	21.04	20.98	20.81	20.75	20.69	20.66	20.63
0.83	24.47	22.33	21.56	21.18	20.97	20.84	20.75	20.68	20.52	20.46	20.40	20.37	20.35
0.84	24.07	22.00	21.24	20.88	20.67	20.54	20.45	20.39	20.23	20.17	20.12	20.09	20.04
0.85	23.67	21.67	20.93	20.57	20.37	20.24	20.16	20.10	19.94	19.88	19.64	19.80	19.78
0.86	23.26	21.33	20.62	20.27	20.07	19.95	19.87	19.81	19.64	19.60	19.56	19.53	19.51
0.87	22.84	21.00	20.31	19.97	19.78	19.64	19.58	19.52	19.38	19.32	19.28	19.25	19.23
0.88	22.42	20.67	20.00	19.67	19.48	19.37	19.29	19.23	19.10	19.04	19.00	18.98	18.96
0.89	21.99	20.33	19.69	19.37	19.19	19.08	19.00	18.95	18.82	18.77	18.73	18.70	18.69
0.90	21.55	20.00	19.38	19.07	18.90	18.79	18.72	18.67	18.54	18.50	18.46	18.43	18.42
0.91	21.11	19.67	19.07	18.78	18.61	18.54	18.44	18.39	18.27	18.23	18.19	18.17	18.15
0.92	20.66	19.33	18.77	18.49	18.33	18.23	18.16	18.11	18.00	17.96	17.92	17.90	17.89
0.93	20.19	19.00	18.46	18.19	18.04	17.95	17.80	17.84	17.73	17.69	17.66	17.64	17.63
0.94	19.73	18.67	18.16	17.90	17.76	17.67	17.61	17.56	17.46	17.43	17.40	17.38	17.37
0.95	19.25	18.33	17.86	17.61	17.48	17.39	17.33	17.29	17.20	17.16	17.14	17.12	17.11
0.96	18.75	18.00	17.55	17.33	17.20	17.12	17.06	17.03	16.94	16.90	16.88	16.87	16.86
0.97	18.25	17.67	17.25	17.04	16.92	16.57	16.79	16.76	16.68	16.65	16.63	16.61	16.61
0.98	17.74	17.33	16.96	16.76	16.65	16.57	16.53	16.49	16.42	16.39	16.37	16.36	16.36
0.99	17.21	17.00	16.66	16.48	16.37	16.31	16.26	16.23	16.16	16.14	16.12	16.11	16.11

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure | **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
1.00	16.67	16.67	16.36	16.20	16.10	16.04	16.00	15.97	15.91	15.89	15.88	15.87	15.87
1.01	16.11	16.33	16.07	15.92	15.83	15.78	15.74	15.72	15.66	15.64	15.63	15.63	15.62
1.02	15.53	16.00	15.78	15.64	15.56	15.51	15.48	15.46	15.41	15.40	15.39	15.39	15.38
1.03	14.93	15.67	15.48	15.37	15.30	15.25	15.23	15.21	15.17	15.15	15.15	15.15	15.15
1.04	14.31	15.33	15.19	15.09	15.03	15.00	14.97	14.96	14.92	14.91	14.91	14.91	14.91
1.05	13.66	15.00	14.91	14.82	14.77	14.74	14.72	14.71	14.68	14.67	14.67	14.68	14.68
1.06	12.98	14.67	14.62	14.55	14.51	14.49	14.47	14.46	14.44	14.44	14.44	14.45	14.45
1.07	12.27	14.33	14.33	14.29	14.26	14.24	14.21	14.22	14.20	14.20	14.21	14.22	14.22
1.08	11.51	14.00	14.05	14.02	14.00	13.99	13.98	13.97	13.97	13.97	13.98	13.99	14.00
1.09	10.71	13.67	13.76	13.76	13.75	13.74	13.73	13.73	13.74	13.74	13.75	13.77	13.77
1.10	9.84	13.33	13.48	13.50	13.49	13.49	13.49	13.50	13.51	13.52	13.53	13.54	13.55
1.11	8.89	13.00	13.20	13.24	13.25	13.25	13.26	13.26	13.28	13.29	13.31	13.32	13.34
1.12	7.82	12.67	12.93	12.98	13.00	13.01	13.02	13.03	13.05	13.07	13.09	13.11	13.12
1.13	6.60	12.33	12.65	12.72	12.75	12.77	12.79	12.80	12.83	12.85	12.87	12.89	12.91
1.14	5.08	12.00	12.37	12.47	12.51	12.54	12.55	12.57	12.61	12.63	12.66	12.68	12.70
1.15	2.87	11.67	12.10	12.22	12.27	12.30	12.32	12.34	12.39	12.42	12.45	12.47	12.49
1.16	0.00	11.33	11.83	11.97	12.03	12.07	12.10	12.12	12.18	12.21	12.24	12.26	12.28
1.17	0.00	11.00	11.56	11.72	11.79	11.84	11.87	11.90	11.96	12.00	12.03	12.06	12.08
1.18	0.00	10.67	11.29	11.47	11.56	11.61	11.65	11.68	11.75	11.79	11.82	11.85	11.88
1.19	0.00	10.33	11.02	11.23	11.33	11.39	11.43	11.46	11.54	11.58	11.62	11.65	11.68
1.20	0.00	10.00	10.76	10.99	11.10	11.17	11.21	11.24	11.34	11.38	11.42	11.46	11.48
1.21	0.00	9.67	10.50	10.75	10.87	10.94	10.99	11.03	11.13	11.18	11.22	11.26	11.29
1.22	0.00	9.33	10.23	10.51	10.65	10.73	10.78	10.82	10.93	10.98	11.03	11.07	11.09
1.23	0.00	9.00	9.97	10.28	10.42	10.51	10.57	10.61	10.73	10.78	10.84	10.88	10.91
1.24	0.00	8.67	9.72	10.04	10.20	10.30	10.36	10.41	10.53	10.59	10.64	10.69	10.72
1.25	0.00	8.33	9.46	9.81	9.98	10.09	10.15	10.21	10.34	10.40	10.46	10.50	10.53
1.26	0.00	8.00	9.21	9.58	9.77	9.88	9.95	10.00	10.15	10.21	10.27	10.32	10.35
1.27	0.00	7.67	8.96	9.36	9.55	9.67	9.75	9.81	9.96	10.02	10.09	10.13	10.17
1.28	0.00	7.33	8.71	9.13	9.34	9.47	9.55	9.61	9.77	9.84	9.90	9.95	9.99
1.29	0.00	7.00	8.46	8.91	9.13	9.26	9.35	9.42	9.58	9.66	9.72	9.78	9.82
1.30	0.00	6.67	8.21	8.69	8.93	9.06	9.16	9.22	9.40	9.48	9.55	9.60	9.64
1.31	0.00	6.33	7.97	8.46	8.72	8.87	8.96	9.03	9.22	9.30	9.37	9.43	9.47
1.32	0.00	6.00	7.73	8.26	8.52	8.67	8.77	8.85	9.04	9.12	9.20	9.26	9.30
1.33	0.00	5.67	7.49	8.05	8.32	8.48	8.59	8.66	8.66	8.95	9.03	9.09	9.13
1.34	0.00	5.33	7.25	7.84	8.12	8.29	8.40	8.48	8.69	8.78	8.86	8.92	8.97
1.35	0.00	5.00	7.02	7.63	7.92	8.10	8.22	8.30	8.52	8.61	6.69	8.76	8.81
1.36	0.00	4.67	6.79	7.42	7.73	7.91	8.04	8.12	8.35	8.44	8.53	8.60	8.65
1.37	0.00	4.33	6.56	7.22	7.54	7.73	7.86	7.95	8.18	8.28	8.37	8.44	8.49
1.38	0.00	4.00	6.33	7.02	7.35	7.55	7.68	7.77	8.01	8.12	8.21	8.28	8.33
1.39	0.00	3.67	6.10	6.82	7.17	7.37	7.51	7.60	7.85	7.96	8.05	8.12	8.18
1.40	0.00	3.33	5.88	6.63	6.98	7.19	7.33	7.44	7.69	7.80	7.90	7.97	8.02
1.41	0.00	3.00	5.66	6.43	6.80	7.02	7.17	7.27	7.53	7.64	7.74	7.82	7.87
1.42	0.00	2.67	5.44	6.24	6.42	6.85	7.00	7.10	7.37	7.49	7.59	7.67	7.73
1.43	0.00	2.33	5.23	6.05	6.45	6.68	6.83	6.94	7.22	7.34	7.44	7.52	7.58
1.44	0.00	2.00	5.03	5.87	6.27	6.51	6.47	6.78	7.07	7.19	7.30	7.38	7.44
1.45	0.00	1.67	4.81	5.68	4.10	6.36	6.51	6.63	6.92	7.04	7.15	7.24	7.30
1.46	0.00	1.33	4.60	5.50	5.93	6.19	6.35	6.47	6.77	6.90	7.01	7.10	7.16
1.47	0.00	1.00	4.39	5.33	5.77	6.03	6.20	6.32	6.63	6.75	6.87	6.96	7.02
1.48	0.00	0.67	4.19	5.15	5.60	5.87	6.04	6.17	6.48	6.61	6.73	6.82	6.88
1.49	0.00	0.33	3.99	4.98	5.44	5.71	5.89	6.02	6.34	6.46	6.60	6.69	6.75
1.50	0.00	0.00	3.80	4.81	5.28	5.56	5.74	5.87	6.20	6.34	6.46	6.55	6.62
1.51	0.00	0.00	3.61	4.64	5.13	5.41	5.60	5.73	6.06	6.20	6.33	6.42	6.49
1.52	0.00	0.00	3.42	4.47	4.97	5.26	5.45	5.59	5.93	6.07	6.20	6.29	6.36

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure | **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
1.53	0.00	0.00	3.23	4.31	4.82	5.12	5.31	5.45	5.80	5.94	6.07	6.17	6.24
1.54	0.00	0.00	3.05	4.15	4.67	4.97	5.17	5.31	5.67	5.81	5.95	6.04	6.11
1.55	0.00	0.00	2.87	4.00	4.52	4.83	5.03	5.18	5.54	5.69	5.82	5.92	5.99
1.56	0.00	0.00	2.69	3.84	4.38	4.69	4.90	5.05	5.41	5.56	5.70	5.80	5.87
1.57	0.00	0.00	2.52	3.69	4.24	4.56	4.77	4.92	5.29	5.44	5.58	5.68	5.75
1.58	0.00	0.00	2.35	3.54	4.10	4.42	4.64	4.79	5.16	5.32	5.46	5.56	5.64
1.59	0.00	0.00	2.19	3.40	3.96	4.29	4.51	4.66	5.04	5.20	5.34	5.45	5.52
1.60	0.00	0.00	2.03	3.25	3.83	4.16	4.38	4.54	4.92	5.08	5.23	5.33	5.41
1.61	0.00	0.00	1.87	3.11	3.69	4.03	4.26	4.41	4.81	4.97	5.12	5.22	5.30
1.62	0.00	0.00	1.72	2.97	3.57	3.91	4.14	4.30	4.69	4.86	5.01	5.11	5.19
1.63	0.00	0.00	1.57	2.84	3.44	3.79	4.02	4.18	4.58	4.75	4.90	5.01	5.08
1.64	0.00	0.00	1.42	2.71	3.31	3.67	3.90	4.06	4.47	4.64	4.79	4.90	4.98
1.65	0.00	0.00	1.28	2.58	3.19	3.35	3.78	3.95	4.36	4.53	4.68	4.79	4.87
1.66	0.00	0.00	1.15	2.45	3.07	3.43	3.67	3.84	4.25	4.43	4.58	4.69	4.77
1.67	0.00	0.00	1.02	2.33	2.95	3.32	3.56	3.73	4.15	4.32	4.48	4.59	4.67
1.68	0.00	0.00	0.89	2.21	2.84	3.21	3.45	3.62	4.05	4.22	4.38	4.49	4.57
1.69	0.00	0.00	0.77	2.09	2.73	3.10	3.34	3.52	3.94	4.12	4.28	4.39	4.47
1.70	0.00	0.00	0.66	1.98	2.62	2.99	3.24	3.41	3.84	4.02	4.18	4.30	4.38
1.71	0.00	0.00	0.55	1.87	2.51	2.89	3.14	3.31	3.75	3.93	4.09	4.20	4.29
1.72	0.00	0.00	0.45	1.76	2.41	2.79	3.03	3.21	3.65	3.83	3.99	4.11	4.19
1.73	0.00	0.00	0.36	1.66	2.30	2.69	2.94	3.11	3.56	3.74	3.90	4.02	4.10
1.74	0.00	0.00	0.27	1.55	2.20	2.59	2.84	3.02	3.46	3.65	3.81	3.93	4.01
1.75	0.00	0.00	0.19	1.45	2.11	2.49	2.75	2.93	3.37	3.56	3.72	3.84	3.93
1.76	0.00	0.00	0.12	1.36	2.01	2.40	2.65	2.83	3.28	3.47	3.63	3.76	3.84
1.77	0.00	0.00	0.06	1.27	1.92	2.31	2.56	2.74	3.20	3.38	3.55	3.67	3.76
1.78	0.00	0.00	0.02	1.18	1.83	2.22	2.47	2.66	3.11	3.30	3.47	3.59	3.67
1.79	0.00	0.00	0.00	1.09	1.74	2.13	2.39	2.57	3.03	3.21	3.38	3.51	3.59
1.80	0.00	0.00	0.00	1.01	1.65	2.04	2.30	2.49	2.94	3.13	3.30	3.43	3.51
1.81	0.00	0.00	0.00	0.93	1.57	1.96	2.22	2.40	2.86	3.05	3.22	3.35	3.43
1.82	0.00	0.00	0.00	0.85	1.49	1.85	2.14	2.32	2.79	2.98	3.15	3.27	3.36
1.83	0.00	0.00	0.00	0.78	1.41	1.80	2.06	2.25	2.71	2.90	3.07	3.19	3.28
1.84	0.00	0.00	0.00	0.71	1.34	1.72	1.98	2.17	2.63	2.82	2.99	3.12	3.21
1.85	0.00	0.00	0.00	0.64	1.26	1.65	1.91	2.09	2.56	2.75	2.92	3.05	3.13
1.86	0.00	0.00	0.00	0.57	1.19	1.58	1.84	2.02	2.48	2.68	2.85	2.97	3.06
1.87	0.00	0.00	0.00	0.51	1.12	1.51	1.76	1.95	2.41	2.61	2.78	2.90	2.99
1.88	0.00	0.00	0.00	0.46	1.06	1.44	1.70	1.88	2.34	2.54	2.71	2.83	2.92
1.89	0.00	0.00	0.00	0.40	0.99	1.37	1.63	1.81	2.28	2.47	2.64	2.77	2.85
1.90	0.00	0.00	0.00	0.35	0.93	1.31	1.56	1.75	2.21	2.40	2.57	2.70	2.79
1.91	0.00	0.00	0.00	0.30	0.87	1.24	1.50	1.68	2.14	2.34	2.51	2.63	2.72
1.92	0.00	0.00	0.00	0.26	0.81	1.18	1.44	1.62	2.08	2.27	2.45	2.57	2.66
1.93	0.00	0.00	0.00	0.22	0.76	1.12	1.37	1.56	2.02	2.21	2.38	2.51	2.60
1.94	0.00	0.00	0.00	0.18	0.70	1.07	1.32	1.50	1.96	2.15	2.32	2.45	2.54
1.95	0.00	0.00	0.00	0.15	0.65	1.01	1.26	1.44	1.90	2.09	2.26	2.39	2.48
1.96	0.00	0.00	0.00	0.12	0.60	0.96	1.20	1.38	1.84	2.03	2.20	2.33	2.42
1.97	0.00	0.00	0.00	0.09	0.56	0.91	1.15	1.33	1.78	1.97	2.15	2.27	2.36
1.98	0.00	0.00	0.00	0.07	0.51	0.86	1.10	1.27	1.73	1.92	2.09	2.21	2.30
1.99	0.00	0.00	0.00	0.05	0.47	0.81	1.05	1.22	1.67	1.86	2.03	2.16	2.25
2.00	0.00	0.00	0.00	0.03	0.43	0.76	1.00	1.17	1.62	1.81	1.98	2.10	2.19
2.01	0.00	0.00	0.00	0.02	0.39	0.72	0.95	1.12	1.57	1.76	1.93	2.05	2.14
2.02	0.00	0.00	0.00	0.01	0.36	0.67	0.90	1.07	1.52	1.71	1.87	2.00	2.09
2.03	0.00	0.00	0.00	0.00	0.32	0.63	0.86	1.03	1.47	1.66	1.82	1.95	2.04
2.04	0.00	0.00	0.00	0.00	0.29	0.59	0.82	0.98	1.42	1.61	1.77	1.90	1.99
2.05	0.00	0.00	0.00	0.00	0.26	0.55	0.77	0.94	1.37	1.56	1.73	1.85	1.94

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure | **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
2.06	0.00	0.00	0.00	0.00	0.23	0.52	0.73	0.90	1.33	1.51	1.68	1.80	1.89
2.07	0.00	0.00	0.00	0.00	0.21	0.48	0.70	0.86	1.28	1.47	1.63	1.76	1.84
2.08	0.00	0.00	0.00	0.00	0.18	0.45	0.66	0.82	1.24	1.42	1.59	1.71	1.79
2.09	0.00	0.00	0.00	0.00	0.16	0.42	0.62	0.78	1.20	1.38	1.54	1.66	1.75
2.10	0.00	0.00	0.00	0.00	0.14	0.39	0.59	0.74	1.16	1.34	1.50	1.62	1.71
2.11	0.00	0.00	0.00	0.00	0.12	0.36	0.55	0.71	1.12	1.30	1.46	1.58	1.66
2.12	0.00	0.00	0.00	0.00	0.10	0.33	0.52	0.67	1.08	1.26	1.42	1.54	1.62
2.13	0.00	0.00	0.00	0.00	0.08	0.30	0.49	0.64	1.04	1.22	1.38	1.50	1.58
2.14	0.00	0.00	0.00	0.00	0.07	0.28	0.46	0.61	1.00	1.18	1.34	1.46	1.54
2.15	0.00	0.00	0.00	0.00	0.06	0.26	0.43	0.58	0.97	1.14	1.30	1.42	1.50
2.16	0.00	0.00	0.00	0.00	0.05	0.23	0.41	0.55	0.93	1.10	1.26	1.38	1.46
2.17	0.00	0.00	0.00	0.00	0.04	0.21	0.38	0.52	0.90	1.07	1.22	1.34	1.42
2.18	0.00	0.00	0.00	0.00	0.03	0.19	0.36	0.49	0.87	1.03	1.19	1.30	1.39
2.19	0.00	0.00	0.00	0.00	0.02	0.17	0.33	0.46	0.83	1.00	1.15	1.27	1.35
2.20	0.00	0.00	0.00	0.00	0.01	0.16	0.31	0.44	0.80	0.97	1.12	1.23	1.31
2.21	0.00	0.00	0.00	0.00	0.01	0.14	0.29	0.41	0.77	0.94	1.09	1.20	1.28
2.22	0.00	0.00	0.00	0.00	0.01	0.13	0.27	0.39	0.74	0.90	1.05	1.17	1.25
2.23	0.00	0.00	0.00	0.00	0.00	0.11	0.25	0.37	0.71	0.87	1.02	1.13	1.21
2.24	0.00	0.00	0.00	0.00	0.00	0.10	0.23	0.34	0.69	0.85	0.99	1.10	1.18
2.25	0.00	0.00	0.00	0.00	0.00	0.09	0.21	0.32	0.66	0.82	0.96	1.07	1.15
2.26	0.00	0.00	0.00	0.00	0.00	0.08	0.20	0.30	0.63	0.79	0.93	1.04	1.12
2.27	0.00	0.00	0.00	0.00	0.00	0.07	0.18	0.29	0.61	0.76	0.90	1.01	1.09
2.28	0.00	0.00	0.00	0.00	0.00	0.06	0.17	0.27	0.58	0.74	0.88	0.98	1.06
2.29	0.00	0.00	0.00	0.00	0.00	0.05	0.15	0.25	0.56	0.71	0.85	0.95	1.03
2.30	0.00	0.00	0.00	0.00	0.00	0.04	0.14	0.23	0.54	0.68	0.82	0.93	1.00
2.31	0.00	0.00	0.00	0.00	0.00	0.04	0.13	0.22	0.52	0.66	0.80	0.90	0.97
2.32	0.00	0.00	0.00	0.00	0.00	0.03	0.11	0.20	0.49	0.64	0.77	0.87	0.95
2.33	0.00	0.00	0.00	0.00	0.00	0.02	0.10	0.19	0.47	0.61	0.75	0.85	0.92
2.34	0.00	0.00	0.00	0.00	0.00	0.02	0.09	0.18	0.45	0.59	0.72	0.82	0.90
2.35	0.00	0.00	0.00	0.00	0.00	0.02	0.08	0.16	0.43	0.57	0.70	0.80	0.87
2.36	0.00	0.00	0.00	0.00	0.00	0.01	0.08	0.15	0.42	0.55	0.68	0.78	0.85
2.37	0.00	0.00	0.00	0.00	0.00	0.01	0.07	0.14	0.40	0.53	0.66	0.75	0.82
2.38	0.00	0.00	0.00	0.00	0.00	0.01	0.06	0.13	0.38	0.51	0.63	0.73	0.80
2.39	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.12	0.34	0.49	0.61	0.71	0.78
2.40	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.11	0.35	0.47	0.50	0.69	0.75
2.41	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.10	0.38	0.45	0.57	0.67	0.73
2.42	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.09	0.32	0.44	0.56	0.65	0.71
2.43	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.30	0.42	0.54	0.63	0.69
2.44	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.29	0.40	0.52	0.61	0.67
2.45	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.07	0.27	0.39	0.50	0.59	0.65
2.46	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.26	0.37	0.48	0.57	0.63
2.47	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.25	0.36	0.47	0.55	0.62
2.48	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.05	0.24	0.34	0.45	0.54	0.60
2.49	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.05	0.23	0.33	0.44	0.52	0.58
2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.21	0.32	0.42	0.50	0.56
2.51	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.20	0.30	0.41	0.49	0.55
2.52	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.19	0.29	0.29	0.47	0.53
2.53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.18	0.28	0.38	0.46	0.51
2.54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.17	0.27	0.37	0.44	0.50
2.55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.17	0.26	0.35	0.43	0.48
2.56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.16	0.25	0.34	0.41	0.47
2.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.15	0.24	0.33	0.40	0.46
2.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.14	0.23	0.32	0.39	0.44
2.59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.13	0.22	0.30	0.38	0.43

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
2.60	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.13	0.21	0.29	0.36	0.41
2.61	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.12	0.20	0.28	0.35	0.40
2.62	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.11	0.19	0.27	0.34	0.39
2.63	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.11	0.18	0.26	0.33	0.38
2.64	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.10	0.17	0.25	0.32	0.37
2.65	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.16	0.24	0.31	0.35
2.66	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.16	0.23	0.30	0.34
2.67	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.15	0.22	0.29	0.33
2.68	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.14	0.22	0.28	0.32
2.69	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.14	0.21	0.27	0.31
2.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.13	0.20	0.26	0.30
2.71	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.12	0.19	0.25	0.29
2.72	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.12	0.18	0.24	0.28
2.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.11	0.18	0.23	0.27
2.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.11	0.17	0.22	0.27
2.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.10	0.16	0.22	0.26
2.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.10	0.16	0.21	0.25
2.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.09	0.15	0.20	0.24
2.78	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.09	0.14	0.19	0.23
2.79	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.08	0.14	0.19	0.23
2.80	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.13	0.18	0.22
2.81	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.13	0.17	0.21
2.82	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.12	0.17	0.20
2.83	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.12	0.16	0.20
2.84	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.06	0.11	0.16	0.19
2.85	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.11	0.15	0.18
2.86	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.10	0.14	0.18
2.87	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.10	0.14	0.17
2.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.09	0.13	0.17
2.89	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.09	0.13	0.16
2.90	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.09	0.12	0.16
2.91	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.08	0.12	0.15
2.92	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.08	0.12	0.14
2.93	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.08	0.11	0.14
2.94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.07	0.11	0.13
2.95	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.07	0.10	0.13
2.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.07	0.10	0.13
2.97	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.10	0.12
2.98	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.09	0.12
2.99	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.09	0.11
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.08	0.11
3.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.05	0.08	0.11
3.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.05	0.08	0.10
3.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.07	0.10
3.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.07	0.09
3.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.07	0.09
3.06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.07	0.09
3.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.06	0.08
3.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.06	0.08
3.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.06	0.08
3.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.08
3.11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.07
3.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.07

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure | *Standard Deviation Method*

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
3.13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.07
3.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.07
3.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.06
3.16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.06
3.17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.06
3.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.06
3.19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.05
3.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.05
3.21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.05
3.22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.05
3.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.05
3.24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.03	0.04
3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.03	0.04
3.26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.04
3.27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.04
3.28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.04
3.29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04
3.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04
3.31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
3.39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02
3.40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02
3.41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02
3.42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02
3.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02
3.44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02
3.45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
3.46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
3.47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
3.48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
3.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
3.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
3.51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.60	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.61	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3.62	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.63	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.64	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.65	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.66	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Table 3. Estimation of lot percent defective based on quality index and sample size (continued).[8]

Variability—Unknown Procedure | **Standard Deviation Method**

Quality Index (Q)	Estimated Lot Percent Defective for Selected Sample Sizes												
	3	4	5	6	7	8	9	10	15	20	30	50	100
3.67	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.68	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.69	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.71	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.72	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
3.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.78	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.79	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Numbers in body of table are estimates of lot percent defective corresponding to specific values of quality index and sample size. For Q values greater than or equal to zero, the percent defective estimate may be read directly from the table. For Q values less than zero, the table value must be subtracted from 100.

Rather than compute the pay factor from equation 1, the average relationship between the pay factor and the LCC_{con} percent defective is determined using a simulation technique within the PaveSpec computer program. To develop this relationship for a given project, a range of the quality characteristics were utilized so that the percent defective varied from 100 to 0. For each lot simulation run, the pay factor is calculated using equation 1 and the percent defective is calculated using the quality index (Q) and the sample size (n). Figure 2 shows an example pay factor simulation using PaveSpec for the inputs of a specific project. An equation of the following form is derived for percent defective less than 90 percent:

$$PF = A - B * PD \tag{3}$$

where:

PF = Pay factor
PD = Percent defective (less than 90 percent)
A,B = Constants to be determined by the agency for each project

When the percent defective is 90 percent or more, the relationship is modified so that the pay factor varies from that at the 90-percent defective level to a value of 50 percent at the 100 percent defective level. The following equation applies:

$$PF = (10 * A) + (900 * B) - 450 + (-0.1*A - 9*B + 5.0) * PD \tag{4}$$

where:

PD = Percent defective (90 percent or greater)
A,B = Constants to be determined by the agency for each project (these will be the same as those determined in equation 3)

The total payment to the contractor for the lot is then equal to the following:

$$\text{Payment} = BP * PF / 100 \tag{5}$$

where:

Payment = Payment to the contractor for the lot
BP = Contractor bid price for the lot

Flow Calculations for the PRS

Three figures illustrate the flow of the calculations in the PRS approach described herein. Figure 3 shows the calculation of the mean LCC_{des}, the target as-designed pavement. The mean LCC_{des} is based upon simulation of many lots using the mean target quality characteristics for the as-designed pavement. Figure 4 shows the approach for the determination of LCC_{con} for the as-constructed lot, which is based on the sampled quality characteristics from each sublot. Finally, figure 5 shows the computation of these life-cycle costs to determine the overall lot pay factor.

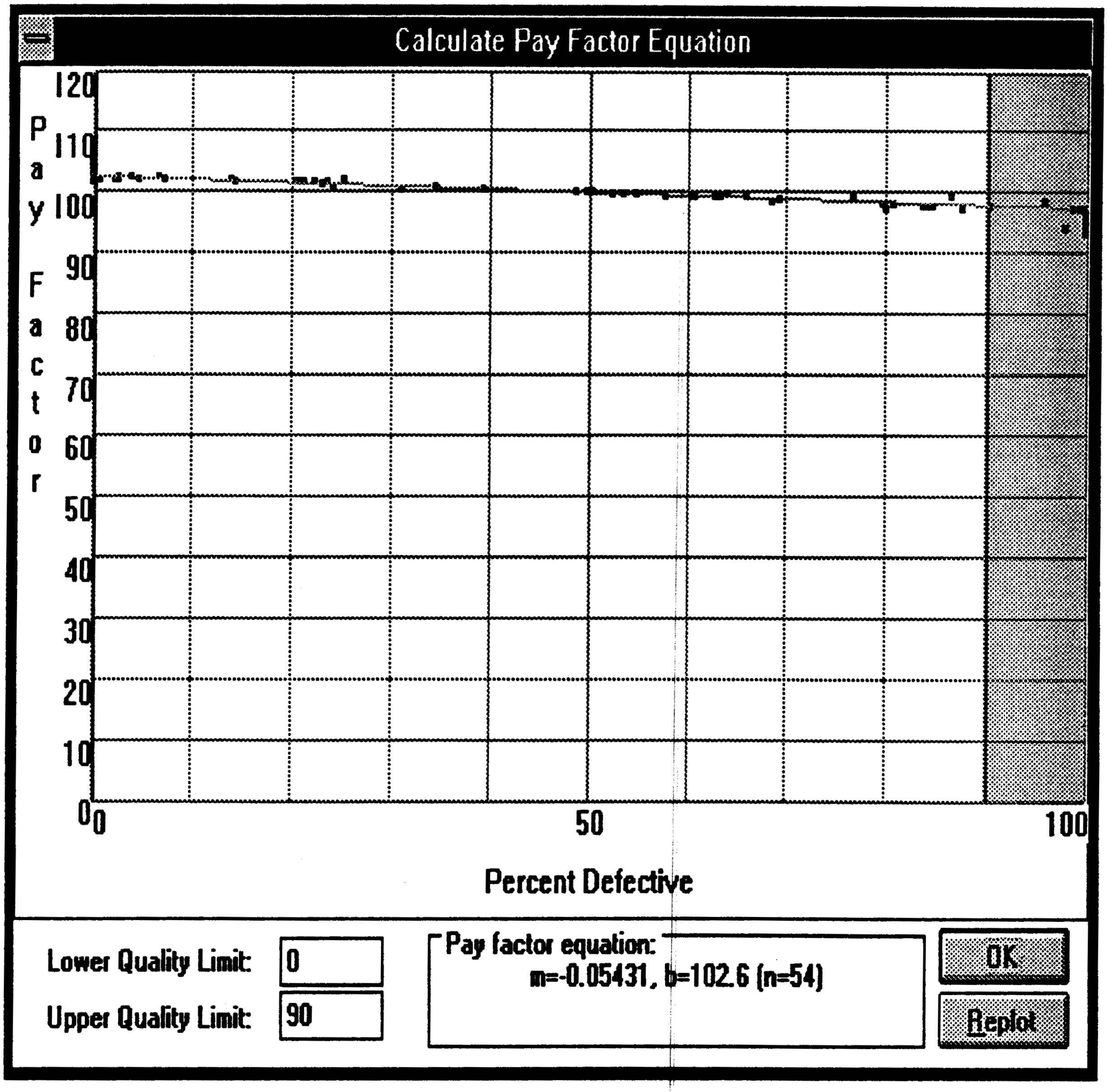


Figure 2. Example pay factor simulation using PaveSpec.

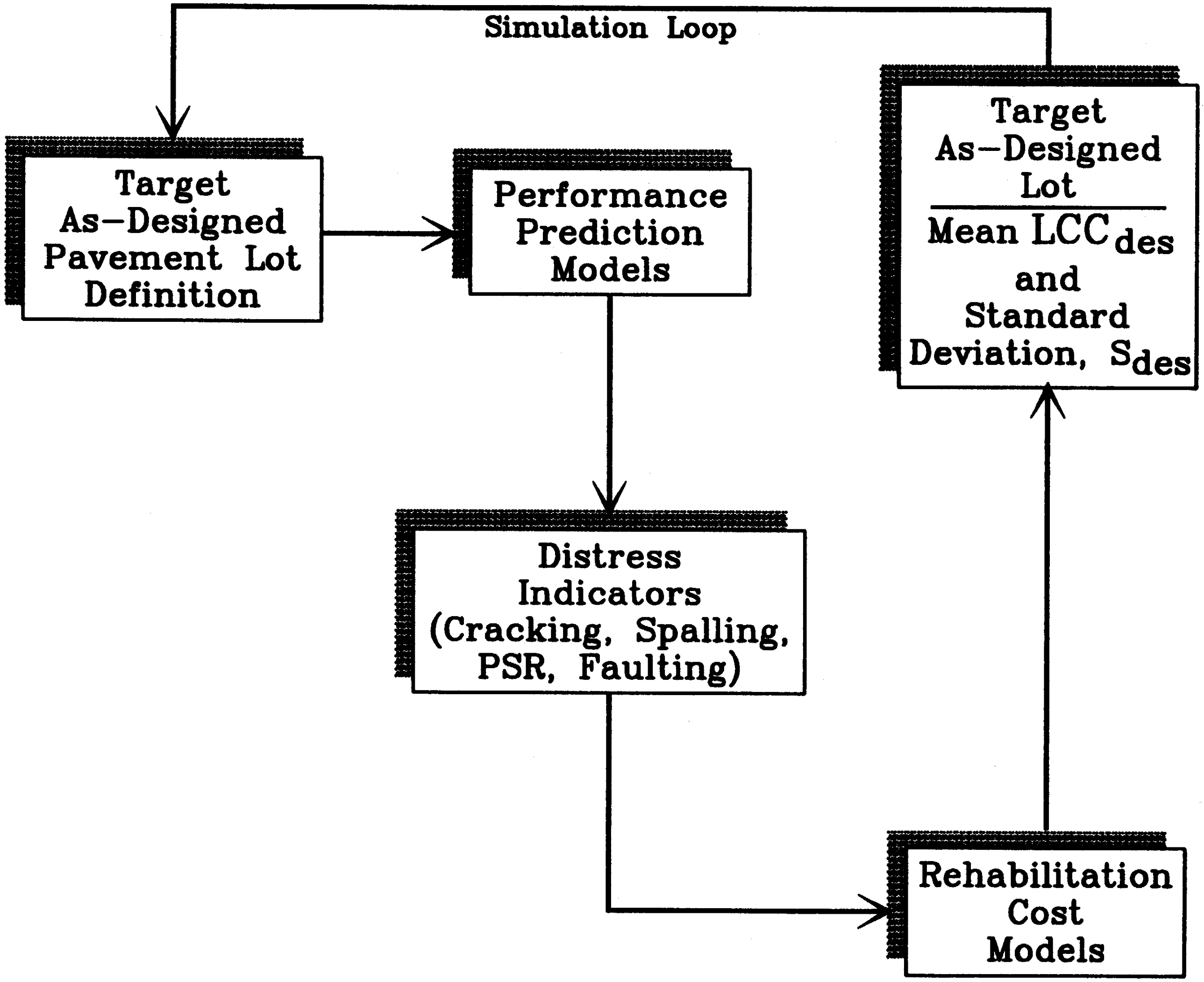


Figure 3. Procedures for estimation of as-designed life-cycle cost (mean LCC_{des} computed from simulation with target S, T, A, and R).

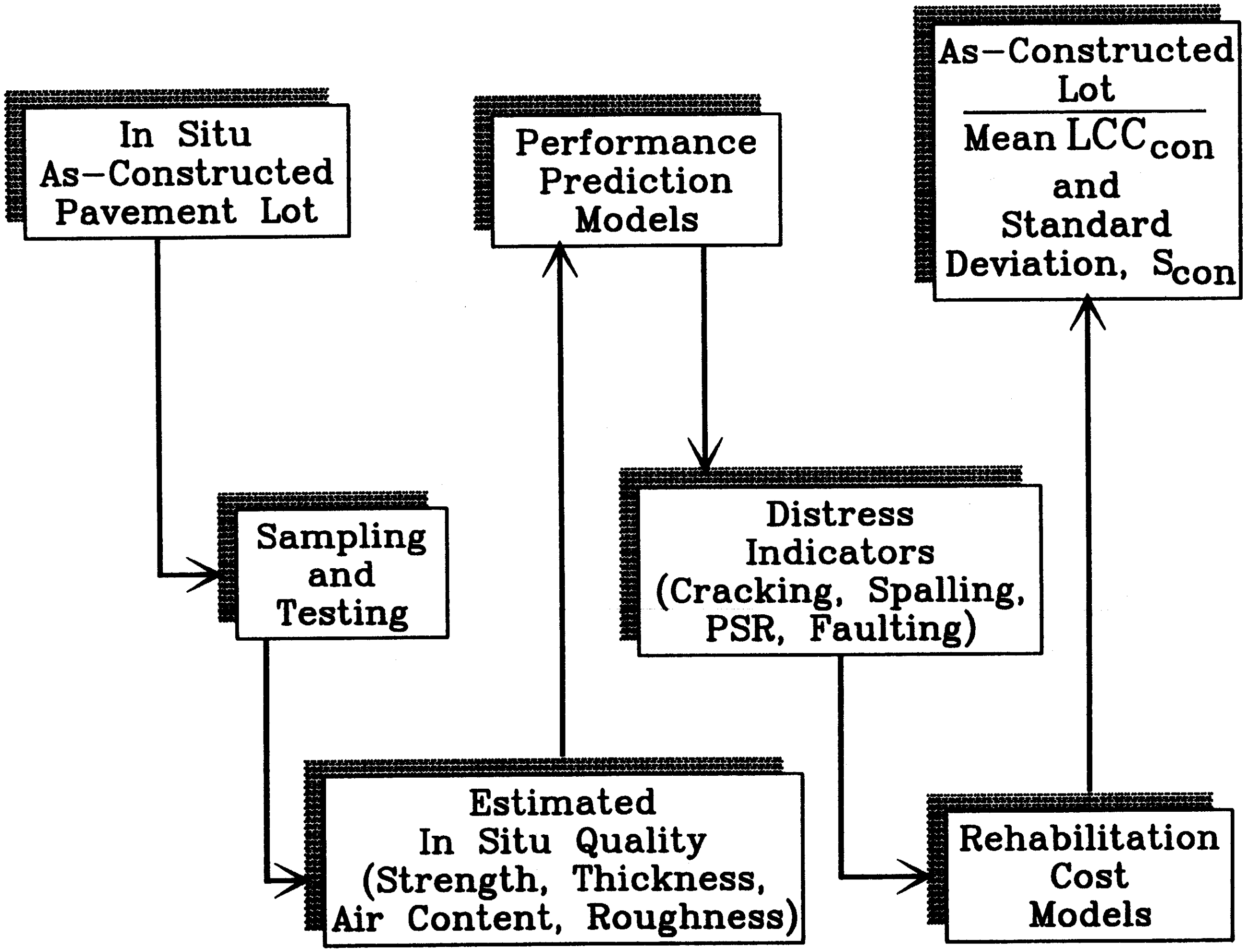


Figure 4. Procedures for estimation of as-constructed life-cycle cost.

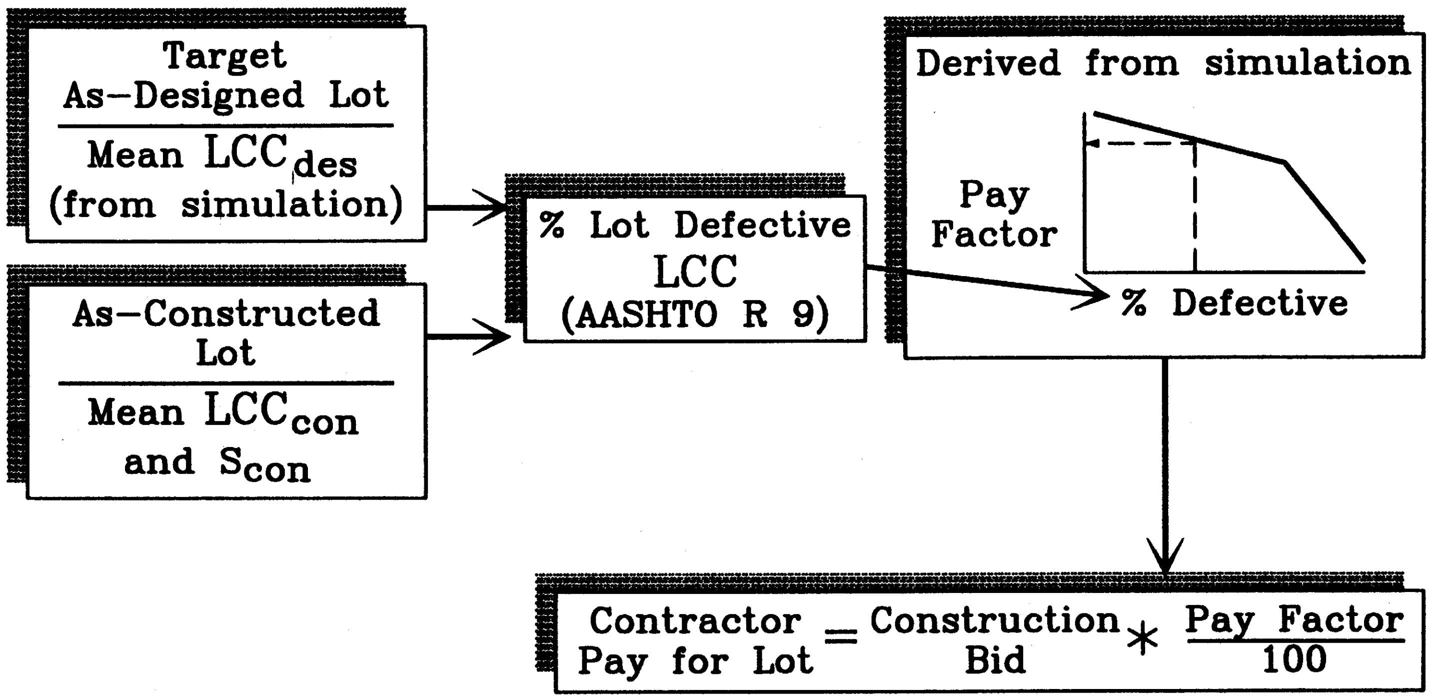


Figure 5. Calculation of contractor pay for pavement lot.

OPERATING CHARACTERISTIC (OC) CURVE

The operating characteristic (OC) curve provides vital information about the performance of a conventional statistical specification that does not include a pay adjustment schedule. The risks to both the contractor and the agency can be determined and controlled at acceptably low risk levels. An OC curve is a plot of the probability that a lot will be accepted versus the lot quality in terms of lot percent defective. The acceptance plan can be expected to perform as indicated by the OC curve. Only through the OC curve can it be determined if the sample size is sufficiently large to enable the procedures to properly discriminate between acceptable and unacceptable work.

When the specification includes a pay adjustment schedule with both positive and negative adjustments, the OC curve concept requires some modification. In this situation, there is no longer an acceptance/rejection situation (100-percent or 0-percent pay), but instead the contractor is paid on an adjusted scale according to the level of quality provided. Therefore, it appears that the degree of risk to the contractor and the agency is somewhat reduced, since the question of acceptance or rejection does not carry with it the large implications of either 100-percent or 0-percent pay.

When an adjusted pay schedule is used, Barros, Weed, and Willenbrock (see reference 9) and Willenbrock and Kopac (see reference 10) suggest that the "expected pay" (EP) can be computed (through simulation) over a range of lot percent defective and plotted similar to an OC curve (called an EP curve). Reference 9 states:

> *Expected pay factors are computed as the sum of the products of all pay factors multiplied by the probability of obtaining each pay factor.*[10] *This computation will numerically identify the mean value of the pay factor distribution. The expected payment (EP) curve relates probable payment to the true level of quality. This allows one to read the average pay factor directly from the Y-axis for any level of true percent defective, analogous to the OC curves already discussed.*

PRACTICALITY OF THE ENTIRE ACCEPTANCE PROCEDURE

The practicality of the acceptance procedures described herein depends on two major aspects: the measurement procedures and the pay adjustment procedures. The practicality of the measurement system depends on the ability to determine *in situ* concrete pavement characteristics vital to predicting its performance. Ideally, the measurement system should provide results soon after placement of the slab to enable the contractor to make adjustments to the production system if the targets are not being met. Another important test of practicality is the number of tests required to conduct the acceptance plan. If a large number of tests are required, the cost and time involved in carrying them out may discourage agencies from adopting the PRS.

The sampling and testing procedures recommended in the PRS for strength and air content represent the ideal level of testing. If they cannot be conducted, the PRS can still be utilized through the use of less than ideal sampling and testing procedures. For example, concrete strength could be determined in the conventional way using cylinders or beams cast from the concrete truck if the agency is willing to assume that the concrete samples obtained in this way will be equal to that measured from cores cut from the actual slab.

The practicality of the pay adjustments and the overall acceptance procedures must be carefully evaluated through many simulation runs of lots having a variety of as-designed and as-constructed characteristics. The results obtained using the recommended acceptance plan must be carefully considered from a practical and theoretical viewpoint. Examples of the procedure are provided in chapter 3.

SUMMARY

An overview of the approach used by the research team in the development of a prototype PRS is provided. The underlying theory of the approach is outlined and the overall methodology for acceptance testing and pay adjustment calculations are presented. The proposed approach was noted to have the following features:

- Mathematical relationships between quality characteristics of the pavement and future distress indicators.

- Inclusion of a rational procedure for computing pay adjustments based on the relationships between quality characteristics, rehabilitation policy, and the legal principle of liquidated damages.

- Consideration of any number of quality characteristics in the development of a rational overall pay adjustment.

- Consideration of within-lot variability and its effect on the pay factor.

- Provision of incentive to the contractor to provide high-quality work by allowing positive as well as negative pay adjustments for all quality characteristics.

- Requirement of using measurements of the *in situ* concrete pavement to provide for a true assessment of its as-constructed properties.

More details of the specification are provided in chapter 3, including several examples that better illustrate its use and application.

CHAPTER 3. DEVELOPMENT OF PROTOTYPE PRS

INTRODUCTION

This chapter describes the development of the prototype PRS for jointed concrete pavements. The PRS builds on the concepts and principles presented in chapter 2. A case study of the application of the prototype PRS is provided, which indicates how the specification may be used in an actual construction project. In addition, a sensitivity analysis of key PRS variables is included. The complete prototype PRS is given in appendix A.

OVERVIEW OF SPECIFICATION

The prototype specification directly considers four quality characteristics: concrete strength (S), slab thickness (T), concrete air content (A), and initial roughness (R). Significant construction items, such as dowel bar placement, methods of joint forming, and depth of reinforcement, are not currently included in the specification. Other factors, such as base, subbase, or shoulder construction, also are not currently included. It is believed that these components must eventually be considered for inclusion into the PRS. The underlying principles of the proposed prototype PRS allow for the inclusion of virtually an unlimited number of quality characteristics, provided that a prediction model exists that relates each quality characteristic to concrete pavement performance.

Definitions

The following specific definitions are provided for the various terms used in the specification:

- **Quality Characteristics:** Inherent characteristics of the pavement that significantly affect the performance of the pavement. This specification includes concrete strength, slab thickness, entrained air content, and initial roughness as quality characteristics.

- **Lot:** A discrete quantity of constructed pavement to which an acceptance procedure is applied. A lot is equal to one day's production or less. The lot consists of a pavement one or more traffic lanes wide (but does not include a shoulder).

- **Sublot:** A portion of a lot. The lot is divided into sublots of approximately equal surface area. This specification requires that sublots are uniquely defined for all sampling in that one or more samples of all quality characteristics are taken from each defined sublot. The minimum length of a sublot is 0.16 km (0.1 mi) so that roughness can be measured.

- **As-Designed Pavement:** The pavement as defined by the engineer. The desired quality level of the pavement must be clearly defined by specifying the means and standard deviations of the quality characteristics.

- **As-Constructed Pavement:** The *in situ* concrete pavement lot as constructed by the contractor.

- **Life-Cycle Cost (LCC):** The total cost of a lot over the pavement's analysis period. LCC in this specification consists of the estimated future rehabilitation costs over the analysis period and is expressed in terms of present worth through use of a specified discount rate. The initial construction cost is not included in the LCC since it is identical for both the as-designed and the as-constructed pavements.

- **Percent Defective (PD):** The percent of the lot falling above the mean target as-designed LCC value.

- **Pay Factor:** The percent of the bid price that the contractor is paid for the construction of a lot of concrete pavement. This is calculated from equation 1 in chapter 2, repeated here for convenience:

$$\text{Pay Factor} = 100\,(\,\text{BID} + \text{DIFF}\,)\,/\,\text{BID} \tag{6}$$

where:

$$\begin{aligned} \text{BID} &= \text{Contractor's bid price for the lot, \$} \\ \text{DIFF} &= LCC_{des} - LCC_{con} \\ LCC_{des} &= \text{As-designed life-cycle cost for lot, \$} \\ LCC_{con} &= \text{As-constructed life-cycle cost for lot, \$} \end{aligned}$$

Sampling of Quality Characteristics

Acceptance of the as-constructed pavement is based on *in situ* tests. This is a marked departure from conventional specifications, where most samples are taken from the plant or from trucks. It is believed that only samples taken from the *in situ* pavement provide a true indication of the properties of the as-constructed pavement, which is what is needed to predict its future performance.

The acceptance sampling and test results are required to calculate the pay adjustment under this specification. All sampling is performed by the agency in accordance with the American Society of Testing and Materials (ASTM) specification D3665, "Standard Practice for Random Sampling of Construction Materials."

The lot is divided into approximately equal area sublots within which each quality characteristic is sampled. The minimum length of a sublot is approximately 0.16 km (0.1 km) to accommodate the measurement of longitudinal roughness. The random selection process illustrated in ASTM D3665 is used within each sublot to select locations for the individual samples of strength, thickness, and air content. A minimum of three sublots is required per lot.

If a lot is constructed that is less than 0.5 km (0.3 mi) long, making it impossible to obtain a minimum of three sublots of 0.16 km (0.1 mi) each, the lot can be accepted by the engineer upon a visual inspection of the section and a review of process control results.

This procedure is unique in that it produces samples of strength, thickness, air content, and roughness from each designated sublot. This is required so that the LCC of each sublot can be calculated and used in the acceptance procedure. Table 4 summarizes the key aspects of the sampling plan.

Table 4. Summary of key aspects of the sampling plan.

Quality Characteristic	Point of Acceptance	Lot Size	Sublot Size
Entrained air content of concrete	Measured behind paver, after concrete placement, or from cores	Maximum: 1 day's production Minimum: 0.5 km (0.3 mi) length (three sublots)	Approximately 0.16 km (0.1 mi) long
Thickness of slab	Cores drilled from hardened concrete		
Strength of concrete	Cores drilled from hardened concrete		
Roughness of surface	Profilograph measurement		

Testing of Quality Characteristics

Any standard test may be used to measure the quality characteristics in the acceptance plan, provided the following conditions are satisfied:

1. The standard test method is pre-approved by the agency.

2. A pre-approved conversion factor to convert the acceptance test concrete strength result to the strength characteristic specified in the design is applied (for example, a 72-h core compressive strength is used for acceptance, but the design specifies a 28-d flexural strength). Each agency should develop their own conversion factors for specific tests and for the specific concrete materials used in the project.

3. Slab thickness and strength are measured on each core sample taken from a sublot.

4. Air content is measured from either hardened concrete (determined by conducting a linear traverse on a different core than those taken for strength and thickness) or from the plastic concrete sampled behind the paver (determined using a conventional air pressure meter).

5. Roughness is measured over the same sublot as designated for strength, thickness, and air content.

Concrete Strength

In situ concrete strength is determined after a minimum of 72 h of equivalent laboratory curing condition maturity from placement. A minimum of two cores (more could be specified if desired) are cut from the slab in each sublot and, after being measured for thickness, are tested to determine the concrete strength. The procedure for coring cylinders from the pavement is specified in AASHTO T23, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," and in ASTM C42, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete."

Specimens are kept moist during coring operations, during transport to the testing facility, and prior to testing. This may be accomplished by covering the samples with a wet blanket of burlap or other suitable absorbent material. The material is kept wet until testing. Specimens are transported to the testing facility in such a way as to not damage them.

Either standard compression or splitting tensile strength tests are conducted by the agency within 4 h of removal of the core from the concrete slab. The minimum 40-h water submersion requirement of ASTM C42 prior to core testing is waived.

Compressive Strength

The compressive strength of cylindrical core concrete specimens is determined using AASHTO T22, "Concrete Strength of Cylindrical Concrete Specimens" or ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." The mean compressive strength of the cores from each sublot is adjusted using maturity methods to obtain an equivalent mean 28-d compressive strength under standard laboratory-cured conditions (this procedure is illustrated in chapter 4). The equivalent mean 28-d compressive strength is then converted to a third-point loading flexural strength using an approved relationship developed from the specific concrete mixture for the lot. The laboratory work in chapter 4 indicates that such relationships could be established for specific mixes. The estimated mean 28-d third-point loading flexural strength is used as the strength of the sublot sample.

Based on the laboratory work conducted under this study, no correction factors are needed for converting the compressive strengths obtained from cores to an equivalent compressive strength obtained from standard-cured cylinders.

Splitting Tensile Strength

The splitting tensile strength is determined using AASHTO T128, "Standard Method of Splitting Tensile Strength of Cylindrical Concrete Specimens," or ASTM C496, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens." The mean splitting tensile strength of the cores from each sublot is adjusted using maturity methods (see chapter 4) to an equivalent mean 28-d splitting tensile strength under standard, laboratory-cured conditions. The equivalent mean 28-d splitting tensile strength is then converted to a mean third-point loading flexural strength using an approved relationship developed from the specific concrete mixture for the lot. The estimated 28-d mean third-point loading flexural strength is used as the strength of the sublot sample.

Maturity Adjustments

The concrete strength results obtained from each core at a minimum of 72 h equivalent laboratory curing are adjusted to obtain an equivalent 28-d, standard laboratory-cured flexural strength under third-point loading. The adjustment is determined from curves derived from actual on-site project materials. The application of maturity concepts in making this adjustment is described in chapter 4.

<u>Slab Thickness</u>

The thickness of the *in situ* pavement is determined by measurements taken in accordance with AASHTO T148, "Standard Method of Measuring Length of Drilled Concrete Cores," or ASTM C174, "Standard Test Method for Measuring Length of Drilled Concrete Cores." The same core samples used to determine slab thickness are used to determine concrete strength. The mean slab thickness of the cores taken in each sublot is used as the thickness of the sublot sample.

<u>Air Content</u>

The following procedures refer specifically to projects located in freeze areas where deicing salts are used and entrained air content is critical to concrete durability. If the project is not located in this type of climate, air content is controlled on a simple acceptance/rejection basis when sampled using conventional procedures.

The air content of the *in situ* slab is sampled in each sublot according to one of the following methods:

1. Plastic concrete is removed from the placed slab behind the paver at a random location in the sublot and tested with an air pressure meter according to AASHTO T152, "Air Content of Freshly Mixed Concrete by the Pressure Method,"

or ASTM C231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method."

2. An approved test method that is capable of determining the air content of plastic *in situ* concrete taken from behind the paver at a random location is used.

3. A linear traverse is performed on a hardened concrete core sample according to ASTM C457, "Standard Test Method For Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete." Separate core samples are taken at random locations exclusively for linear traverse testing from each sublot.

Two samples per sublot are required regardless of the method used. The mean air content determined using any of the above methods is used as the air content of the sublot sample.

Pavement Roughness

Pavement roughness is tested with a standard profilograph device in accordance with specified procedures. One such device is the California profilograph, described in California DOT specification CA-526. Roughness is measured as soon as the concrete has hardened sufficiently so that it can be tested without damage.

Profile measurements are recommended 0.9 m (3 ft) from and parallel to each longitudinal traffic lane edge of pavement. The roughness measurements are conducted along the length of each sublot and the measurement converted to the standard unit of m/km (in/mi). All roughness profile measurements for the sublot are then averaged. Roughness measurements taken prior to any surface correction are used as the roughness of the sublot sample.

Retesting Procedures

Additional sampling and testing for any of the quality characteristics for acceptance testing may be requested at any time by the contractor or by the agency. The agency conducts all of the sampling and testing for any retesting activities.

The pavement can be retested only once. Except for cases of testing errors (that are agreed upon by both the contractor and the agency), initial test results are included along with the retest values in the acceptance process by averaging all values from each sublot.

Acceptance

Quality Characteristics

The acceptance of a single lot is based on the following quality characteristics of the concrete pavement:

- Strength of concrete slab.
- Thickness of slab.
- Air content of concrete slab.
- Roughness of slab surface.

These quality characteristics are combined into a single quality characteristic, the future LCC of the pavement. The future LCC quality characteristic relates directly to the future performance of the pavement, and is used as the single overall quality characteristic for acceptance.

Life-Cycle Costs

The LCC for both the target as-designed (LCC_{des}) and the as-constructed (LCC_{con}) pavement lots include only the estimated future rehabilitation costs. The future rehabilitation costs consist of costs for full-depth repairs, slab replacement, and overlays, according to a specified rehabilitation policy. These costs are calculated over the designated design analysis period and expressed as a present worth cost. All cost calculations are performed using the PaveSpec computer software.

The rehabilitation policy used to calculate the LCC_{con} must be specified by the agency. Three options exist in the PaveSpec software:

- Rehabilitation Option (A). Individual sublots are rehabilitated through full-depth repairs, slab replacements, and overlays, independently of each other.

- Rehabilitation Option (B). Individual sublots are rehabilitated by slab replacements and full-depth repairs independently; however, when a critical amount of distress has occurred over the entire lot, the entire lot is overlaid.

- Rehabilitation Option (C). This option is the same as (B), but when a critical amount of distress has occurred over a selected percent of sublots, the entire lot is overlaid.

As-Designed Target Pavement

The target as-designed pavement is defined as the desired construction quality for which the agency will pay 100 percent of the bid price. It includes target means and standard deviations for each of the quality characteristics considered in the acceptance plan. The target standard deviations of the quality characteristics are representative of acceptable quality. These standard deviations are point-to-point variations (including testing errors) for strength, thickness, and entrained air content in the concrete slab, and variations between longitudinal profiles for initial roughness.

The lot target as-designed mean LCC_{des} is determined by simulating a large number of lots using the target means and standard deviations of the quality characteristics. The as-designed target quality characteristics are the mean values of thickness, strength, air content, and roughness set by the designer as mean targets to be achieved for the as-constructed lot. There are also many as-designed pavement constant inputs that must be specified for a given contract (traffic factors, climatic factors, and so on).

As-Constructed Pavement Lot

The as-constructed pavement lot is divided into sublots (a minimum of three) and each is randomly sampled and tested. The sublot mean sample values of strength, thickness, air content, and roughness (along with a selected rehabilitation policy) are used in the PaveSpec computer program to calculate the expected future LCC_{con} for the as-constructed lot. The rehabilitation costing policy to be used in PaveSpec must be specified.

A basic assumption of variable acceptance theory is that the population (lot) that is being sampled is normally distributed. This assumption is critical in the case of the LCC and, if not correct, could result in error in the calculation of percent defective. The chi-square goodness-of-fit test, which is a statistical procedure used to verify the validity of an assumed distribution, was run on several LCC simulations to test normality. Figure 6 shows a plot of one of the lots that includes 100 sublots. The theoretical expected frequencies for a normal distribution are also shown. The chi-square test indicates that the normal distribution is a valid model for LCC at the 0.0026 significance level. Observations of several LCC distributions, however, show that there can be a definite skew to the right (tendency for a few higher LCC values). Further research is needed on this topic.

Percent Defective Calculation

The percent defective of the as-constructed lot is defined as the proportion of the lot having an LCC_{con} greater than the LCC_{des}. The percent defective is calculated from equation 2 in chapter 2, repeated here for convenience:

$$Q = (LCC_{des} - LCC_{con}) / S_{con} \tag{7}$$

where:

Q	=	Quality index
LCC_{des}	=	As-designed target life-cycle cost, $
LCC_{con}	=	As-constructed life-cycle cost of lot, $
S_{con}	=	Standard deviation of LCC between as-constructed sublots, $

The percent defective as-constructed LCC is determined from the PaveSpec software or from table C in AASHTO R 9.[8] (The table from reference 8 is also provided as table 3 in chapter 2.) The tables must be entered with the quality index (Q) and the sample size (n), the latter of which is equal to the number of sublots, because all the within-sublot samples are averaged to obtain a sublot mean.

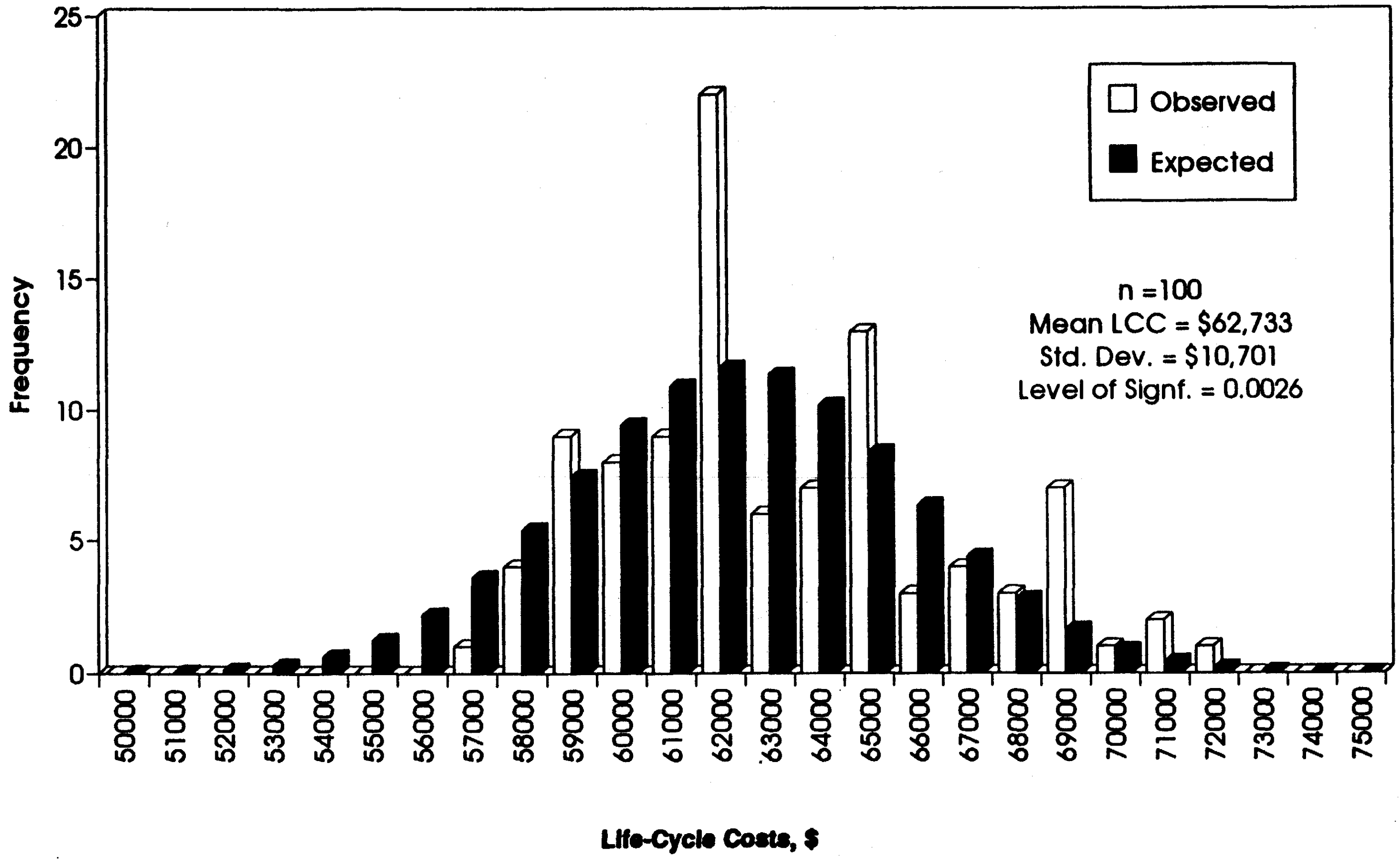


Figure 6. Frequency distribution of the LCC for 100 sublots (within a lot) and comparison with the theoretical normal distribution.

Rejected Quality Level

The constructed lot can be rejected and removed at the contractor's expense if any of the individual quality characteristics exceed the following limits (after retesting has been performed, if requested):

1. The mean lot as-constructed thickness is less than 90 percent of the as-designed target value.

2. The mean lot as-constructed concrete flexural strength is less than 75 percent of the as-designed target value.

3. The mean lot as-constructed total air content is less than 65 percent of the as-designed target value. (This requirement is only for projects in freeze areas where deicing salts are used.)

Retesting

If retesting of any of the quality characteristics is requested by either the agency or the contractor, this is carried out as previously described. A new mean for the quality characteristic is computed (including the previously determined values) and used to calculate a new as-constructed LCC_{con} for the lot. The original LCC_{con} value is disregarded.

Basis Of Payment

The contractor is paid based upon the achieved quality of the as-constructed pavement lot. The contractor payment is adjusted by a pay factor when the quality level of the as-constructed pavement varies from the quality level of the as-designed pavement.

The pay factor is determined using equation 3 from chapter 2 for percent defective values less than 90 percent, repeated here for convenience:

$$PF = A - B * PD \tag{8}$$

where:

PF	=	Pay factor
PD	=	Percent defective (less than 90 percent)
A,B	=	Constants to be determined by the agency for each project

The constants A and B are determined by the agency and will vary with different projects. They are determined through simulation using the PaveSpec program.

When the percent defective is 90 percent or more, the relationship is modified so that the pay factor varies from that at the 90-percent defective level to a value of 50 percent at the 100-percent defective level. This results in equation 4 from chapter 2, repeated here for convenience:

$$PF = (10 * A) + (900 * B) - 450 + (-0.1*A - 9*B + 5.0) * PD \tag{9}$$

where:

PD = Percent defective (90 percent or greater)
A,B = Constants to be determined by the agency for each project (these will be the same as those determined in equation 8)

The total payment to the contractor for the lot is determined using equation 5 from chapter 2, repeated here for convenience:

$$\text{Payment} = BP * PF / 100 \tag{10}$$

where:

Payment = Payment to the contractor for the lot
BP = Contractor bid price for the lot

OPERATING CHARACTERISTIC (OC) CURVE

As discussed in chapter 2, the concept behind the OC curve must be modified for specifications containing adjusted pay schedules. This is because the acceptance plan does neither "accept" nor "reject" (except for very poor lot quality), but rather calculates a pay factor for the lot based on the level of the quality characteristics sample. When an adjusted pay schedule is used, the "expected pay" (or EP) is computed (through simulation) over a range of lot percent defective and plotted in a format similar to that for an OC curve.

PaveSpec Version 1.0 does not have the capability to compute the expected pay factor curve automatically. However, it can provide the data required to establish the graph for a given acceptance plan and project. This is accomplished through a simulation of sampling from each of many lots that have a wide range of percent defective (i.e., 0 to 100 percent). For each point on the expected pay (EP) curve, the coordinates are determined as follows:

1. A large number of samples (say 50 or more) are obtained through simulation from a given lot using PaveSpec for specific as-designed and as-constructed quality characteristics. This results in a series of predictions of the lot pay factor and of the lot percent defective.

2. The percent defective is determined as the mean percent defective from all of the samples.

3. The pay factor is determined as the mean pay factor from all of the samples.

This procedure establishes a single point on the EP curve. This process is repeated many times by varying the quality characteristics so that a range of pay factors and percent defectives are obtained. The resulting EP curve relates the probable contractor payment to the level of quality (percent defective) in the pavement lot, which is a graphical representation of the operation of the specific acceptance plan.

USING PREDICTION AND COST MODELS TO CALCULATE LCC

Performance Prediction Models

The life-cycle costs on which the proposed PRS approach is based depend heavily on the predicted performance of the as-designed and as-constructed lots. The present approach for predicting performance (as programmed in the PaveSpec computer software) uses a set of distress models that are believed to be the best models currently available. Distress indicators are estimated by predicting yearly values of joint faulting, transverse cracking, and joint spalling, which are then used in turn to estimate the present serviceability rating (PSR).

Improvements in the accuracy of the prediction models over time can enhance the capabilities of the specification. However, it is also important to note that both the as-designed and the as-constructed lots are subjected to the same performance prediction models, meaning that any deficiencies in the models should affect the as-designed and as-constructed pavements equally.

The following models (for both jointed plain and jointed reinforced concrete pavements) are currently programmed in the PaveSpec computer program:

- Transverse cracking model from reference 6.
- Transverse joint faulting model from reference 11.
- Transverse joint spalling model from reference 11, with a modification based on the results from the laboratory study in chapter 4.
- Present Serviceability Rating (PSR) model from reference 11.
- Initial Present Serviceability Rating (PSR) model as a function of initial roughness index measured with a profilograph (project-specific model).

The models and their inputs are described in detail in appendix B.

Joint Spalling Model

Spalling at transverse joints is caused by several factors, including incompressibles, lack of consolidation, dowel bar alignment, D-cracking, reactive aggregate, and damage due to a deficient air void system. The initial PRS described in this report includes air content as a quality characteristic, which would have a direct effect on spalling (especially in freeze climates where deicing salts are used).

The most comprehensive distress models for joint spalling available were developed in 1990 for the FHWA using the large data base of inservice concrete pavements.[11] Spalling models that predict the number of medium- and high-severity spalled joints per mile were developed independently for JRCP and JPCP and include factors such as pavement age, freezing index, joint sealant type, and the presence of D-cracking and reactive aggregates. These models are referred to as "field spalling models." They generally show that joint spalling occurs only after 10 to 15 years, mostly due to a buildup of incompressibles in the joint. Few, if any, of the pavement sections used to develop these models showed any signs of scaling or spalling due to an air content deficiency, and these are distresses that usually occur within the first 10 years or so. Thus, in order to directly consider the effect of deficient air content as well as the effects of incompressibles, lack of dowel alignment, and other factors, the field spalling models were modified using the extensive laboratory study described in chapter 4.

The laboratory study included the freezing and thawing of concrete blocks with a wide range of concrete material properties, some of which were subjected to a calcium chloride solution. Laboratory models were developed that predict the percent of joint length spalled. Equation 21 (in chapter 4) was selected to be used in the PRS prediction of spalling caused by inadequate air content. This laboratory model includes inputs of calcium chloride usage, number of freeze-thaw cycles, air content, and compressive strength. This model is, however, based on laboratory results and thus would have questionable applicability to the development of actual joint spalling in the field. A methodology by which the field spalling models were modified by a ratio of spalling at different air contents was developed from the laboratory model. The final spalling prediction model is as follows:

$$\text{Joint Spalling} = \text{Spalling}_{FM} * (\text{A-C Spalling} / \text{A-D Spalling}) \qquad (11)$$

where:

Joint Spalling	=	Total number of spalled joints per mile from all causes
Spalling_{FM}	=	Spalling predicted from the field spalling model
A-C Spalling	=	Spalling predicted from the laboratory spalling model using air content and compressive strength from the as-constructed pavement
A-D Spalling	=	Spalling predicted from the laboratory spalling model using air content and compressive strength from the as-designed pavement

The complete spalling model development is discussed in appendix B.

Some practical limits were necessary for this model. A lower limit to the as-constructed spalling/as-designed spalling ratio was set at 1.0. This specifies that spalling will not decrease when the as-constructed air content is greater than that specified for the as-designed pavement. Further evaluation of the model has also shown the need for an upper limit of approximately 3.0, to avoid a large jump in spalling over the early freeze cycles due to the logarithmic nature of the laboratory spalling model.

The effect of air content on joint spalling is greatly influenced by the use of deicing salts containing chlorides. In geographical areas where no deicing salts are used, there is likely to be little freezing of the pavement and thus the purpose of entrained air is solely to improve workability. In this case, the field and laboratory models will show little effect of air content on spalling.

This spalling model should be considered as approximate, and further research work is required to verify the model in the field. The point to be remembered, however, is that the pay adjustment depends on the difference between the as-designed and the as-constructed pavement. It is the relative difference in these predictions that affects the pay adjustment, not the absolute prediction of spalling or any other distress type.

The sensitivity of the spalling model is shown in figure 7 for a JPCP located in a freeze area (freezing index = 300 degree days below freezing, 10 freeze-thaw cycles) where deicing salts are used. The effect of air content below the as-designed level is very significant.

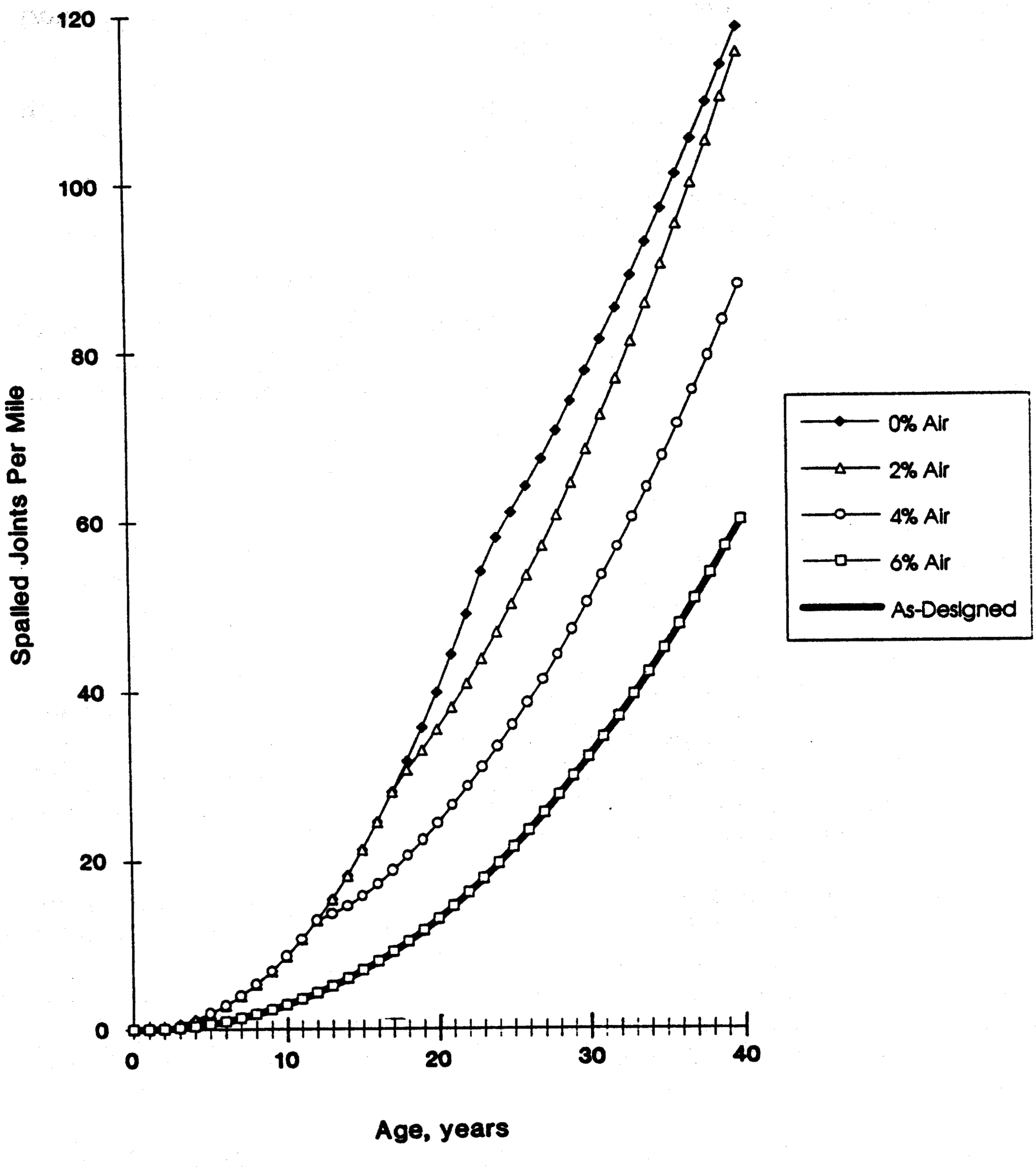
Cost Models (Rehabilitation Plans)

Cost models translate the distress indicators into life-cycle rehabilitation costs. The rehabilitation policy uses the predicted distress indicator levels to determine when and how much rehabilitation is needed throughout the analysis life.

The current approach (used by the PaveSpec computer program) applies a simplified rehabilitation policy based on a number of assumptions. Rehabilitation is assumed to consist of two types: localized and overlay. Localized rehabilitation consists of replacing cracked slabs (due to projected linear cracking) and replacing spalled joints (due to projected joint spalling). The rehabilitation plan was greatly simplified by assuming the replacement of all such distresses every year.

Overlay rehabilitation consists of an AC overlay that is applied either over the sublot experiencing serious distress or over the total area of the lot (depending upon the rehabilitation policy). The timing of the overlay application is selected by predetermined trigger values. These trigger values are applied to the development of the distresses since construction, as well as to the PSR. Other overlay-related assumptions include a user-defined overlay life and the application of additional overlays after the previous overlays have reached the end of their useful life.

The cost models and procedures work together in the following manner. The distress indicators are calculated for each year, and a PSR is estimated from the distress indicators. The PSR and the individual distress indicators are compared to predetermined trigger values that would signal an overlay. If an overlay is needed, the distress indicators are no longer used and an after-overlay rehabilitation plan is



1 mi = 1.61 km

Figure 7. Sensitivity of the joint spalling model used in PaveSpec.

adopted. If an overlay is not needed, the distress indicator histories are updated and necessary local rehabilitation is conducted.

Yearly costs are calculated by summing the individual local rehabilitation and overlay costs for the respective years. The present worth of the yearly costs are calculated to obtain a total sublot LCC. The total sublot LCC for both the as-designed and as-constructed lots may then be used in the calculation of the pay factor. The cost models employed in the computation are presented in appendix B.

PAVESPEC COMPUTER PROGRAM

The entire specification, including prediction models, cost models, percent defective calculations, and pay factor computations, has been programmed into the PaveSpec software program. The PaveSpec software is fully described in appendix B. The software is programmed in C and operates in the Windows® environment.

A working version of the program is available that accomplishes all of the important tasks. The program makes it possible to define the specification that is proposed. It also allows the user to simulate the construction and sampling process so that the expected pay curve for the specification and project can be developed. A brief description of some of the features of the program follows:

- The program performs three major functions: simulation of pavement construction parameters for a lot, sampling of these parameters, and prediction of performance and costs.

- The user can specify a lot in terms of distributions of variables and the simulator creates a complete "pavement" lot with these characteristics. The sampler mimics a specification process of taking random samples from each sublot, and calculates the sample mean and standard deviation.

- The prediction engine accepts the as-constructed sampling data, the specified as-designed characteristics, and specific climate, traffic, and other input data to estimate performance and calculate predicted LCC.

- The results are presented to the user in tabular and graphical formats; the user can compare the as-designed and the as-constructed LCC to obtain a pay factor for a lot.

The PaveSpec program is utilized and discussed further in the next section.

CASE STUDY OF SPECIFICATION APPLICATION FOR A PAVEMENT LOT

A simulation of the construction and quality acceptance of a concrete pavement lot using the new PRS is given. The acceptance is based on *in situ* testing of the concrete slab after placement. All calculations were performed using the PaveSpec computer program.

The case study project is a JPCP that is located on a rural four-lane divided freeway and is subjected to heavy truck traffic. The as-constructed pavement lot is 0.81 km (0.5 mi) long and is divided into five equal length sublots, each of which is sampled twice for each quality characteristic, according to the PRS acceptance plan. The following steps outline the procedure for calculating the pay factor for the pavement lot, and include a sensitivity analysis of the results.

Step 1. As-Designed Pavement Definition and LCC Calculations.

The as-designed pavement (and also the as-constructed pavement) constant input variables are shown in the PaveSpec screen in figure 8. This pavement was designed for 20 years of traffic (about 28 million equivalent single-axle load [ESAL] applications) using the 1986 AASHTO Design Guide and a 95-percent design reliability level. Note that any design procedure or design standard could have been used to develop the target pavement design, but all of the inputs specified in figure 8 would still need to be provided to define the as-designed pavement. A 40-year design analysis period is selected over which the future LCC will be calculated.

The lot consists of two lanes (each 3.7 m [12 ft] wide) of a concrete pavement of uniform thickness constructed over a length of 0.81 km (0.5 mi) in a given day. The unit bid price of the jointed plain concrete pavement is $23.92/m^2 ($20/yd^2).

The target, as-designed pavement quality characteristic values as specified by the designer are as follows:

- The target, as-designed mean strength (defined as the 28-d, standard laboratory-cured flexural strength tested under third-point loading) is 4.9 MPa (707 lbf/in^2). The as-designed target standard deviation for the strength is set at 0.49 MPa (71 lbf/in^2). This includes point-to-point strength variation and also variation due to testing error.

  Temperature was monitored in the slab immediately after concrete placement and maturity was calculated at the time of coring and used to extrapolate to a 28-d standard-cured strength. The slab was cored at 3 d and the cores tested immediately for compressive strength. This strength was extrapolated to a 28-d, standard laboratory-cured compressive strength using maturity methods. Laboratory correlations for the project concrete mixture show the following relationship between compressive and flexural strength:

  $$\text{Flexural Strength (lbf/in}^2) = 10\ (\text{Compressive Strength [lbf/in}^2])^{0.5} \qquad (12)$$

  The extrapolated 28-d standard laboratory-cured compressive strength was converted to third-point loading flexural strength using the above relationship. For this example, a 28-d standard laboratory-cured compressive strength of 34.5 MPa (5000 lbf/in^2) is required to meet the 4.9 MPa (707 lbf/in^2) flexural strength. The 4.9 MPa (707 lbf/in^2) mean flexural strength was used in the prediction models for the as-designed pavement distress predictions.

PaveSpec v1.0 r93.04.02c: Example 1 [Constant inputs]

Session Constants Window Tables Graphs Calculate Print Output

PROJECT	
Pavement Type	Plain, doweled
Road Location	Rural, 2 lanes
Design life, years	40 yr
Joint spacing	15 ft.
Joints per mile	352
Lot length	0.5 mi.
Number of lanes in one direction	1
Number of directions	1
Total lanes	1
Lane width	12 ft.
Total traffic lanes area	3,520 yd^2
Total shoulder width	16 ft.
Total shoulder area	4,693 yd^2
Total project area	8,213 yd^2
TRAFFIC	
Directional factor	100%
Percent trucks	20%
Percent trucks in outer lane	75%
Average truck load equivalency factor	1.7
Initial MESALs	1 MESAL
Initial ADT for considered directions	10,163
Growth factor	5%
Growth type	Simple
ADT-to-MESAL ratio	10,744
CLIMATIC AND MATERIALS	
Mean annual temperature range	48°F
Freezing index	300 deg. days
Mean annual precipitation	37 in.
D-cracking present?	No
Liquid sealant in joint?	Yes
Preformed compression seal in joint?	No
Reactive aggregate	No
Target air content	6.5%
BASE	
Base type	Granular
Effective modulus of reaction	300

PaveSpec v1.0 r93.04.02c: Example 1 [Constant inputs]

Session Constants Window Tables Graphs Calculate Print Output

BASE	
Base type	Granular
Effective modulus of reaction	300
Soil type	Fine-grained
Sub-drains present?	No
LOAD TRANSFER	
Tied PCC Shoulder?	No
Dowel diameter	1 in.
Area of reinforcement steel	N/A
COST	
Construction bid, traffic lanes	$ 20/yd^2
Total traffic lanes area	7,040 yd^2
Total traffic lanes bid price	$ 140,800
Total shoulder area	9,387 yd^2
Total lot bid price	$ 140,800
Overlay unit cost	$ 9/yd^2
Patch cost	$ 80/yd^2
Slab replacement cost	$ 70/yd^2
Annual inflation rate	3%
Annual interest rate	6%
Initial cost year	1,993
REHABILITATION	
Trigger on PSR?	Yes
Trigger on cracking?	Yes
Trigger on spalling?	Yes
Trigger on faulting?	Yes
PSR value after overlay	4.5
Assumed overlay life	10 yr
Joint patch width	6 ft
Inner-lane cracking	10% of outer lane

MESAL = Million Equivalent Single-Axle Load applications
ADT = Average Daily Traffic
1 yd^2 = 0.836 m^2; 1 ft = 0.305 m; 1 mi = 1.61 km; °C = (°F-32)$\frac{5}{9}$

Figure 8. Constant variables defined for the as-designed and the as-constructed pavement.

- The as-designed target mean slab thickness is 305 mm (12 in), determined using the 1986 AASHTO Design Guide and a 95-percent reliability level. The as-designed target standard deviation of the thickness is set at 6 mm (0.25 in). This value includes point-to-point thickness variation and also variation due to testing error.

- The as-designed target mean initial roughness (as measured by a profilograph) is 0.11 m/km (7.0 in/mi). According to the relationship shown in equation 13, this initial roughness is equivalent to an initial PSR of 4.5.

$$\text{Initial PSR} = 5.0 - 0.0714 * \text{Initial RI} \tag{13}$$

where:

RI = Roughness index measured by a California-type profilograph, in/mi

The as-designed target standard deviation for roughness is set at 0.016 m/km (1.0 in/mi). This value includes longitudinal profile variation and also variation due to testing error.

- The as-designed target air content is 6.5 percent. The as-designed target standard deviation for air content is set at 0.5 percent. This value includes point-to-point air content variation as well as variation due to testing error.

The mean target LCC_{des} is calculated using the PaveSpec program with the above target values and the constant input variables given in figure 8. The mean LCC_{des} is determined from 100 simulated lots, each containing 5 sublots and 2 samples of each quality characteristic per sublot. The mean LCC_{des} is computed to be $43,288/km ($69,694/mi), which means that any as-constructed sublot having an LCC above $43,288/km ($69,694/mi) would be considered as part of the percent defective. The details of the computation of future LCC_{des} is described under step 3, since the procedures are the same for both LCC_{con} and LCC_{des}.

Step 2. Sampling and Testing the As-Constructed Pavement.

Sampling. The sampling plan calls for two samples per sublot for each quality characteristic. The 0.81-km (0.5-mi) lot is divided into five equal sublots that are sampled randomly for strength, thickness, air content, and roughness.

Slab thickness. This testing is conducted in accordance with ASTM C174 for each core cut from the pavement. The two thickness results are averaged. The mean slab thickness value for each sublot is used for computation of LCC_{con}.

Strength. Each core sample is a 102-mm (4-in) diameter core cut from the pavement at 3 d, in accordance with ASTM C31. Each core is tested immediately for compressive strength using ASTM C39. The strength of the core is adjusted to a 28-d strength using maturity methods based on a laboratory-developed curve for the actual project materials. This strength is then adjusted to a flexural strength using the project-specific relationship from Step 1. The two results are averaged, and this mean concrete strength value for each sublot is used in the computation of LCC_{con}.

Air content. Plastic concrete is sampled at two random locations behind the paver and the air content is determined using an approved air pressure meter. The two results are averaged, and this mean air content value for each sublot is used in the computation of LCC_{con}.

Initial roughness. The California DOT specification CA 526 is followed to measure the initial roughness of the pavement for two profiles, 0.91 m (3 ft) from each edge of the outer lane. The measurements taken for each profile in a sublot are extrapolated to m/km (in/mi) and then averaged. The mean roughness value for each sublot is used to compute the initial serviceability rating (PSR), which in turn is used in the computation of LCC_{con}.

Sample values obtained for each of the quality characteristics are shown in table 5. The 3-d core strength values have been adjusted to a 28-d standard laboratory-cured compressive strength using maturity relationships. They will be converted to 28-d flexural strengths by PaveSpec for LCC_{con} computations using a correlation developed from project-specific materials.

Table 5. Summary of quality characteristics obtained from sampling the lot.

Pavement Section	Compressive Strength (lbf/in^2)	Thickness (in)	Air Content (percent)	Roughness (in/mi)	LCC_{con} (\$/mi)
As-designed	5,000	12.0	6.5	7.0	$ 69,694
Sublot 1*	5,450	12.3	6.5	8.2	61,245
Sublot 2	4,695	11.8	6.7	6.5	72,348
Sublot 3	4,983	12.6	6.6	6.5	61,420
Sublot 4	4,993	11.8	6.1	7.2	70,372
Sublot 5	5,027	12.0	6.8	5.9	65,971
Mean	5,030	12.1	6.5	6.9	66,271
Std. Dev.	270	0.33	0.3	0.9	5,065

* Mean of two samples
1000 lbf/in^2 = 6.9 MPa
1 in = 25.4 mm
1 in/mi = 0.016 m/km
1 mi = 1.61 km

Step 3. Computation of the Life-Cycle Cost of the As-Constructed Lot (LCC_{con}).

The above data are input into the PaveSpec program for computation of the LCC_{con} of the as-constructed pavement lot. PaveSpec first predicts all of the distress indicators (faulting, transverse cracking, joint spalling, and PSR) over each year of the analysis period for each sublot. An output of distress prediction is shown in figure 9 for one of the sublots.

The rehabilitation policy is applied each year to the distress data. For this example, it is assumed that sublots are rehabilitated by slab replacements for slab cracking and full-depth repairs for joint spalling independently of other sublots. When 20 percent of the sublots reached a critical trigger value for rehabilitation, the entire lot is overlaid. For this example, the trigger values given in table 6 were used:

Table 6. Trigger values used in case study for distress indicators.

Distress Indicator	Trigger Value
Mean Joint Faulting, in	0.12 (maximum)
Transverse Cracking, cracks/mi	67 (maximum)
Joint Spalling, spalls/mi	75 (maximum)
PSR	3.0 (minimum)

1 in = 25.4 mm
1 mi = 1.61 km

An example of the PaveSpec future cost output for one of the sublots is shown in figure 10. The LCC_{con} of each of the five sublots is shown in table 5, with the mean, as-constructed LCC_{con} = \$41,162/km (\$66,271/mi).

Step 4. Calculation of the Percent Defective of the Lot.

The percent defective of the as-constructed lot is defined as the proportion of the lot having an LCC_{con} greater than the as-designed LCC_{des}. The percent defective is calculated as follows:

$$Q = (LCC_{des} - LCC_{con}) / S_{con} = +0.676$$

where:

LCC_{des} = \$43,288/km (\$69,694/mi)
LCC_{con} = \$41,162/km (\$66,271/mi)
S_{con} = \$ 3,146/km (\$5,065/mi)

PaveSpec v1.0 r93.03.09a: Example 1

Session Constants Window Tables Graphs Calculate Print Output

Param > Year	MESAL	Faulting	Cracking	Spalling	Total Repairs	Repair cracks	Repair spalls	PSR
1	0 MESAL	0 in.	0 ft/mi	0 jts/mi	0	0	0	4.5
2	1 MESAL	0.01 in.	22.13 ft/mi	0 jts/mi	1	1	0	4.434
3	2.05 MESAL	0.01 in.	31.72 ft/mi	0 jts/mi	1	0	0	4.423
4	3.15 MESAL	0.02 in.	39.37 ft/mi	0 jts/mi	2	1	0	4.358
5	4.3 MESAL	0.02 in.	46.09 ft/mi	0 jts/mi	2	0	0	4.348
6	5.5 MESAL	0.02 in.	52.27 ft/mi	0 jts/mi	2	0	0	4.346
7	6.75 MESAL	0.03 in.	58.12 ft/mi	0 jts/mi	3	1	0	4.281
8	8.05 MESAL	0.03 in.	63.74 ft/mi	1 jts/mi	4	0	1	4.27
9	9.4 MESAL	0.03 in.	69.25 ft/mi	1 jts/mi	4	0	0	4.26
10	10.8 MESAL	0.04 in.	74.69 ft/mi	2 jts/mi	6	1	1	4.194
11	12.25 MESAL	0.04 in.	80.12 ft/mi	2 jts/mi	6	0	0	4.174
12	13.75 MESAL	0.04 in.	85.6 ft/mi	3 jts/mi	7	0	1	4.171
13	15.3 MESAL	0.05 in.	91.16 ft/mi	4 jts/mi	9	1	1	4.097
14	16.9 MESAL	0.05 in.	96.83 ft/mi	5 jts/mi	10	0	1	4.076
15	18.55 MESAL	0.05 in.	102.7 ft/mi	6 jts/mi	11	0	1	4.065
16	20.25 MESAL	0.05 in.	108.7 ft/mi	7 jts/mi	13	1	1	4.054
17	22 MESAL	0.06 in.	114.9 ft/mi	8 jts/mi	14	0	1	3.968
18	23.8 MESAL	0.06 in.	121.5 ft/mi	9 jts/mi	15	0	1	3.956
19	25.65 MESAL	0.06 in.	128.3 ft/mi	10 jts/mi	17	1	1	3.944
20	27.55 MESAL	0.06 in.	135.4 ft/mi	11 jts/mi	18	0	1	3.922
21	29.5 MESAL	0.07 in.	143 ft/mi	13 jts/mi	20	0	2	3.845
22	31.5 MESAL	0.07 in.	151 ft/mi	14 jts/mi	22	1	1	3.824
23	33.55 MESAL	0.07 in.	159.4 ft/mi	16 jts/mi	24	0	2	3.801
24	35.65 MESAL	0.07 in.	168.3 ft/mi	17 jts/mi	26	1	1	3.779
25	37.8 MESAL	0.08 in.	177.8 ft/mi	19 jts/mi	28	0	2	3.692
26	40 MESAL	0.08 in.	187.9 ft/mi	21 jts/mi	31	1	2	3.669
27	42.25 MESAL	0.08 in.	198.6 ft/mi	23 jts/mi	34	1	2	3.637
28	44.55 MESAL	0.08 in.	210.1 ft/mi	25 jts/mi	36	0	2	3.605
29	46.9 MESAL	0.09 in.	222.3 ft/mi	27 jts/mi	39	1	2	3.517
30	49.3 MESAL	0.09 in.	235.3 ft/mi	29 jts/mi	42	1	2	3.484
31	51.75 MESAL	0.09 in.	249.1 ft/mi	32 jts/mi	45	0	3	3.45
32	54.25 MESAL	0.09 in.	263.9 ft/mi	34 jts/mi	48	1	2	3.418
33	56.8 MESAL	0.1 in.	279.7 ft/mi	36 jts/mi	51	1	2	3.32
34	59.4 MESAL	0.1 in.	296.5 ft/mi	39 jts/mi	55	1	3	3.285
35	62.05 MESAL	0.1 in.	314.5 ft/mi	42 jts/mi	59	1	3	3.242
36	64.75 MESAL	0.1 in.	333.6 ft/mi	44 jts/mi	62	1	2	3.199
37	67.5 MESAL	0.11 in.	354 ft/mi	47 jts/mi	66	1	3	3.099
38	70.3 MESAL	0.11 in.	375.7 ft/mi	50 jts/mi	70	1	3	3.055
39	73.15 MESAL	0.11 in.	398.9 ft/mi	53 jts/mi	75	2	3	3.01
40	76.05 MESAL	0.11 in.	423.5 ft/mi	56 jts/mi	79	1	3	4.5
41	79 MESAL	0 in.	0 ft/mi	0 jts/mi	0	0	0	4.35

MESAL = Million Equivalent Single-Axle Load applications
1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Figure 9. Prediction of distress indicators for an as-constructed sublot.

PaveSpec v1.0 r93.03.09a: Example 1

Session Constants Window Tables Graphs Calculate Print Output

Sublot >

Param > / Year	Repair cracks	Repair spalls	Rehab Total	User cost	Overlay	Total	PW cracks	PW repair spalls	PW rehab	PW user	PW overlay	PW total
Initial				$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 1	$ 1,400	$ 0	$ 1,540	$ 0	$ 0	$ 1,540	$ 1,359	$ 0	$ 1,495	$ 0	$ 0	$ 1,495
Year 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 4	$ 1,400	$ 0	$ 1,540	$ 0	$ 0	$ 1,540	$ 1,244	$ 0	$ 1,368	$ 0	$ 0	$ 1,368
Year 5	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 6	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 7	$ 1,400	$ 640	$ 2,820	$ 0	$ 0	$ 2,820	$ 1,138	$ 520.4	$ 2,293	$ 0	$ 0	$ 2,293
Year 8	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 9	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 490.5	$ 981	$ 0	$ 0	$ 981
Year 10	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 11	$ 1,400	$ 640	$ 2,820	$ 0	$ 0	$ 2,820	$ 1,011	$ 462.3	$ 2,037	$ 0	$ 0	$ 2,037
Year 12	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 448.9	$ 897.8	$ 0	$ 0	$ 897.8
Year 13	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 435.8	$ 871.6	$ 0	$ 0	$ 871.6
Year 14	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 423.1	$ 846.2	$ 0	$ 0	$ 846.2
Year 15	$ 1,400	$ 640	$ 2,820	$ 0	$ 0	$ 2,820	$ 898.6	$ 410.8	$ 1,810	$ 0	$ 0	$ 1,810
Year 16	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 398.8	$ 797.7	$ 0	$ 0	$ 797.7
Year 17	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 387.2	$ 774.4	$ 0	$ 0	$ 774.4
Year 18	$ 1,400	$ 640	$ 2,820	$ 0	$ 0	$ 2,820	$ 822.4	$ 375.9	$ 1,656	$ 0	$ 0	$ 1,656
Year 19	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 365	$ 730	$ 0	$ 0	$ 730
Year 20	$ 0	$ 1,280	$ 2,560	$ 0	$ 0	$ 2,560	$ 0	$ 708.7	$ 1,417	$ 0	$ 0	$ 1,417
Year 21	$ 1,400	$ 640	$ 2,820	$ 0	$ 0	$ 2,820	$ 752.6	$ 344	$ 1,516	$ 0	$ 0	$ 1,516
Year 22	$ 0	$ 1,280	$ 2,560	$ 0	$ 0	$ 2,560	$ 0	$ 668	$ 1,338	$ 0	$ 0	$ 1,338
Year 23	$ 0	$ 640	$ 1,280	$ 0	$ 0	$ 1,280	$ 0	$ 324.3	$ 648.6	$ 0	$ 0	$ 648.6
Year 24	$ 1,400	$ 1,280	$ 4,100	$ 0	$ 0	$ 4,100	$ 688.7	$ 629.7	$ 2,017	$ 0	$ 0	$ 2,017
Year 25	$ 0	$ 1,280	$ 2,560	$ 0	$ 0	$ 2,560	$ 0	$ 611.3	$ 1,223	$ 0	$ 0	$ 1,223
Year 26	$ 0	$ 1,280	$ 2,560	$ 0	$ 0	$ 2,560	$ 0	$ 593.5	$ 1,187	$ 0	$ 0	$ 1,187
Year 27	$ 1,400	$ 1,280	$ 4,100	$ 0	$ 0	$ 4,100	$ 830.3	$ 576.2	$ 1,846	$ 0	$ 0	$ 1,846
Year 28	$ 0	$ 1,280	$ 2,560	$ 0	$ 0	$ 2,560	$ 0	$ 559.5	$ 1,119	$ 0	$ 0	$ 1,119
Year 29	$ 1,400	$ 1,280	$ 4,100	$ 0	$ 0	$ 4,100	$ 594.1	$ 543.2	$ 1,740	$ 0	$ 0	$ 1,740
Year 30	$ 1,400	$ 1,920	$ 5,380	$ 0	$ 0	$ 5,380	$ 576.8	$ 791	$ 2,218	$ 0	$ 0	$ 2,218
Year 31	$ 0	$ 1,280	$ 2,560	$ 0	$ 0	$ 2,560	$ 0	$ 512	$ 1,024	$ 0	$ 0	$ 1,024
Year 32	$ 1,400	$ 1,280	$ 4,100	$ 0	$ 0	$ 4,100	$ 543.7	$ 497.1	$ 1,592	$ 0	$ 0	$ 1,592
Year 33	$ 0	$ 1,920	$ 3,840	$ 0	$ 0	$ 3,840	$ 0	$ 723.9	$ 1,448	$ 0	$ 0	$ 1,448
Year 34	$ 1,400	$ 1,920	$ 5,380	$ 0	$ 0	$ 5,380	$ 512.5	$ 702.8	$ 1,969	$ 0	$ 0	$ 1,969
Year 35	$ 1,400	$ 1,280	$ 4,100	$ 0	$ 0	$ 4,100	$ 497.5	$ 454.9	$ 1,457	$ 0	$ 0	$ 1,457
Year 36	$ 1,400	$ 1,920	$ 5,380	$ 0	$ 0	$ 5,380	$ 483	$ 662.5	$ 1,856	$ 0	$ 0	$ 1,856
Year 37	$ 0	$ 1,920	$ 3,840	$ 0	$ 0	$ 3,840	$ 0	$ 643.2	$ 1,286	$ 0	$ 0	$ 1,286
Year 38	$ 1,400	$ 1,920	$ 5,380	$ 0	$ 42,240	$ 47,620	$ 455.3	$ 624.4	$ 1,750	$ 0	$ 13,738	$ 15,487
Year 39	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Year 40	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Total	$ 22,400	$ 33,920	$ 92,480	$ 0	$ 42,240	$ 134,720	$ 12,208	$ 15,889	$ 45,207	$ 0	$ 13,738	$ 58,945
Cost per mile	$ 22,400	$ 33,920	$ 92,480	$ 0	$ 42,240	$ 134,720	$ 12,208	$ 15,889	$ 45,207	$ 0	$ 13,738	$ 58,945

PW = present worth
1 mi = 1.61 km

Figure 10. Prediction of future rehabilitation costs for an as-constructed sublot.

The percent defective as-constructed LCC_{con} is determined from the PaveSpec software or from AASHTO R 9, table C (included as table 3 in chapter 2). Using a sample size of n = 5 and a Q = + 0.676, the percent defective is 26.54 percent.

Step 5. Determination of Pay Factor-Percent Defective Relationship.

The relationship between the lot percent defective LCC and the pay factor for this specific project was obtained through simulation using the PaveSpec program. A large number of lots (100, for example) are simulated using the as-designed quality characteristics (means and standard deviations) and the percent defective and pay factor are computed for each lot.

For one such point on the graph, consider the following results. The as-designed LCC_{des} is the same as before. A lot is sampled for S, T, A, and R and the as-constructed LCC_{con} is computed the same as in step 4. The percent defective is then computed, also as described in step 4. The pay factor is calculated according to equation 1:

$$\begin{aligned}
\text{Pay Factor} &= (\text{Bid Price} + \text{Diff}) \,/\, \text{Bid Price} \\
&= (\$281{,}600/\text{mi} + \$1{,}342\ \text{mi}) \,/\, (\$281{,}600/\text{mi}) \\
&= \mathit{(\$174{,}907/km + \$834/km)\,/\,(\$174{,}907/km)} \\
&= 1.005, \text{ or } 100.5 \text{ percent}
\end{aligned}$$

where:

$$\begin{aligned}
\text{Diff} &= LCC_{des} - LCC_{con} \\
&= 69{,}694 - 66{,}271 = \$1{,}342/\text{mi } (\$834/\text{km}) \\
\text{Bid Price} &= \text{Contractor's bid price for the lot (this is computed using the} \\
&\ \text{contractor's unit bid price } [\$/\text{yd}^2] \text{ times the yd}^2 \text{ of the lot)} \\
&= \$20.00/\text{yd}^2 * 14{,}080\ \text{yd}^2/\text{mi} = \$281{,}600/\text{mi } (\$174{,}907/\text{km})
\end{aligned}$$

Therefore, a pay factor of 100.5 percent and a percent defective of 26.54 provide one such point for the plot. The pay factor and percent defective computations are performed for each lot simulation until a sufficient number of data points exist to establish a pay factor equation (similar to that shown in figure 2 in chapter 2). A linear relation of the pay factor as a function of the percent defective is established over a portion of the curve from 0 to 90 percent defective using the least squares regression in PaveSpec, and is given in equation 14. The relationship obtained for this example is:

$$\text{Pay Factor} = 102.0 - 0.041 * (\text{ Percent Defective} < 90\ ) \tag{14}$$

Step 6. Calculation of the Lot Pay Factor and Adjusted Bid Price.

Applying the previously determined pay factor equation (equation 14), the pay factor is determined as:

$$PF = 102.0 - 0.041 * 26.54 = 100.91$$

An examination of the quality characteristic example results shows that the contractor met the as-designed target values for both the means and standard deviations. That is why the pay factor is so close to 100 percent.

The total payment to the contractor for the 0.81 km (0.5 mi) lot is then equal to the following:

Contractor Lot Payment	=	Lot Bid Price * Pay Factor / 100
	=	$281,600/2 * 1.009
	=	$142,067 per 0.81-km (0.5-mi) lot

Step 7. Development of Expected Pay Curve and Sample Size.

To evaluate the suitability of this acceptance plan, it is customary to construct an OC curve for the specification. Since this specification has pay adjustments, the construction of the conventional OC curve requires some modification. As previously described, a plot of the expected pay (EP) versus the percent defective is analogous to the OC curve when pay adjustments are used. An EP curve was constructed for the specific case study project and acceptance plan using PaveSpec simulation as shown in figure 11. This plot shows the mean expected pay factor versus the percent defective of the LCC_{con} as computed using PaveSpec. Each point is the mean of 10 simulated lots, and thus there is some scatter in the results. The plot shows that, on average, when the lot is approximately 50 percent defective, the mean pay factor is 100 percent, which is exactly what it should be. If the lot is 25 percent defective, the mean pay factor is between 102 and 103 percent, and if the lot is 75 percent defective, the mean pay factor is 95 percent. While a pavement having a percent defective LCC of 75 percent appears to indicate a very poor pavement, this is not necessarily the case, as will be shown in the sensitivity analysis.

Another part of an OC curve for a specification with a pay adjustment is the plot of the probability of acceptance (defined as receiving 100 percent pay or more) versus the percent defective. This simulation was carried out using PaveSpec and is shown in figure 12. This plot shows that if the as-constructed lot is 50 percent deficient, the probability of receiving at least 100 percent pay is 50 percent. As the percent defective of the lot decreases, the probability of at least 100 percent pay increases until it approaches 100 percent. Each point on the graph represents the results from 20 simulated lots, where the percent defective and probability of acceptance was computed from the 20 lots. The number of samples per sublot were varied from one to four, but did not appear to have any effect on the relationship. The sample size per sublot does not seem to have any effect on the probability of the pay factor being greater than 100 percent.

The effect of the number of samples per sublot can be seen in figure 13. This figure plots the standard deviation of the pay factor versus the number of samples per sublot. The number of sublots for all of these simulations is five for the case

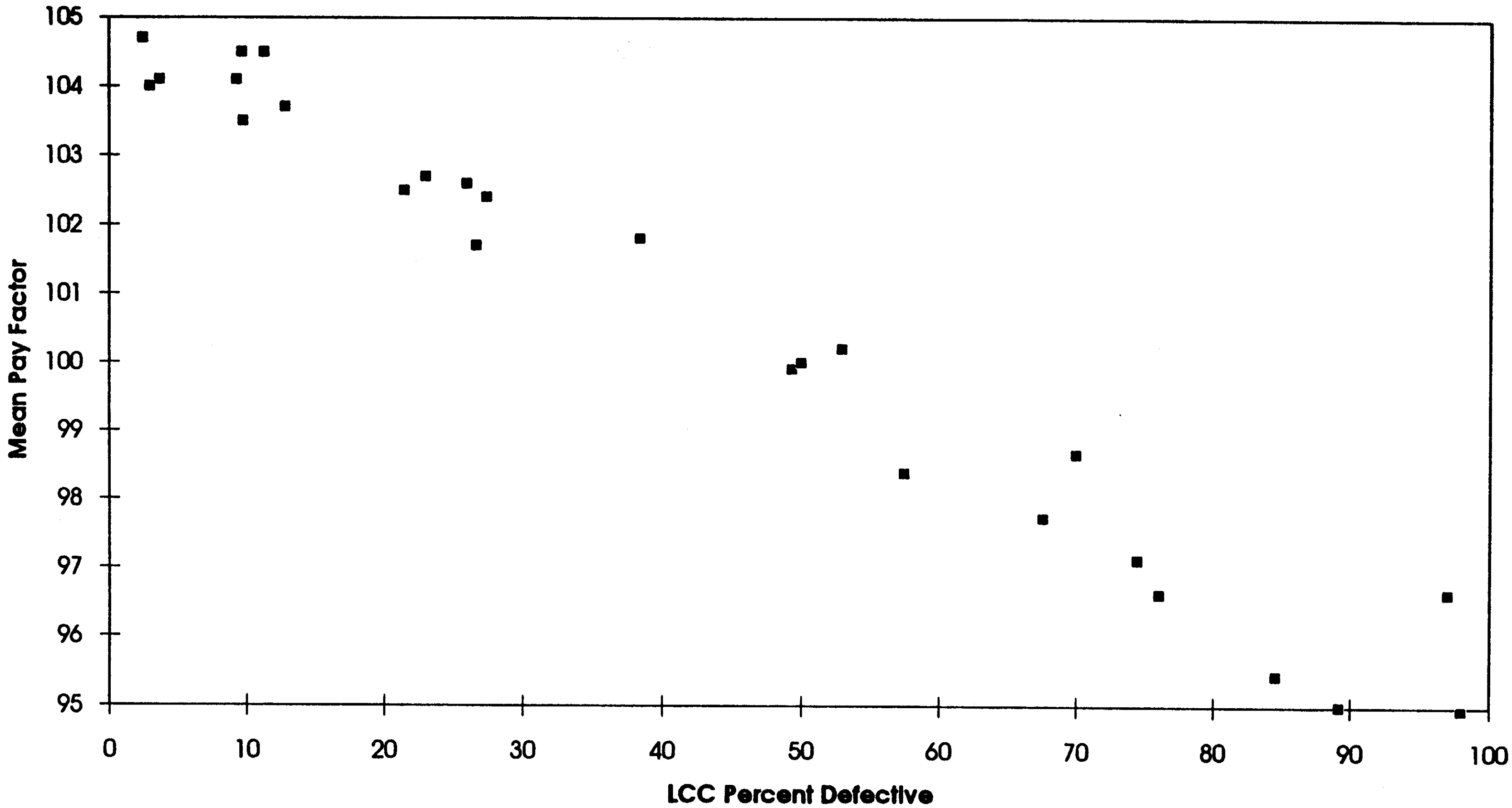


Figure 11. Expected pay curve for example project obtained from simulation using PaveSpec (5 sublots, 2 samples per sublot, and each point represents the mean of 10 simulated lots).

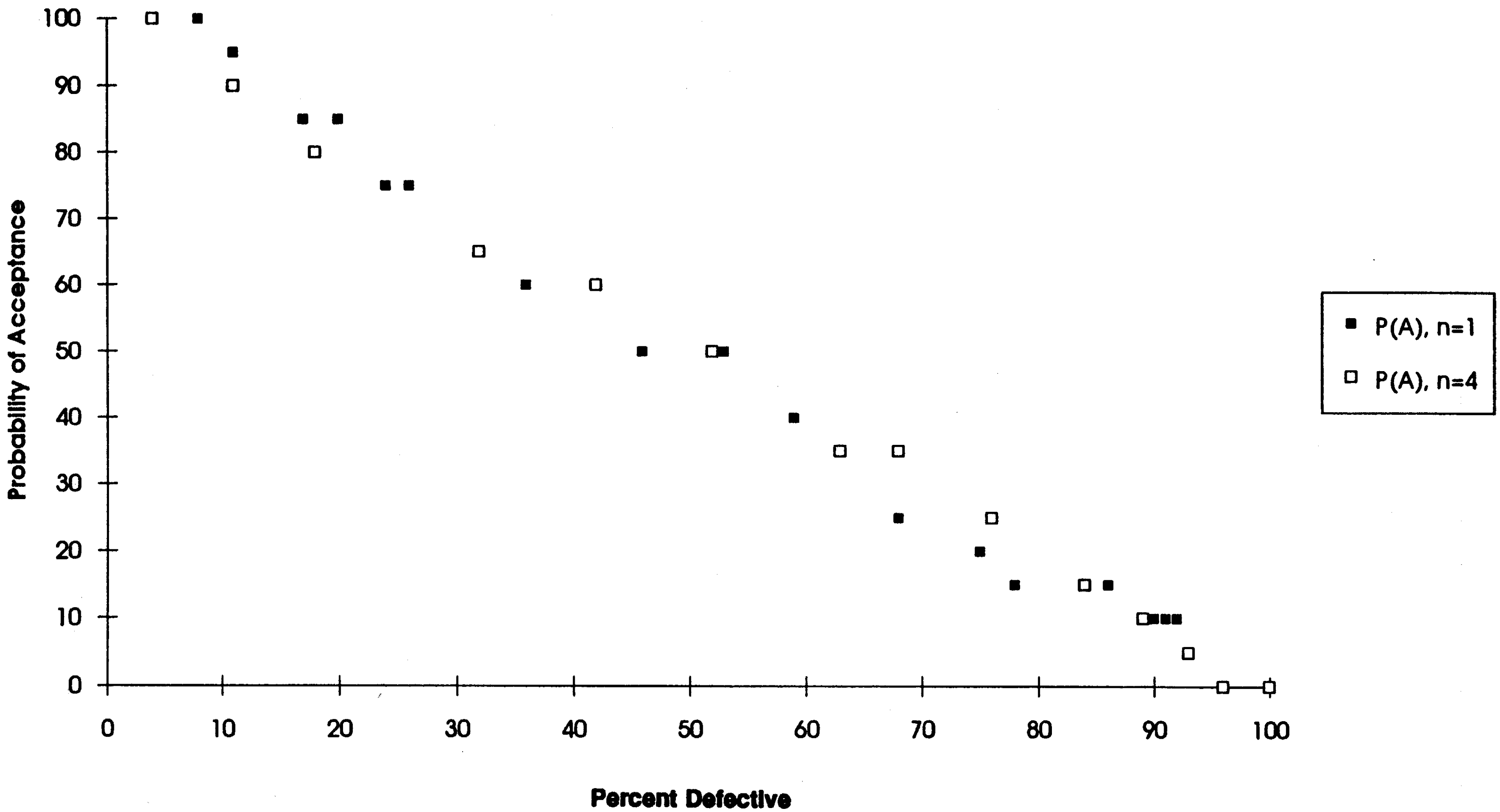


Figure 12. Simulation of probability of acceptance (100-percent pay factor or greater) and percent defective for case study (5 sublots per lot, and each point represents the mean of 20 simulated lots).

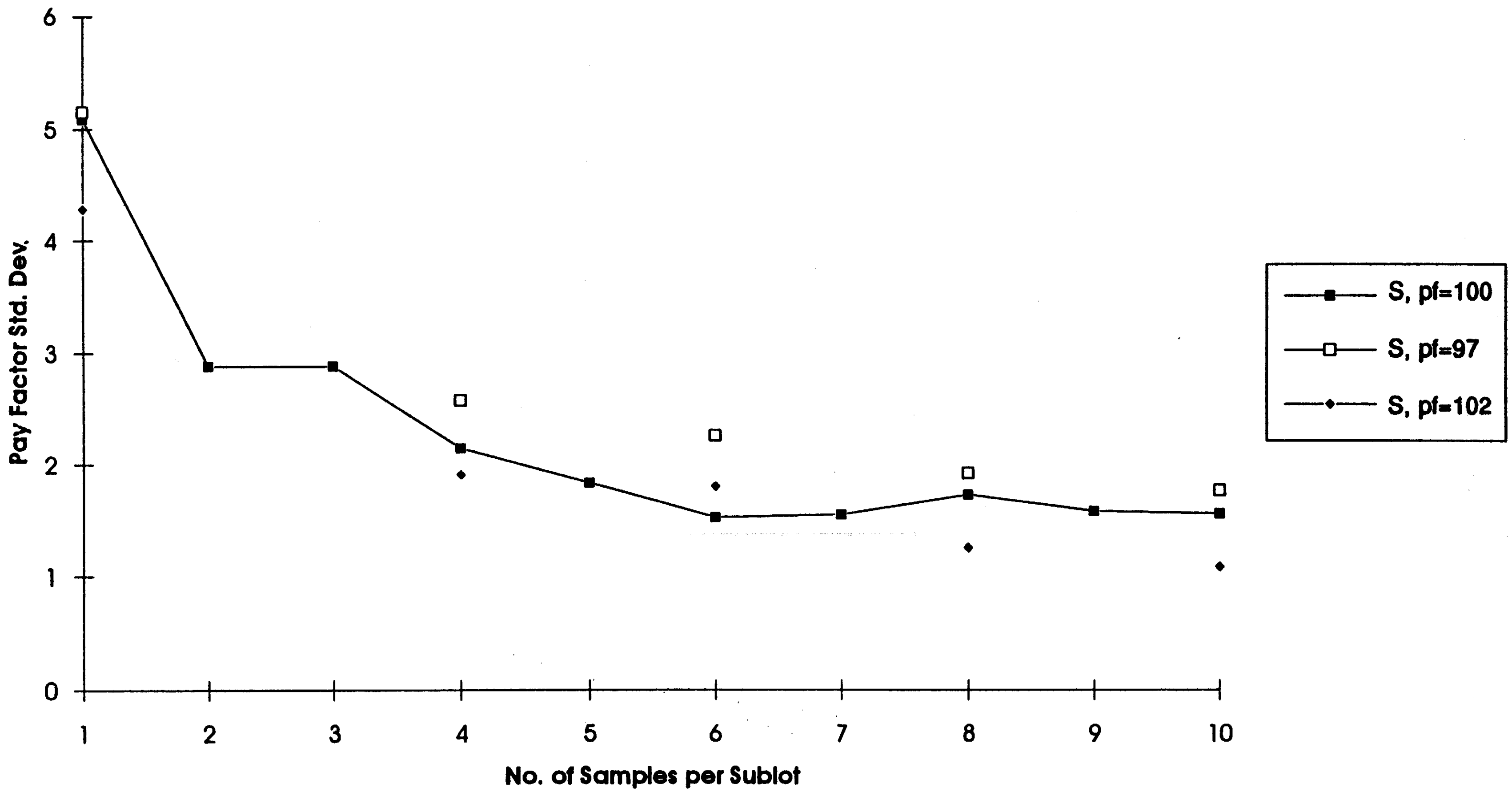


Figure 13. Simulation of pay factor standard deviation for varying numbers of samples per sublots using PaveSpec (each point represents 100 simulated lots; different symbols show different mean pay factors).

study project. Each point in figure 13 was obtained from simulation of 100 lots and the mean and standard deviation of the pay factor was computed. These results show that there is a large decrease in the standard deviation of the pay factor when two or more samples are taken per sublot. This reduction is quite significant in that the standard deviation of the pay factor decreases from about 5 percent to 3 percent. For a specific lot that was to be tested and the pay factor estimated, the risk of computing a pay factor far from the actual quality of the lot is greatly reduced by taking at least two samples per sublot. Further research is needed into determining the risks involved by the agency and contractor when using a pay adjustment schedule.

Step 8. Sensitivity Analysis of Quality Characteristics.

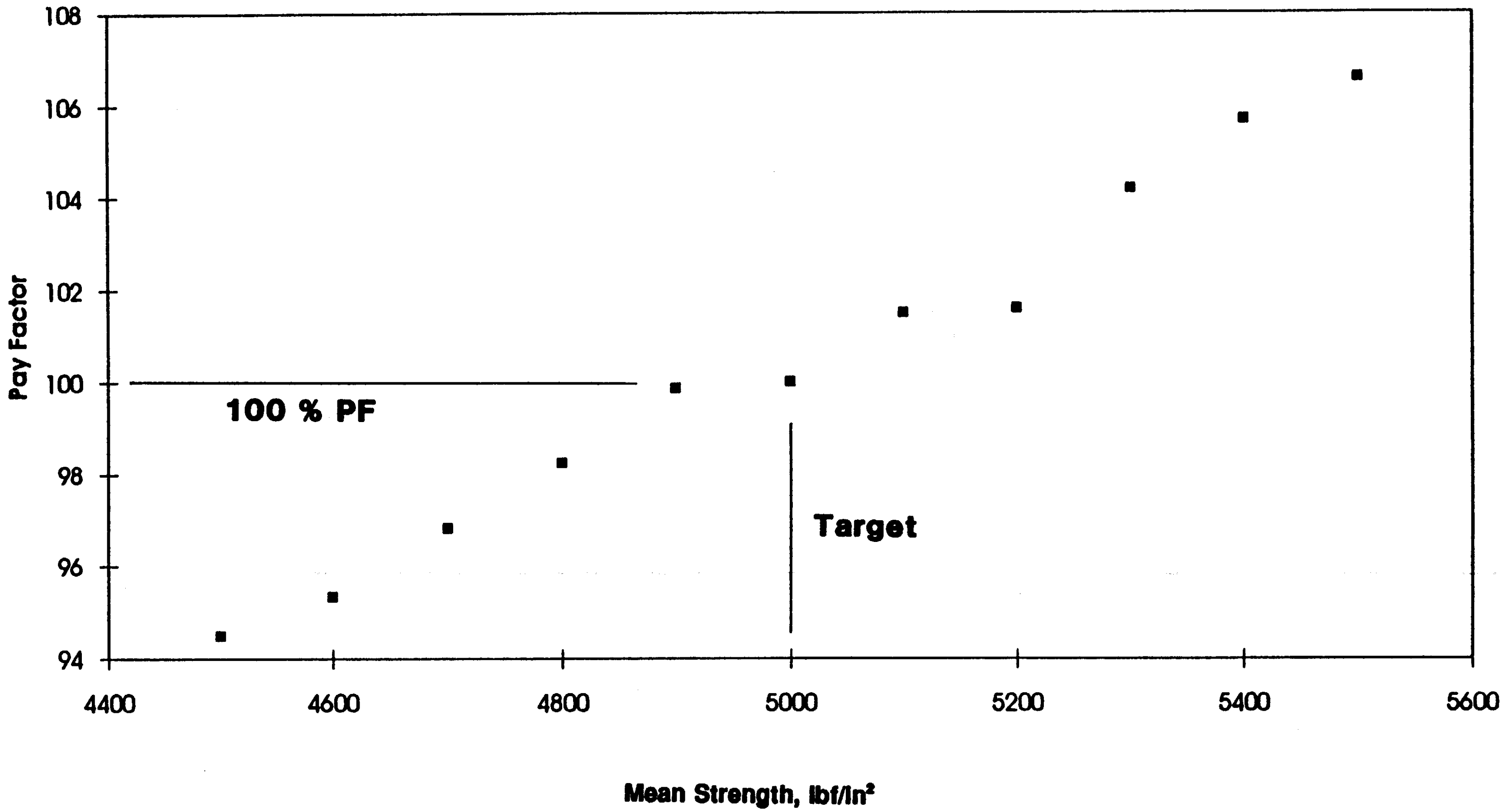
A sensitivity analysis is conducted for the example project to illustrate the general effects of each of the quality characteristics and their variations. The PaveSpec program can be utilized to illustrate the significance of each quality characteristic on the resulting pay factor.

The PaveSpec program is used to simulate the as-constructed lots where the population means of strength, thickness, air content, and initial roughness were varied over a typical range. The mean pay factor is computed for each run and then plotted as shown in figures 14, 15, 16, and 17. These graphs show that each of the four quality characteristics have a fairly significant effect on the pay factor. Table 7 summarizes the results.

The PaveSpec program is then used to simulate as-constructed lots where the population coefficients of variation of strength, thickness, air content, and initial roughness were varied over a typical range. The mean pay factor is computed for each run and then plotted as shown in figures 18, 19, 20, and 21. These graphs show that the variation of each of the four quality characteristics have a fairly significant effect on the pay factor. Table 8 summarizes the results.

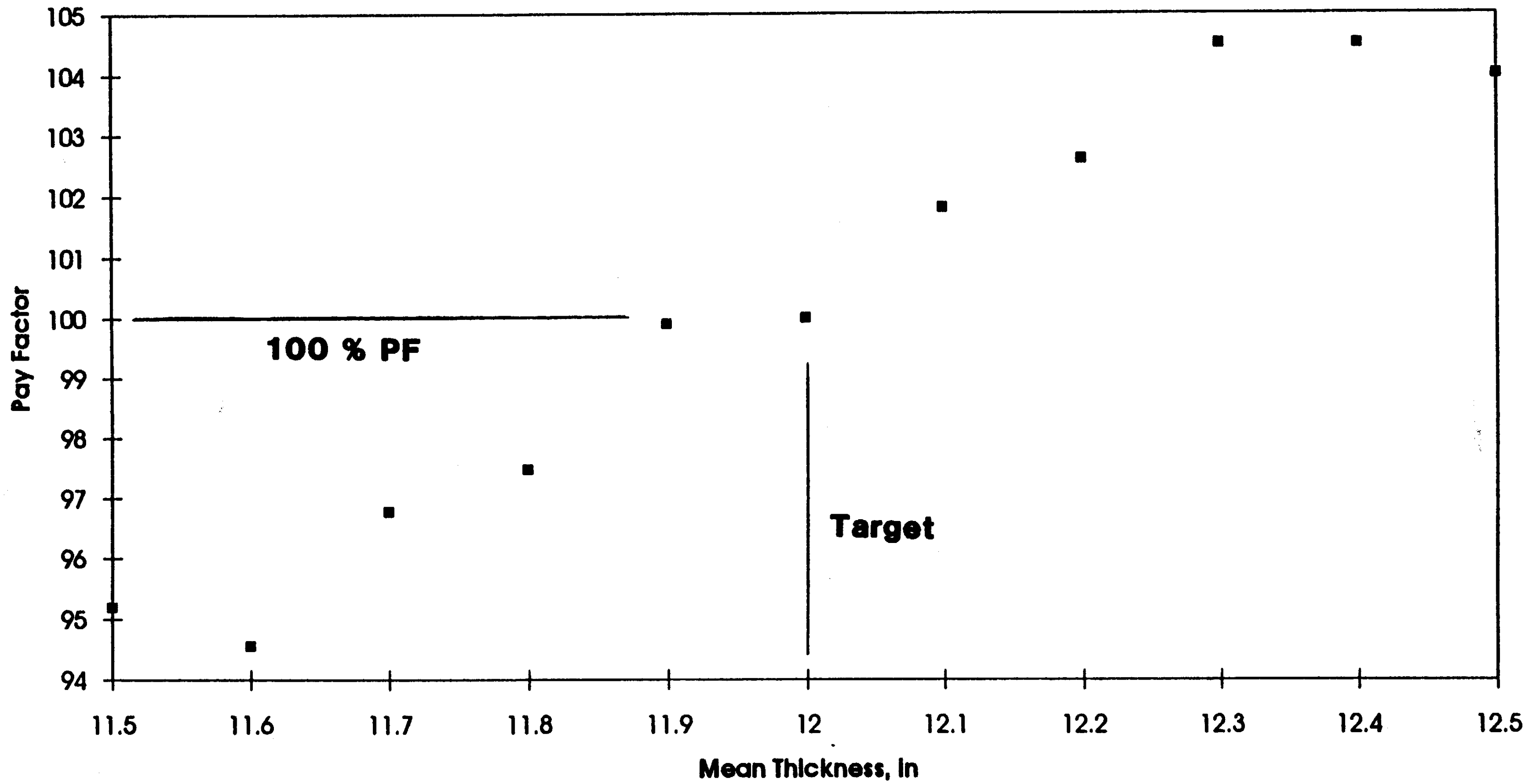
There are at least two important implications of these results. One is that they demonstrate the ability to utilize the PRS technology to show the effects of different levels of quality on the resulting costs and performance of the pavement. The shapes of the curves can be used as an indication of optimum levels of quality for each quality characteristic. The importance of this ability cannot be overstated, because this allows the level of quality (pavement performance) to be quantified in terms of the target quality characteristics.

Another implication of these results is that they show that the prototype PRS provides strong incentive to the contractor to produce high quality construction, not only from the standpoint of the means of the quality characteristics (i.e., increase in strength and thickness and decrease in initial roughness), but also from the standpoint of producing a more uniform pavement lot. A lower construction variation in strength, thickness, air content, and initial roughness all result in increased pay factors.



1000 lbf/in² = 6.9 MPa

Figure 14. Mean lot pay factor vs. mean concrete compressive strength of lot for example project (each point represents mean of 10 lots).



1 in = 25.4 mm

Figure 15. Mean lot pay factor vs. mean concrete slab thickness of lot for example project (each point represents mean of 10 lots).

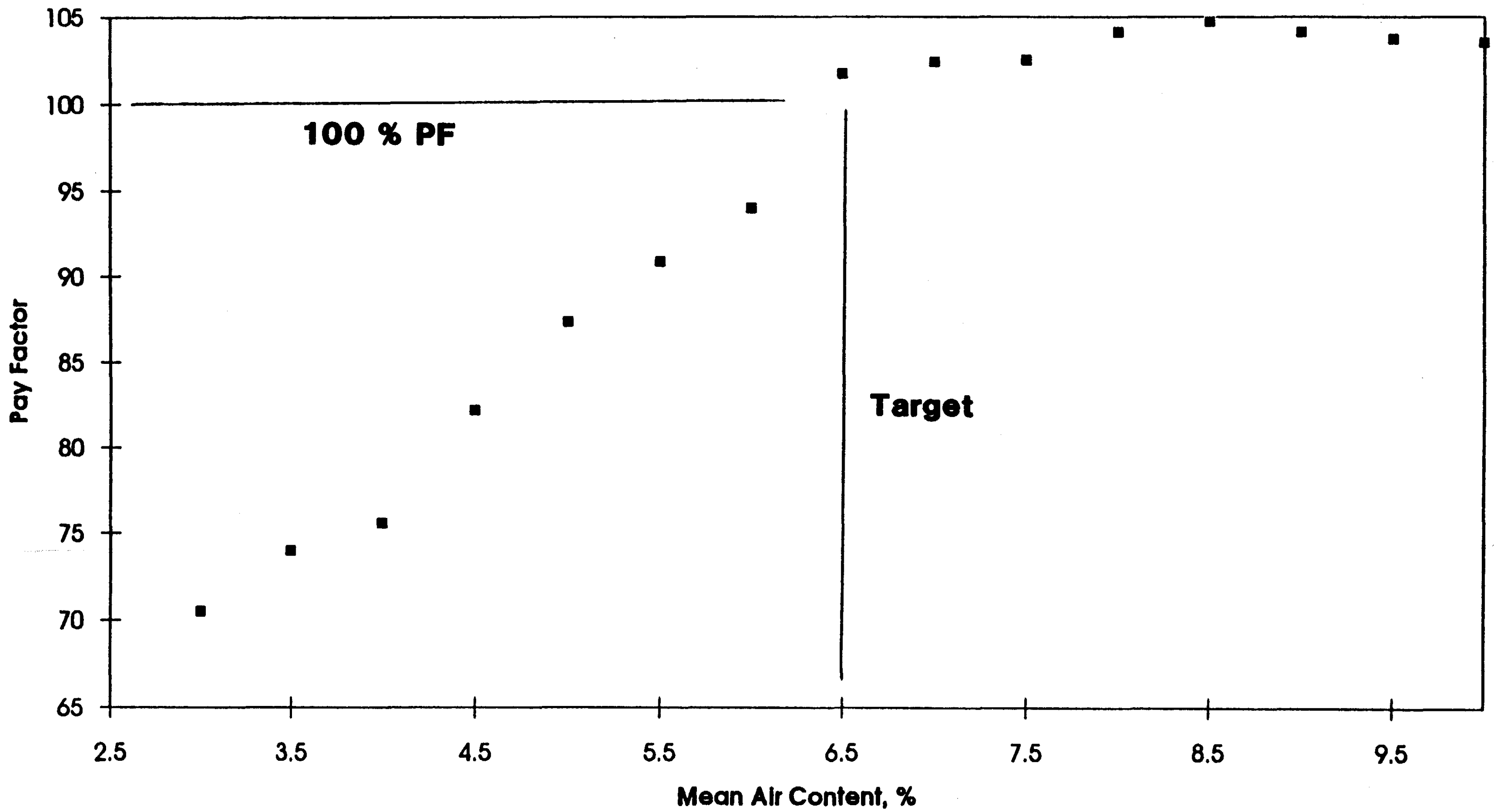
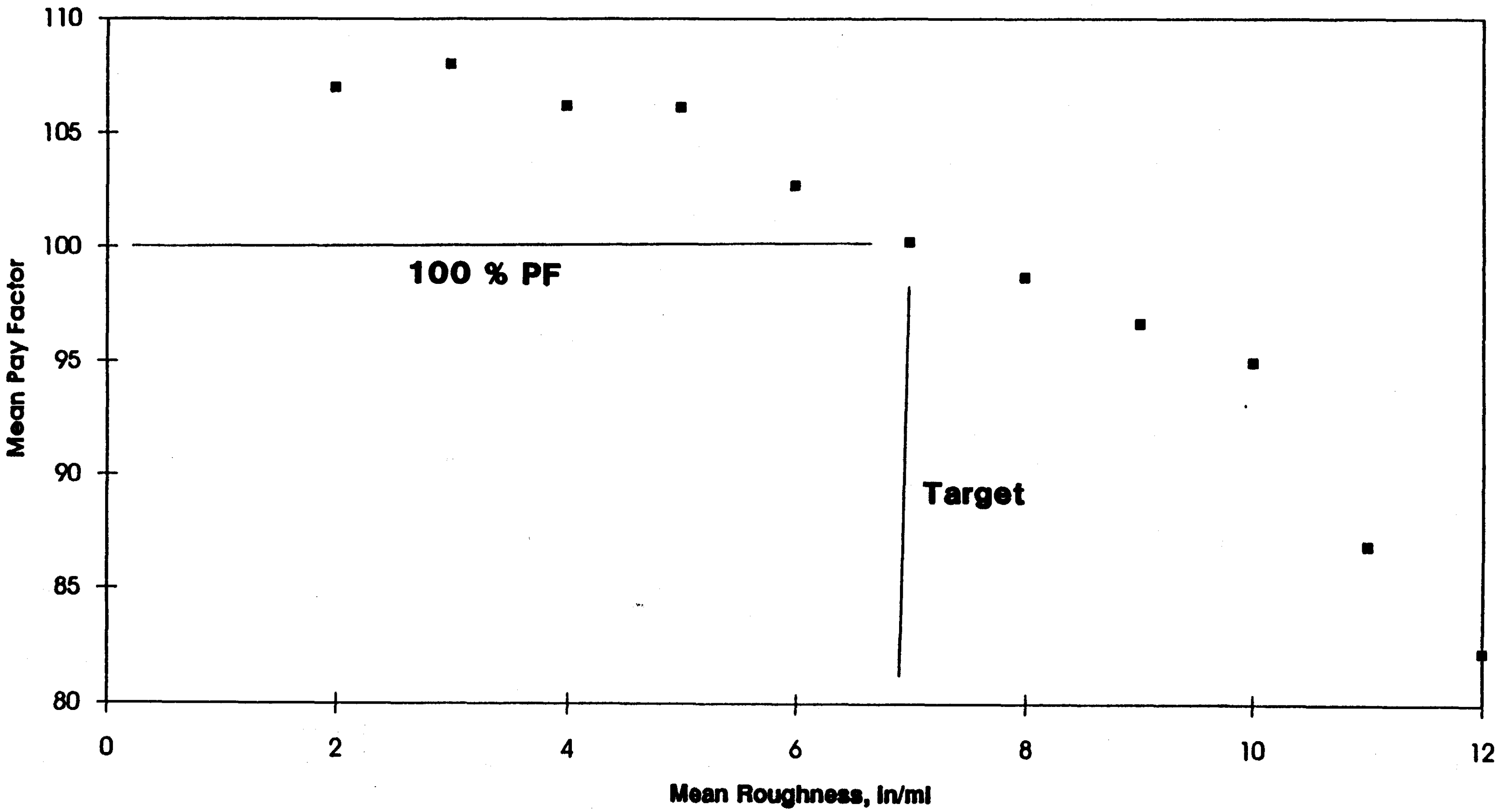


Figure 16. Mean lot pay factor vs. mean concrete air content of lot for example project (each point represents mean of 10 lots).



1 in/mi = 0.016 m/km

Figure 17. Mean lot pay factor vs. mean initial roughness of lot for example project (each point represents mean of 10 lots).

Table 7. Approximate pay factor ranges for mean quality characteristic values.

Quality Characteristic	Range of Mean Values	Pay Factor
Strength, lbf/in^2	4500	94
	5000	100
	5500	107
Thickness, in	11.5	95
	12.0	100
	12.5	104
Air Content, %	2.5	65
	6.5	100
	9.5	104
Roughness, in/mi	12	82
	7	100
	2	108

1 in = 25.4 mm
1000 lbf/in^2 = 6.9 MPa
1 in/mi = 0.016 m/km

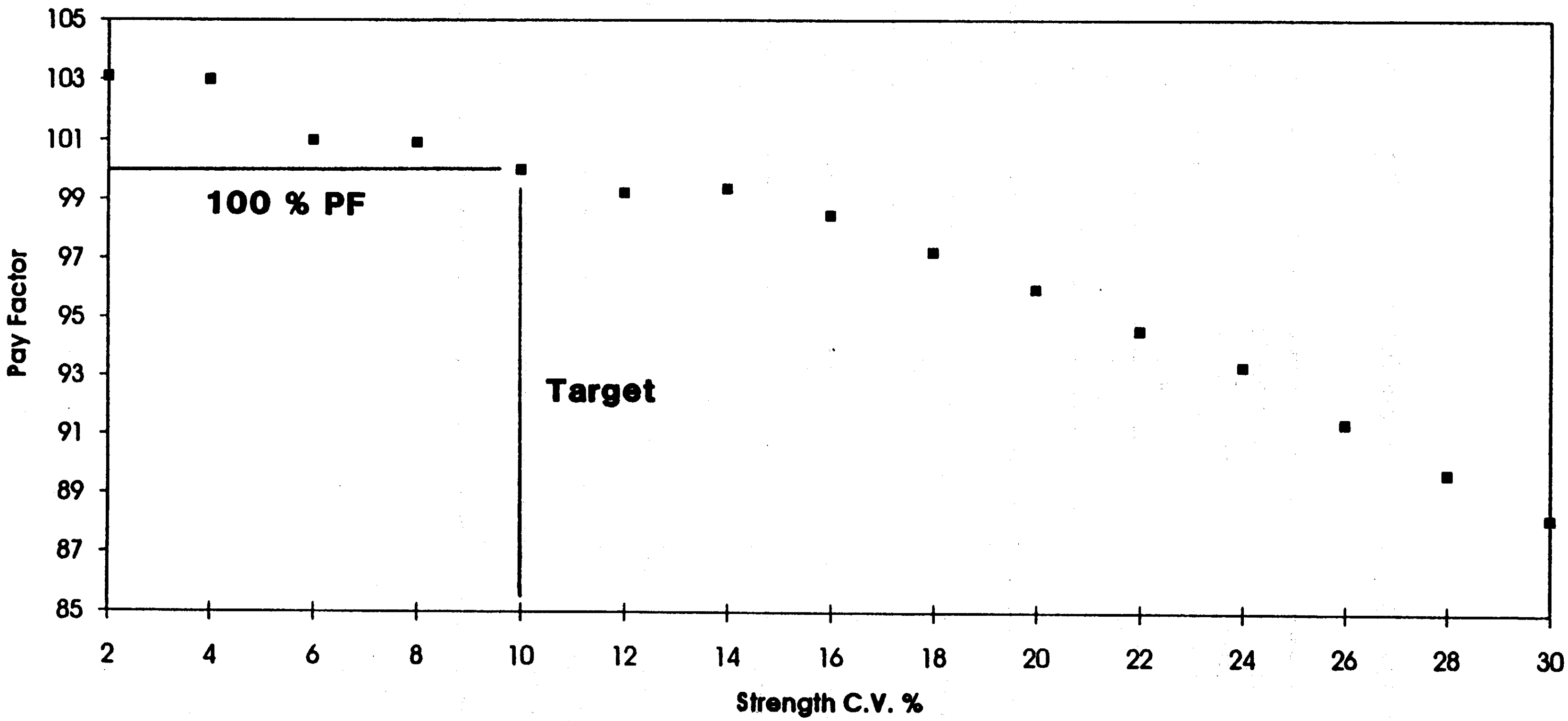


Figure 18. Mean lot pay factor vs. coefficient of variation of concrete compressive strength of lot for example project (each point represents mean of 10 lots).

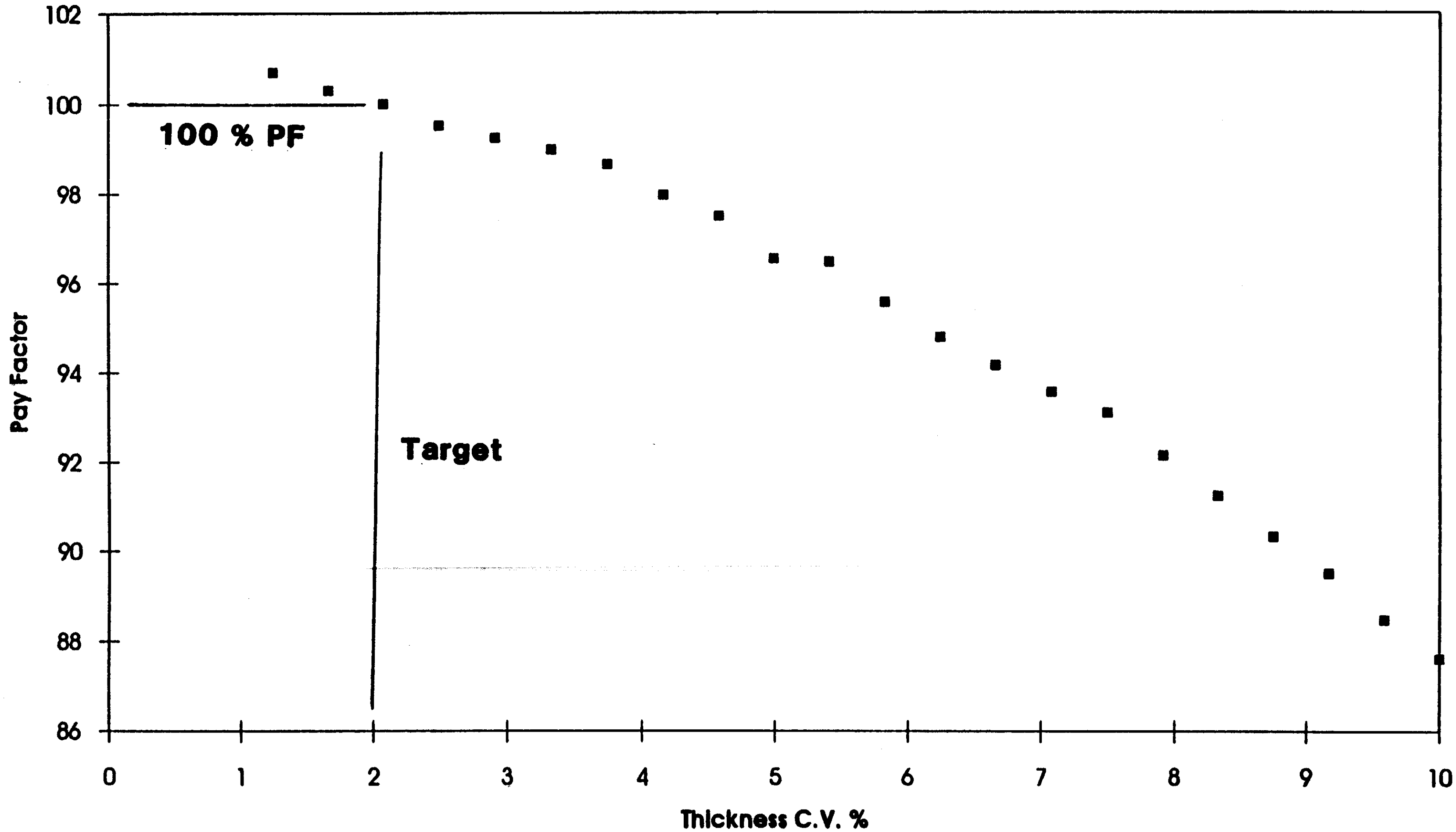


Figure 19. Mean lot pay factor vs. coefficient of variation of slab thickness of lot for example project (each point represents mean of 10 lots).

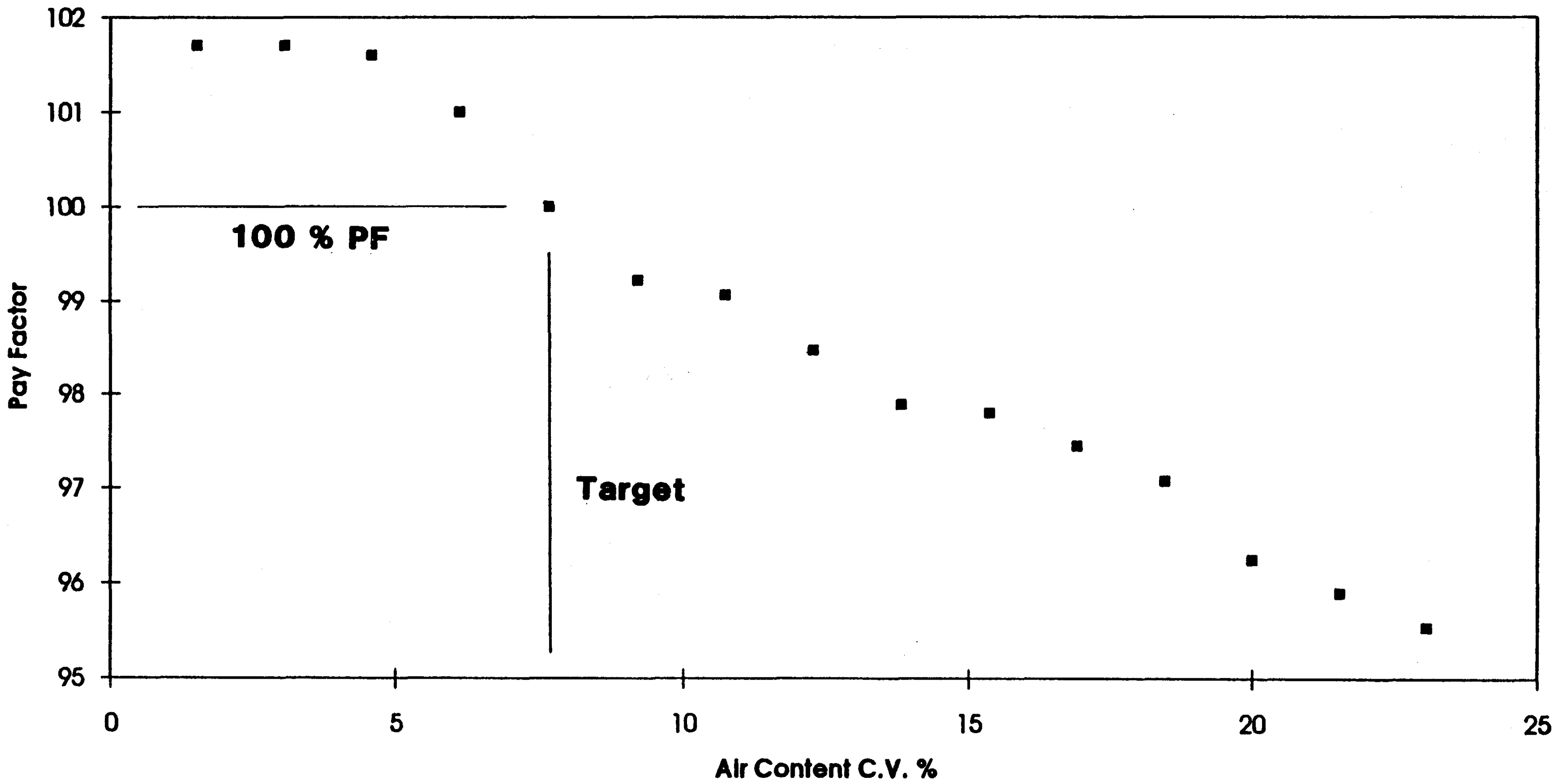


Figure 20. Mean lot pay factor vs. coefficient of variation of concrete air content of lot for example project (each point represents mean of 10 lots).

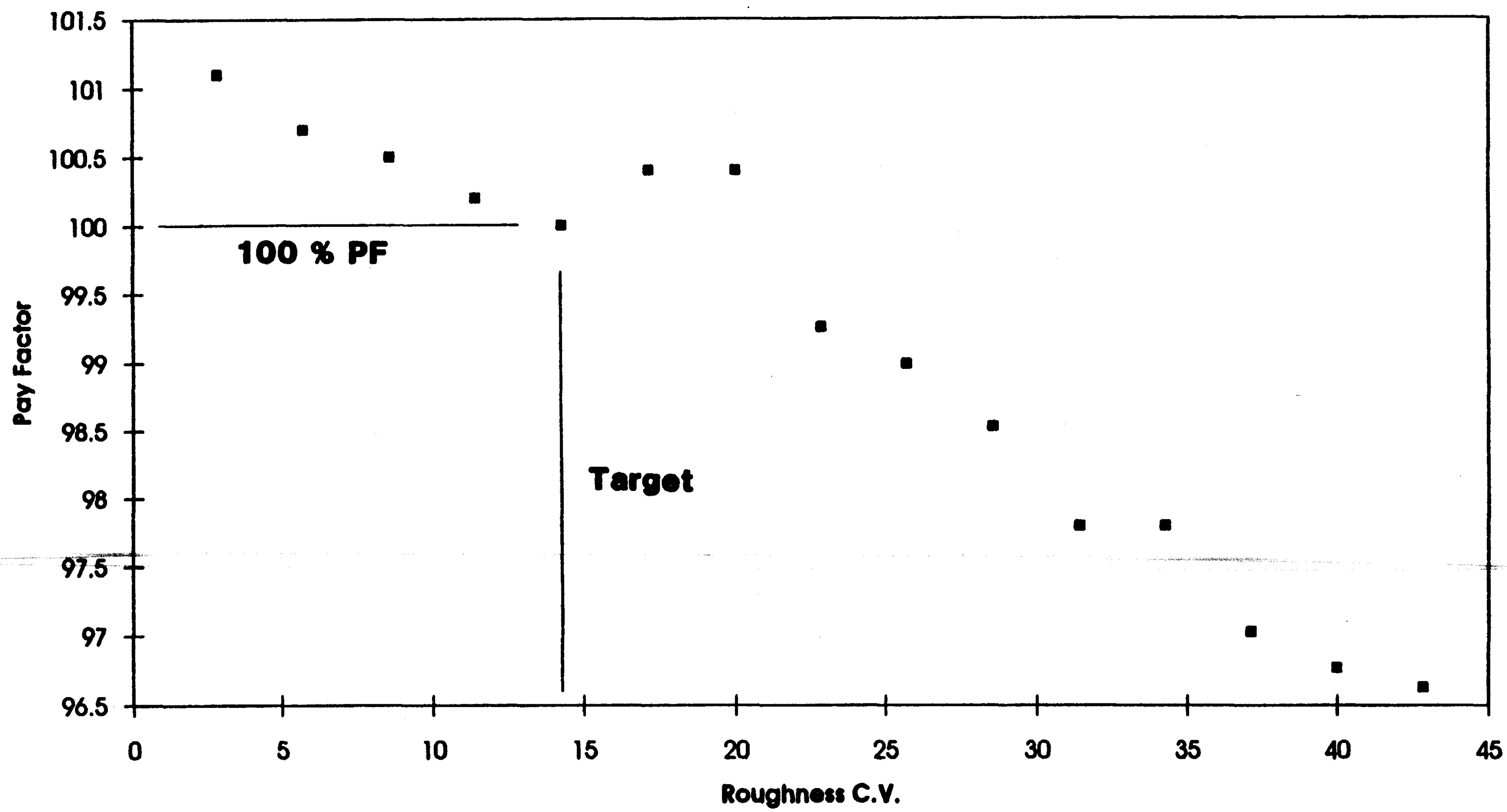


Figure 21. Mean lot pay factor vs. coefficient of variation of initial roughness of lot for example project (each point represents mean of 10 lots).

Table 8. Approximate pay factor ranges for coefficient of variation values for quality characteristics.

Quality Characteristic	Range of COV Values	Pay Factor
Strength, lbf/in^2	20	96
	10	100
	2	103
Thickness, in	8	92
	2	100
	1	101
Air Content, %	18	97
	8	100
	2	102
Roughness, in/mi	30	98
	14	100
	2	101

1 in = 25.4 mm
1000 lbf/in^2 = 6.9 MPa
1 in/mi = 0.016 m/km

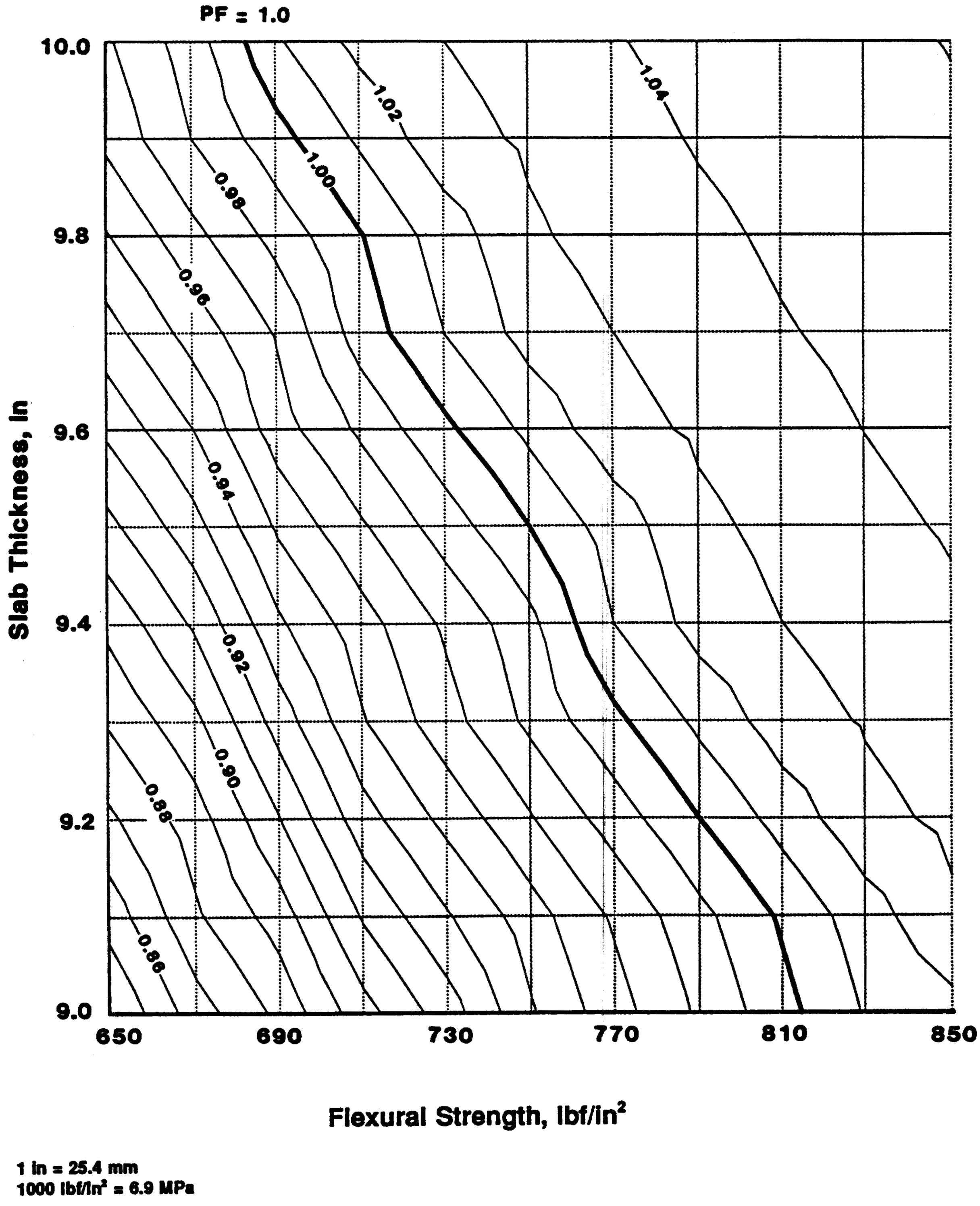


Figure 22. Contour plot showing pay factor as function of concrete strength and slab thickness.

A final example of the type of sensitivity analysis that can be conducted with the PaveSpec software is shown in figure 22. This figure presents a contour plot showing how changes in strength and thickness can still result in a pay factor of 100 percent (holding air content and roughness constant). This figure was created for a target thickness of 241 mm (9.5 in) and a target strength of 4.9 MPa (750 lbf/in^2). This information can be very useful to contractors in assessing the effects of "trade-offs" between quality characteristics.

SUMMARY

This chapter presents an example of how the prototype PRS may be applied to an actual construction project. LCC computations for the as-designed and as-constructed pavements are calculated, resulting in the generation of the overall pay factor. A sensitivity analysis of the four quality characteristics on the pay factor is presented.

The prototype PRS developed in this study is a complicated procedure that requires numerous inputs, many of which are currently difficult to estimate for a given contract. In addition, the PRS is based on estimating the LCC of a pavement which in itself has several advantages and disadvantages.

The main advantage of the LCC approach is that it is possible to realistically consider any number of quality characteristics (both means and variations) in the rational calculation of a single pay factor for the lot. This approach can be extended to include all aspects of the pavement/subgrade system. There are no judgments required as to how to combine several different pay factors into a single pay factor for the lot.

The primary disadvantage of this approach is that the calculation procedures for LCC are very controversial, and of themselves raise many questions. In addition, the computation of LCC for a lot that included variation of quality characteristics is a very difficult technical problem that is only solved approximately in the prototype specification.

The implementation of this prototype PRS will require further testing and evaluation of the technical and practical aspects of the specification. Further sensitivity and evaluation of the prototype PRS and the PaveSpec computer program may show that it can be simplified, without great loss of accuracy, to make it far easier to use in the field. This would involve the development of pay factor equations through regression analyses based upon many runs of PaveSpec for a range of project conditions. Future work should focus on this important aspect of PRS implementation.

CHAPTER 4. LABORATORY TESTING

INTRODUCTION

This chapter provides details of the experimental designs, implementation procedures, and analyses of the laboratory materials study that was conducted in support of the development of a prototype PRS for concrete pavements. Laboratory testing was conducted to investigate key relationships between concrete material quality characteristics and two pavement distress indicators:

- Transverse cracking caused by repeated loading and thermal curling.
- Joint spalling caused by an inadequate air-void system.

Transverse cracking of concrete slabs is the result of many factors, including late joint sawing, subbase restraint to concrete shrinkage, inadequate base support, subbase restraint stress due to excessive joint spacing, inadequate flexural strength, repeated traffic loading, and thermal curling. The combination of the latter three factors was evaluated in the laboratory investigation. The material quality characteristics of interest are the concrete flexural strength and the concrete elastic modulus. Several prediction models currently exist that may be used to estimate transverse cracking, and all consider the concrete flexural strength and the concrete modulus of elasticity.

Joint spalling can be attributed to repeated traffic loading, misalignment of dowel bars, the presence of incompressibles in the joint, and an inadequate air-void system. These four causes can generally be classified, respectively, into traffic, construction, maintenance, and material categories. Of these, only dowel misalignment and an inadequate air-void system are under the control of the contractor. To address durability problems in the development of PRS, a laboratory testing program was conducted to correlate air-void system parameters with spalling. Although the occurrence of an inadequate air-void system may be infrequent, it may significantly affect pavement service life in areas with a large number of annual freeze-thaw (F-T) cycles. It was assumed in the laboratory investigation that the degree of spalling will directly influence pavement performance.

LABORATORY EVALUATION OF CONCRETE STRENGTH/STIFFNESS PARAMETERS

The first part of the laboratory materials study investigated factors that affect concrete strength and modulus of elasticity. These are factors that are under the control of the contractor and can significantly influence concrete pavement performance in terms of the development of transverse cracking. Several additional laboratory materials studies were conducted to examine other factors of interest in the development of PRS. The objectives of this part of the laboratory testing program were to:

- Investigate influences and significance of controlled mix design variables (such as air content, cement content, and aggregate type) on strength and elastic modulus.

- Establish estimates of within-test variability for strength and elastic modulus testing. This aided in determining the frequency of sampling and the required number of specimens to characterize a material lot.

- Develop relationships between different strength types (compressive, flexural, and splitting tensile). Mix design parameters and curing ages were varied to determine and demonstrate their effects on interstrength relationships.

- Develop relationships between elastic modulus and strength types. This demonstrated how strength monitoring can be used to determine the concrete elastic modulus.

- Investigate the appropriateness of ultrasonic pulse velocity and maturity nondestructive testing (NDT) methods to demonstrate their applicability and feasibility in monitoring material responses. Rapid test methods must be available in PRS schemes to allow for a quick assessment of material properties so that a contractor can make any needed corrections.

- Develop strength relationships between 102-mm (4-in) diameter cores and 152-mm (6-in) diameter cylinders cured under identical (maturity) conditions. These factors were investigated to evaluate actual material responses with regard to the type of specimen used in a PRS.

- Investigate the effects of consolidation level on concrete strength. Specimens were fabricated at several consolidation levels (densities) and tested to determine its effect on strength.

The experimental designs, implementation procedures, and analyses of the laboratory materials study for use in the PRS are summarized in this chapter. The development of the laboratory testing program is described in more detail in appendix D.

Design Variables for Laboratory Study

Given the prioritization of the variables as discussed in appendix D, the following variables were considered in the expanded laboratory investigation:

1. Coarse Aggregate Hardness (CAH) — two levels (0=soft, 1=hard).
2. Coarse Aggregate Geometry (CAG) — two levels (0=rounded, 1=crushed).
3. Coarse Aggregate Maximum Size (CAM) — one level (25 mm [1 in]).
4. Fine Aggregate Fineness Modulus (FAM) — one level (FM = 2.68).
5. Air Content (AIR) — two levels (5 and 7 percent, ± 0.5 percent).
6. Coarse Aggregate Volume Percentage (CAP) — two levels (35 and 41 percent).

7. Cement Volume Percentage (CEP) — two levels (9.3 and 11.1 percent).
8. Water Volume Percentage (WAP) — one level (14.1 percent).
9. Fine Aggregate Type (FAT) — one level (quartzitic sand).
10. Consolidation Level (CSL) — three levels (100, 97, and 94 percent).
11. Mineral Admixture (Fly Ash) — one level (none).
12. Cement Type (CET) — one level (type I).
13. High-Range Water Reducer (HRWR) — one level (none).

Aggregates were obtained from sources approved for use in highway pavement construction. Types of coarse aggregate and their sources are:

- Crushed-hard (CH) — Crushed quartzite from the Sioux Falls, SD area.

- Crushed-soft (CS) — Dolomitic carbonate from the Chicago, IL area.

- Round-hard (RH) — Siliceous (granitic and volcanic) river gravel from the Eau Claire, WI area.

- Round-soft (RS) — Glacial gravel from the Elgin, IL area.

All aggregates were classified as innocuous or mildly deleterious for potential of alkali-silica reactivity (ASR). Coarse aggregate properties and gradations are summarized in tables 1 and 2 of appendix D. The course aggregate volume percentages (CAP) were fixed at 35 and 41 percent. This corresponds to approximately 920 to 943 kg/m^3 (1550 to 1590 lb/yd^3) for 35-percent coarse aggregate by volume and 1080 to 1103 kg/m^3 (1820 to 1860 lb/yd^3) for 41-percent coarse aggregate by volume. These aggregate quantities are in the range normally used for highway concrete pavement construction.

The fine aggregate was a natural sand (composed of varying amounts of quartz, quartzite, feldspar, and other siliceous particles) with a fineness modulus of 2.68. The fine aggregate came from a Chicago-area glacial source approved for Illinois pavement construction. Fine aggregate properties and gradation are summarized in tables 1 and 3 of appendix D.

Two water-cement ratios (WC) were selected: 0.40 and 0.48. These values correspond to cement volumes (CEP) of 9.3 and 11.1 percent, respectively, and a water volume (WAP) of 14.1 percent. Cement contents were 292 kg/m^3 (492 lb/yd^3) and 350 kg/m^3 (590 lb/yd^3), values typically used for highway pavement construction.

To obtain constant levels of air content for each mix, several trials at various dosage levels of an air entraining admixture were required. A commercially available air entraining admixture was used in the laboratory study. Two levels of air content (AIR), 5 and 7 percent ($\pm$ 0.5 percent), were evaluated. Air content was measured in the plastic state after batching using a pressure air meter similar to those used in highway pavement construction quality control.

Three levels of consolidation (94, 97, and 100 percent) were selected to evaluate the effects of consolidation on compressive strength, modulus of rupture (flexural strength), splitting tensile strength, and static elastic modulus. Fully consolidated specimens were made using external vibration supplied by a vibrating table. Specimens at the other consolidation levels were made by rodding specimens with 97 and 94 percent of the material weight of fully consolidated specimens.

These variables were investigated in separate laboratory investigations geared to filling specific gaps that existed in the current specifications. Each of these separate investigations are discussed in later sections.

Laboratory Mixing and Testing Procedure

The coarse aggregates were presoaked a minimum of 16 h prior to mixing to provide a saturated surface dry (SSD) aggregate. Fine aggregates were kept in a moist condition throughout the testing program. All mixes were made at SSD conditions to minimize the effects of coarse aggregate absorption (if less than SSD), which is important since mixing times are relatively short for small batches. To minimize the effects of different initial concrete temperatures, aggregates were conditioned at 22 °C ± 1.1 °C (72 °F ± 2 °F) for a minimum of 16 h in a temperature-controlled laboratory.

The mixes were produced in a random order to minimize any time-series effects, such as systematic variation in air temperatures or equipment wear. Initial concrete temperatures were determined and used to ensure that no systematic sources of error were present that may affect concrete strength. Air entrainer dosages were varied and air contents measured to ensure targeted air contents were achieved.

To minimize the effects of time during fabrication of the test specimens, several small (0.04-m^3 [1.5-ft^3]) batches were made for each mix. Aggregates and cement were mixed for 2 min, water was added and mixed for an additional 2 min, and then the mixture was covered for 3 min prior to performing concrete tests and specimen fabrication. Fully consolidated specimens were consolidated using procedures in accordance with ASTM Designation C192, "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory." This consisted of consolidation by external vibration using a table vibrator with a frequency of 7000 vibrations per min (117 Hz) and an amplitude of 0.1 mm (0.004 in). Specimens at other consolidation levels were made by rodding specimens with 97 and 94 percent of the material weight of the fully consolidated specimens. Care was taken to ensure material density uniformity throughout the specimen.

After concrete initial set, specimens were covered with wet burlap and polyethylene and then left undisturbed in a controlled laboratory at 22 °C ± 1.1 °C (72 °F ± 2 °F) and 50 percent relative humidity (PRH) for 24 h. After 24 h, the specimens were demolded and stored in a fog room maintained at 22 °C ± 1.1 °C (72 °F ± 2 °F).

Plastic Concrete Tests

Several tests were run on batches in the plastic state, including the following:

- Slump, in accordance with ASTM Designation C143, "Standard Test Method for Slump of Hydraulic Cement Concrete."
- Unit weight, in accordance with ASTM Designation C138, "Standard Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete."
- Initial concrete temperature.
- Air content, in accordance with ASTM Designation C231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method."

Hardened Concrete Tests

Hardened concrete tests for each batch were made at 7, 14, and 28 d. Three specimens (triplicates) were tested at each age. Tests on hardened concrete included the following:

- Compressive strength of 152-mm by 305-mm (6-in by 12-in) cylinders in accordance with ASTM Designation C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens."
- Splitting tensile strength of 152-mm by 305-mm (6-in by 12-in) cylinders in accordance with ASTM Designation C496, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens."
- Third-point loading flexural strength (modulus of rupture) of 152-mm by 152-mm by 533-mm (6-in by 6-in by 21-in) long beams in accordance with ASTM Designation C78, "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)."
- Modulus of elasticity of 152-mm by 305-mm (6-in by 12-in) cylinders in accordance with ASTM Designation C469, "Standard Test Method for Static Elastic Modulus and Poisson's Ratio of Concrete in Compression."

Strength and Consolidation Study

Recent trends in concrete pavement construction have been toward earlier opening times to traffic, particularly with developments in fast track paving, faster production rates, and modified mixes. With the emphasis on quicker and easier determination of strength, the use of third- or center-point flexural strength testing is likely to decrease. Present indications are that most agencies will be adopting either the compressive or splitting tensile strength as the standard indicator of slab strength. Since the PRS models for strength require flexural strengths to evaluate fatigue damage (which is dependent upon the ratio of the flexural stress to the flexural strength), flexural strength was investigated. Flexural strength data were used to demonstrate relationships between compressive or splitting tensile and flexural strength. Highway agencies can then use the results to evaluate important mix variables affecting compressive or splitting tensile strength. By establishing mix-specific relationships using project materials, the mix variables affecting future performance may be inferred. Previous studies and data from the initial lab study indicate that reliable interstrength relationships can be established for project-specific mixes.

An experimental design was generated to evaluate the effects of coarse aggregate hardness (CAH, two levels), coarse aggregate geometry (CAG, two levels), air content (AIR, two levels), water-cement ratio (WC, two levels), and coarse aggregate volume percentage (CAP, two levels). The effects of consolidation were evaluated at three levels. To achieve this, a total number of 96 (2^5*3^1) mixes, as shown in table 9, would be required for a full factorial design. Using half-fraction factorial design principles, the number of mixes was reduced to 48. This allowed for maximum statistical information to be derived while reducing the size and cost of the experiment. The orthoganality of the experimental design, as illustrated in table 9, ensured that all main effects and two-factor interactions are additive and independent of one another. Four-factor effects are confounded with three-factor effects and could not be statistically differentiated. Similar to other designs of experimental studies, the contribution of three- and four-factor interaction effects is assumed to be much smaller than that of main and two-factor interaction effects. The benefits of evaluating more material variables are greater than the expected information gained from rigorous investigation of higher factor interaction effects.

Two of the 48 mixes made could not be consolidated at the 94-percent consolidation level. These two mixes were those containing the round-soft and round-hard coarse aggregates at the 41-percent (high) coarse aggregate percent level and 7-percent air content (high). The difficulty in fabricating these mixes was due to the significant settlement that occurred when specimens were fabricated at a level less than 97-percent consolidation.

Nine duplicate mixes of the primary laboratory mix designs were made to determine variances in hardened concrete strength both between and within mixes. Both of these variance sources represent chance variation or experimental error. The triplicate variance reflects differences due to variability in material, preparation, fabrication, curing, and testing. Differences between triplicate specimens reflect the effects of only fabrication and testing variances. The replication (between-mix) variance should be higher than the duplication (within-mix) variance. The analysis of the additional nine mixes was used as an indicator of whether significant mix variables or random chance is responsible for observed differences in strength or elastic modulus.

Table 9. Factorial design for concrete strength study.

			CONSOLIDATION LEVEL											
			94 Percent				97 Percent				100 Percent			
			Coarse Agg. Hardness				Coarse Agg. Hardness				Coarse Agg. Hardness			
			Soft		Hard		Soft		Hard		Soft		Hard	
			Coarse Agg. Geometry		Coarse Agg. Geometry		Coarse Agg. Geometry		Coarse Agg. Geometry		Coarse Agg. Geometry		Coarse Agg. Geometry	
AIR CONTENT	W/C Ratio	Coarse Agg. Quantity	Round	Crushed	Round	Crushed	Round	Crushed	Round	Crushed	Round	Crushed	Round	Crushed
5 %	0.40	Low		25, 52	31			38	34, 54			5, 51	9	
5 %	0.40	High	17			22	35, 55			42	6			16, 48
5 %	0.48	Low	15			26	41			37	14			19
5 %	0.48	High		11	27			33	45			10	8	
7 %	0.40	Low	18			24	44			39	3			7, 50
7 %	0.40	High		12	—			47	29			1, 46	21, 53	
7 %	0.48	Low		28	23			40	36			20	2	
7 %	0.48	High	—			30	32			43	13, 49			4

Notes: Only one level of water content (medium) and two levels of cement content (low and high) were evaluated.
Mix numbers are indicated in table.

Mix Designs

The coarse aggregate volume percentage (CAP) was fixed at 35 and 41 percent. This corresponded to a saturated surface dry coarse aggregate quantity of approximately 920 to 943 kg/m^3 (1550 to 1590 lb/yd^3) for 35-percent coarse aggregate by volume and 1080 to 1103 kg/m^3 (1820 to 1860 lb/yd^3) for 41-percent coarse aggregate by volume. These aggregate quantities are in the range normally used for highway concrete pavement construction. The water-cement ratios (WC) selected were 0.40 and 0.48. The selected WC ratios correspond to cement volumes (CEP) of 9.3 and 11.1 percent, respectively, with water volumes (WAP) of 14.1 percent. Cement contents were 292 kg/m^3 (492 lb/yd^3, 5.2 bag) and 350 kg/m^3 (590 lb/yd^3, 6.3 bag), respectively, values typical of highway pavement construction. Two levels of air content (AIR) of 5 and 7 percent ($\pm$ 0.5 percent) were evaluated. Since coarse aggregate, cement, water, and air content volumes were held at constant levels, only the fine aggregate volume was allowed to vary. Fine aggregate quantity was dependent on the specific gravity and the volumes of coarse aggregate, cement, water, and air. The saturated surface dry fine aggregate quantity ranged from approximately 706 to 967 kg/m^3 (1190 to 1630 lb/yd^3). Mix designs for the 16 different mixes (at three consolidation levels) are shown in table 5 of appendix D.

Plastic Concrete Data

The concrete slump ranged from 10 to 124 mm (0.4 to 4.9 in), and averaged 46 mm (1.8 in) for the 48 primary mixes. Differences between the measured and the targeted air contents ranged from 0 to 0.8 percent and averaged 0.3 percent, well within the targeted nominal values. The unit weight ranged from 2259 to 2383 kg/m^3 (141.0 to 148.8 lb/ft^3) and averaged 2332 kg/m^3 (145.6 lb/ft^3). Relative yield ranged from 99.0 to 102.7 and averaged 100.8 percent.

Initial concrete temperatures were measured and used to ensure that no systematic sources of error influencing strength existed. For the 48 mixes plus 9 replicates, initial concrete temperatures ranged from 18.9 to 23.3 °C (66 to 74 °F) and averaged 21.2 °C (70.1 °F). The correlation coefficient between time and initial mix temperature was 0.19, indicating no significant trend existed that may have influenced the laboratory data.

Plastic concrete data for the 16 different mixes (at 3 consolidation levels) and for the 9 replicate mixes are shown in table 6 of appendix D.

Hardened Concrete Data

Hardened concrete tests, including flexural, splitting tensile, and modulus of elasticity tests at 7, 14, and 28 d, were run on three specimens for the 48 primary and 9 replicate mixes. Strength data for the 55-mix (primary plus replicate) data base are summarized in table 10. The average 28-d, moist-cured strengths at 100 percent consolidation were 4.93, 3.52, and 38.75 MPa (715, 510, and 5620 lbf/in^2) for flexural, splitting tensile, and compressive strength, respectively. The average elastic

Table 10. Hardened concrete data used in strength and consolidation study.

CSL, %	Statistic	Flexural Strength, lbf/in²			Splitting Tensile Strength, lbf/in²			Compressive Strength, lbf/in²			Elastic Modulus, million lbf/in²		
		Age, days			Age, days			Age, days			Age, days		
		7	14	28	7	14	28	7	14	28	7	14	28
94	Count	15	15	15	15	15	15	15	15	15	15	15	15
	Min.	340	405	450	265	350	395	1430	2050	2280	2.10	2.35	2.50
	Max.	525	575	625	430	470	490	3490	3860	4460	3.95	4.15	4.35
	Avg.	445	490	530	355	405	445	2540	2930	3400	2.65	2.80	3.05
97	Count	18	18	18	18	18	18	18	18	18	18	18	18
	Min.	420	465	530	310	355	385	2150	2840	3150	2.65	2.95	3.15
	Max.	625	675	715	480	520	545	4410	5190	5790	4.05	4.50	4.50
	Avg.	505	560	615	395	445	470	3460	4120	4660	3.45	3.70	3.95
100	Count	22	22	22	22	22	22	22	22	22	22	22	22
	Min.	485	530	600	325	370	390	3230	3770	4200	3.15	3.45	3.65
	Max.	745	760	910	545	585	610	5910	6380	7320	5.00	5.30	5.50
	Avg.	600	650	715	435	470	510	4410	4940	5620	4.15	4.30	4.65

NOTES:
1. CSL = consolidation level, percent
2. 1000 lbf/in² = 6.9 MPa

modulus value at 28 d was 32,061 MPa (4,650,000 lbf/in^2). Consolidation significantly reduced strengths and elastic modulus. The 28-d strengths at 97-percent consolidation averaged 4.24, 3.24, and 32.13 MPa (615, 470, and 4660 lbf/in^2) for flexural, splitting tensile, and compressive strength, respectively. The average 97-percent consolidated elastic modulus at 28 d was 27,234 MPa (3,950,000 lbf/in^2). The 3 percent decrease in consolidation corresponded to 14, 8, 17, and 15 percent average decreases at 28 d for flexural, splitting tensile, compressive strength, and elastic modulus, respectively.

Hardened data used in the strength and consolidation study are presented in tables 7 and 8 of appendix D. Individual specimen (triplicates) data for each mix are presented along with summary statistics.

Within-Batch and Between-Batch Variance

Nine duplicate mixes of the primary laboratory mix designs were made to determine variances in hardened concrete strength both between and within mixes. The analysis of the additional nine mixes was used as an indicator of whether significant mix variables or random chance is responsible for observed differences in strength or elastic modulus.

Six of the nine replicate mixes were at the 100-percent consolidation level, two mixes were replicated at the 97-percent consolidation level, and the last replicate mix was at the 94-percent consolidation level. Strength and elastic modulus data are summarized in tables 7 and 8 of appendix D.

The within-batch variance was evaluated by computing the standard deviation of the triplicate specimen hardened concrete tests for the 18 individual mixes (9 different replicated mixes) at each of the 7-, 14-, and 28-d test periods. For flexural strength, on average, the within-batch standard deviation, coefficients of variation, and range increased for tests done at 7 d to tests done at 28 d. Average coefficients of variation increased from 4.1 percent at 7 d to 5.0 percent at 28 d. Similar trends in splitting tensile strength were also observed. The 18-mix average coefficient of variation for splitting tensile strength increased from 6.0 percent at 7 d to 6.6 percent at 28 d. For compressive strength, slight increases in the standard deviation and range were observed. Increases were not significant and resulted in no change in the average coefficient of variation between 7 and 28 d. The 18-mix elastic modulus average standard deviation, coefficient of variation, and range decreased with testing age. Average coefficient of variation decreased from 4.3 to 2.3 percent between 7 and 28 d. The 18-mix average standard deviation, coefficient of variation, and range data are summarized in table 9 of appendix D.

For all 18 mixes in each of the 3 testing periods, the coefficients of variation averaged 4.4, 5.9, 3.1, and 3.1 percent for flexural strength, splitting tensile strength, compressive strength, and elastic modulus, respectively. Overall average within-batch standard deviations were 0.19, 0.19, 0.97, and 862 MPa (28, 28, 140, and 125,000 lbf/in^2) for flexural strength, splitting tensile strength, compressive strength, and elastic modulus, respectively.

The within-batch pooled standard deviation was also computed for the 12 mixes prepared at 100-percent consolidation, for the 6 mixes prepared at less than 100-percent consolidation, and for the combined 18-mix pool. As with the averaged data, the flexural strength, splitting tensile strength, and compressive strength standard deviations tended to increase with testing age. Generally, the pooled standard deviation at any age was less for the 100-percent consolidation mixes than for the pooled standard deviation of the less than fully consolidated mixes. The 12 mixes prepared at 100-percent consolidation had pooled standard deviations of 0.25, 0.30, 1.18, and 758 MPa (36, 43, 171, and 110,000 lbf/in^2) at 28 d for flexural strength, splitting tensile strength, compressive strength, and elastic modulus, respectively. This is shown in table 11, along with the pooled within-batch standard deviations.

Using procedures presented in American Concrete Institute (ACI) 214, the average within-batch range can be estimated.[12] For instance, for a within-batch standard deviation of 0.28 MPa (40 lbf/in^2), the expected average range between two cores taken from the same sublot is 0.31 MPa (45 lbf/in^2). Similarly, if three cores were sampled from the same sublot the average expected range would be 0.47 MPa (68 lbf/in^2). The within-batch variation is of such magnitude that when sampling sublots during construction, more than one core should be used in the determination of pay factors. Sampling multiple cores from sublots also has the advantage of identifying areas that should be retested. For example, if the range suddenly increases significantly compared to previous sublot ranges, then the sublot should probably be retested.

The between-batch standard deviations were computed from the differences between replicate batch means. The mean square of the nine-mean differences is called the replicate mean square (Rep. MS). The square root of the replicate mean square (replicate root mean square [Rep. RMS]), is the pooled estimate of the standard deviation between replicate mixes. The Rep. RMS and corresponding between-batch coefficients of variation increased with testing age for flexural strength, compressive strength, and elastic modulus. For splitting tensile strength, the Rep. RMS did not change significantly. The coefficient of variation for splitting tensile strength decreased with time. The replicate standard deviations at 28 d are 0.26, 0.13, 2.61, and 1517 MPa (37, 19, 379, and 220,000 lbf/in^2) for flexural strength, splitting tensile strength, compressive strength, and elastic modulus, respectively. Replicate root mean square data are shown in table 10 of appendix D and are summarized in table 11.

The Rep. RMS was used in the regression analyses to evaluate the goodness of fit. If the regression prediction errors are significantly smaller than the variability between replicate batches, then the regression equation can be considered a good fit of the experiment data. Use of the Rep. RMS in evaluating goodness of fit is described later.

Table 11. Summary of within-batch and between-batch variances.

Variance	Description	Flexural Strength, lbf/in²			Splitting Tensile Strength, lbf/in²			Compressive Strength, lbf/in²			Elastic Modulus, million lbf/in²		
		Age, days			Age, days			Age, days			Age, days		
		7	14	28	7	14	28	7	14	28	7	14	28
Within Batch	Pooled Std. Dev.	26	29	42	31	28	38	162	163	216	0.20	0.17	0.12
	100% CSL Std. Dev.	26	24	36	34	25	43	124	116	171	0.12	0.16	0.11
	<100% CSL Std. Dev.	27	36	51	23	33	25	220	231	285	0.29	0.20	0.12
Between Batch	Overall Mean	577	626	691	430	473	508	4143	4781	5397	3.86	4.09	4.43
	Rep. MS	463	669	1400	417	689	358	115,606	105,583	143,839	0.03	0.04	0.05
	Rep. RMS	22	26	37	20	26	19	340	325	379	0.18	0.19	0.22
	COV, %	3.7	4.1	5.4	4.7	5.6	3.7	8.2	6.8	7.0	4.7	4.6	5.0
Overall	Std. Dev.	34	36	52	40	36	47	362	345	416	0.22	0.24	0.25

NOTES:
1. Overall standard deviation based on 100-percent consolidated pooled within-test standard deviation (12 mixes) and all 9 Rep. RMS.
2. CSL = consolidation level, percent; COV = coefficient of variation
3. 1000 lbf/in^2 = 6.9 MPa

As shown in table 9 of appendix D, there is a significant difference in the within-batch pooled standard deviation estimates. Generally, the less than fully consolidated specimens exhibited a higher within-test variance than the fully consolidated mixes. The within-batch and between-batch standard deviations were very close at all test ages for flexural strength. For splitting tensile data, the between-batch variation was significantly lower at 7 and 28 d than the 100-percent consolidated pooled within-batch standard deviation. For compressive strength and elastic modulus data, the within-batch standard deviation was significantly lower than the between-batch standard deviation.

To estimate an overall standard deviation (both within-batch and between-batch), the 100-percent consolidated pooled estimate was combined with the replicate standard deviation. As summarized in table 11, the overall standard deviations tended to increase between 7 and 28 d. The overall standard deviation was 0.36, 0.32, 2.86, and 1724 MPa (52, 47, 416, and 250,000 lbf/in^2) at 28 d for flexural strength, splitting tensile strength, compressive strength, and elastic modulus, respectively. The overall standard deviations between flexural and splitting tensile strength were close when measured at all three testing periods. Since the overall standard deviations were not significantly different at each test period, the average pooled standard deviation was calculated. The average pooled standard deviations were 0.29, 0.29, 2.58, and 1627 MPa (41.0, 41.0, 374, and 236,000 lbf/in^2) for flexural strength, splitting tensile strength, compressive strength, and elastic modulus, respectively.

The overall standard deviation data can be used in the development of mix designs, as illustrated in ACI 214.[13] It is assumed that the *in situ* core splitting tensile strength testing is to be done at ages where the pavement *in situ* strength is approximately that of 7-d moist lab cure cylinders. From table 11, the overall standard deviation at 7 d is 0.28 MPa (40 lbf/in^2). If the target split tensile strength is 3.10 MPa (450 lbf/in^2), the design strength would be computed as follows:

$$ST_{req.} = ST_{des} + t\,\sigma \tag{15}$$

where:

$ST_{req.}$	=	Required 7-d mix design strength
ST_{des}	=	7-d design strength
t	=	Constant depending on proportion of tests that may fall below the design strength
σ	=	Overall standard deviation

For a 10 percent probability that the strength will fall below the design strength, the value of t is 1.28. Using equation 15, the required splitting tensile strength targeted in the mix design phase would be 3.45 MPa (501 lbf/in^2).

Strength and Mix Design Factors

The data from the fully consolidated mixes shown in table 9 were used in evaluating significant mix design factors affecting strength and elastic modulus. The purpose of the analysis was only to assist in developing a mix design and not to predict strength. The 16-mix, fully consolidated data were grouped into one data set combining all data at all ages. Additional main factor variables used in the analysis were the fine aggregate volume (FAP) and test time (AGE). By definition, the fine aggregate volume is 100 minus the sum of coarse aggregate volume (CAP), cement volume (CEP), water volume (WAP), and air volume (AIR). Two-factor interactions were also used in the strength and elastic modulus prediction analysis.

To reduce multicollinearity among dependent variables, each main factor (independent variable) was transformed by subtraction of the level mean. For example, the two levels of WC that were used in the laboratory experiment were 0.40 and 0.48, with an average value of 0.44. The independent variable used in the regression analysis was (WC - 0.44). The WC minus mean independent variable was renamed WCD (WC deviation of variable from mean value). The transformation was necessary to reduce intercorrelations between interaction variables. For instance, the correlation coefficient between two cross product variables (interaction effect) may be close to 1.0, whereas the correlation between the deviation cross product variables may be significantly less.

A stepwise regression procedure was used to identify significant main and interaction-effect variables. The stepwise procedure starts with one independent variable that has the highest simple correlation with the dependent variable. The next most significant variable enters the model and if the first variable significance drops below a threshold value, that variable is dropped out of the model. The procedure stops when all significant variables have been identified. The threshold for adding and deleting variables was set at a 5-percent significance level.

The regression analysis to predict strength and elastic modulus identified between 7 and 12 significant variables. For the prediction of flexural strength, splitting tensile strength, compressive strength, and elastic modulus, between five and six main effects were identified as being significant. Test age (AGED), air content (AIR), coarse aggregate hardness (CAHD), and coarse aggregate geometry (CAGD) were significant to all four regression equations. For all three strength types, these main effects resulted in increased strengths if the aggregate geometry was crushed (CAG=1) or if the hardness was soft (CAH=0). With the exception of splitting tensile strength, where cement volume (CEPD) was significant, the remaining equations contained the main effect of water-cement ratio (WCD). As the WCD decreased (increasing cement), or CEPD increased, the predicted strength increased.

Different interaction effects were significant to each regression equation developed. No two-way interaction was significant to all four equations developed. The number of interaction effects varied from two to seven for the four regression equations. With the exception of two interaction terms containing coarse aggregate volume (CAPD) in the splitting tensile and elastic modulus models, all interaction terms contained significant main effects.

The coefficient of determination (R^2) is defined as the amount of variability in the dependent variable explained by the regression equation. For example, if R^2 is 0.85, the regression equation can account for 85 percent of the variability in the dependent variable. Coefficients of determination ranged from 0.85 to 0.95 for the four models developed. Results of the four regression analyses are summarized in table 12. The predicted versus measured flexural strength is plotted in figure 23. Splitting tensile, compressive strength, and elastic modulus are shown in figures 1 through 3 of appendix D. The analysis was repeated using log transformations of strength and elastic modulus. Coefficients of determination were slightly lower, indicating that the regression equations would not be improved using dependent variable transformations.

The replicate standard deviations determined from analysis of the nine replicate mixes is also shown in table 12. The Rep. RMS was estimated at 7, 14, and 28 d for flexural strength, splitting tensile strength, compressive strength, and elastic modulus. Since the regression analysis combined all test data at 7, 14, and 28 d, the pooled Rep. RMS (7, 14, and 28 d) was used in evaluating the regression fit, as reported in table 12. The error mean square (EMS) ratio listed in table 12 is computed as the square of the regression standard error of estimate (SEE) divided by the Rep. RMS. The F-distribution was used to test for the regression variance being less than the replicate variance. If the regression variance is greater than the replicate variance, then the regression prediction errors are greater than what would be expected between the replicate batches, and the usefulness of the regression equation would be limited. All four regression equations were not significant at the 10-percent level of significance; therefore, it can be inferred that all SEE are within the expected variation of the corresponding replicate standard deviations.

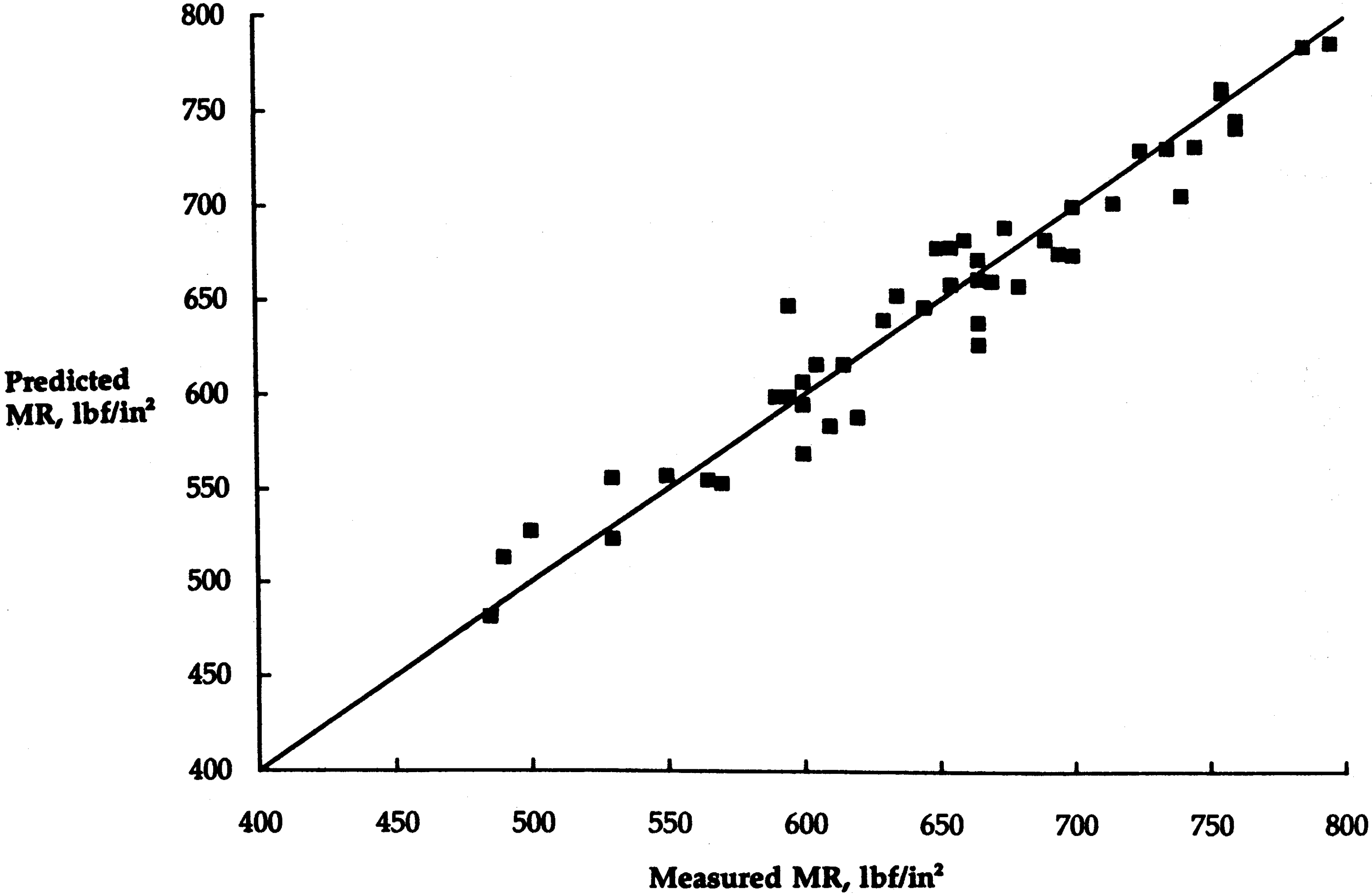
Since the 28-d flexural strength is of the most interest in the mix design phase, a sensitivity analysis was conducted using the flexural strength equation in table 12. For each aggregate type, the average change in flexural strength was computed for level changes in air content (AIR), water-cement ratio (WC), and coarse aggregate volume (CAP). For example, for both levels of CAP and water-cement ratio, the percent change in modulus of rupture was computed for each of the four aggregate types as air content was decreased from 7 to 5 percent. The average percent change when air content is reduced for all WC and CAP combinations is reported in table 13. A similar analysis was done when WC or CAP was changed.

As expected, the largest increases in flexural strength occurred with a decrease in WC. For all four aggregate types, the increase in flexural strength due to a decrease in WC ranged from 11.2 to 13.4 percent. Decreases in air content significantly increased strength only for the harder aggregates. For softer coarse aggregates, where the aggregate can be weaker than mortar, changes in mortar properties (air content and strength) did not significantly affect strength. Coarse aggregate volume decreases resulted in increased flexural strength only for crushed coarse

Table 12. Mix design variable effect on strength and elastic modulus.

Independent Variable	Flexural Strength, lbf/in²	t-stat.	Split Tensile Strength, lbf/in²	t-stat.	Compressive Strength, lbf/in²	t-stat.	Elastic Modulus, million lbf/in²	t-stat.
CAHD = (CAH - 0.5)	-27.5	-4.34	-14.4	-2.23	-185	-2.35	0.231	6.96
CAGD = (CAG - 0.5)	82.5	13.02	63.1	9.80	577	7.35	0.752	22.63
AIRD = (AIR - 6)	-24.6	-7.76	-19.9	-6.18	-200	-5.09	-0.0865	-5.20
WCD = (WC - 0.44)	-635	-8.02			-10,786	-11.00	-3.36	-8.09
CAPD = (CAP - 38)	-2.29	-2.17						
CEPD = (CEP - 10.2)			21.4	5.98				
AGED = (AGE - 16.33)	4.99	13.75	3.57	9.67	54.3	12.09	0.0232	12.20
CAHD • CAGD	-74.2	-5.85			-631	-4.02	-0.521	-7.84
CAHD • AIRD	-30.4	-4.80						
CAHD • WCD	-406	-2.56					-2.86	-3.45
CAHD • CAPD							0.0299	2.70
CAHD • AGED							0.00765	2.01
CAGD • AIRD			15.2	2.36			0.119	3.57
CAGD • WCD								
CAGD • CAPD	-7.5	-3.55						
CAGD • AGED					22.6	2.52	0.0105	2.75
AIRD • WCD							1.02	2.45
AIRD • CAPD			2.47	2.30				
AIRD • AGED	0.99	2.73						
Constant	652	205.85	469	145.60	4944	126.00	4.33	260.37
R^2 adj.	0.925		0.853		0.887		0.950	
SEE	21.95		22.32		271.8		0.1151	
SEE df	36		40		40		35	
Pooled Rep. SD	29.0		21.9		348.7		0.197	
Rep. df	9		9		9		9	
EMS Ratio	0.6		1.0		0.6		0.3	
EMS Ratio Signif.	NS		NS		NS		NS	

NOTES:
1. EMS ratio = $(SEE/Rep.\ RD)^2$
2. NS = EMS ratio not significant at 10-percent significance level.
3. All variable coefficients significant at the 5-percent level of significance.
4. 1000 lbf/in² = 6.9 MPa



1000 lbf/in² = 6.9 MPa

Figure 23. Predicted vs. measured flexural strength (MR) as a function of mix design inputs.

Table 13. Flexural strength sensitivity to mix component analysis.

	Average Change in Flexural Strength, %			
	CH	RH	CS	RS
WC 0.48 to 0.40	12.3	13.2	11.2	13.4
Air Content 7% to 5%	8.3	8.8	-0.5	-0.6
CAP 41% to 35%	5.2	-1.3	4.8	-1.3

NOTES:

1. CH = crushed hard aggregate
2. RH = rounded hard aggregate
3. CS = crushed soft aggregate
4. RS = rounded soft aggregate
5. WC = water-cement ratio
6. CAP = coarse aggregate volume, percent

aggregates. Changes in flexural strength due to decreases in air content were greater than those due to decreases in CAP.

The equations in table 12 can be used as an aid during the mix design process. For example, in PRS, the flexural strength is of primary concern. The contractor may have options in using different coarse aggregate sources with different aggregate geometries or hardnesses. The flexural strength equation can be used to evaluate the effect of using different aggregate sources, volumes, and air contents on flexural strength. The relative effects in variable changes can be used to economically design a mix with minimal risk of incurring a pay disincentive.

Simple Interstrength Relationships

The 7-, 14-, and 28-d strength data from the 16 fully consolidated mixes were analyzed to develop mix-independent interstrength relationships. Simple regression equations predicting strength or elastic modulus as a function of one independent strength variable were developed and are summarized in table 14. The predicted versus measured variables are plotted in figures 24 through 27.

The coefficients of determination for the regression equations ranged from 0.76 to 0.78. The regression analysis of elastic modulus on the square root of compressive strength indicated that the equation constant was not significant (t-stat = -0.92). The regression between elastic modulus and square root of compressive strength forces the equation through zero. The coefficient of determination is not computed when regression equations are forced through zero. The coefficient of determination was 0.60 when the nonsignificant constant is included.

An analysis of the error mean square (EMS) ratio, as previously described, was conducted to evaluate the fit of the regression equations. As illustrated in table 14, the elastic modulus equation has a significant EMS ratio that indicates a serious lack of fit. However, the remaining interstrength equations developed in this study do not exhibit a serious lack of fit.

Previous research on interstrength relationships has shown that a better fit could be developed for mix-specific strength data. Therefore, mix-specific interstrength and elastic modulus relationships were developed because of the criticality of the interstrength relationships in estimating the *in situ* flexural strength pay factors in a PRS.

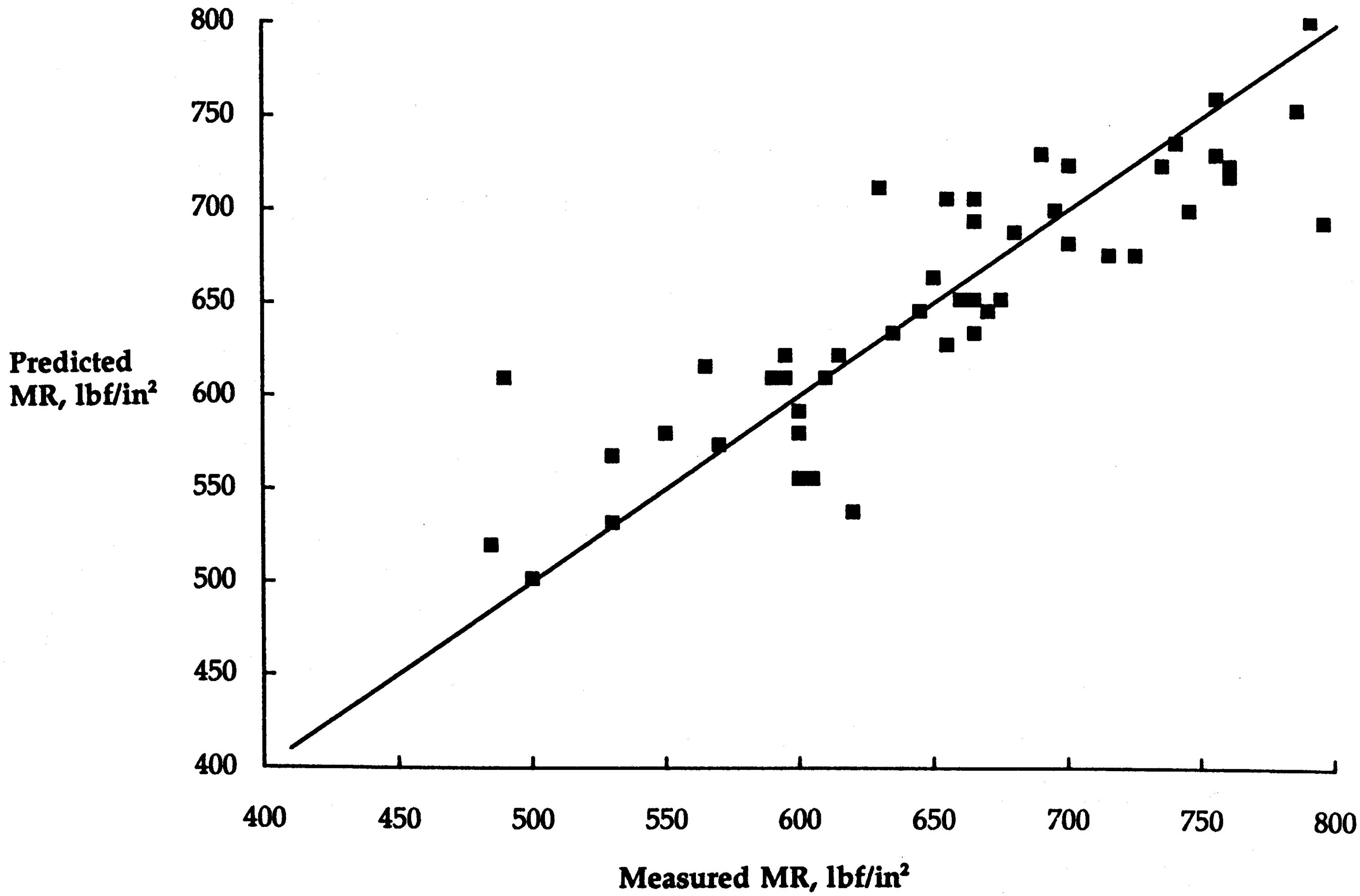
Mix-Specific Strength Interrelationships

Analysis of the 16-mix data base indicated that a general interstrength relationship did not yield results that could be applied reliably in a PRS. Since the flexural strengths are to be determined indirectly from *in situ* core testing (compressive or splitting tensile), it is essential to minimize prediction errors when using interstrength relationships. Therefore, mix-specific interstrength relationships were developed for all 46 mixes. The analysis was done to demonstrate that for the range of different

Table 14. Simple interstrength regression equations.

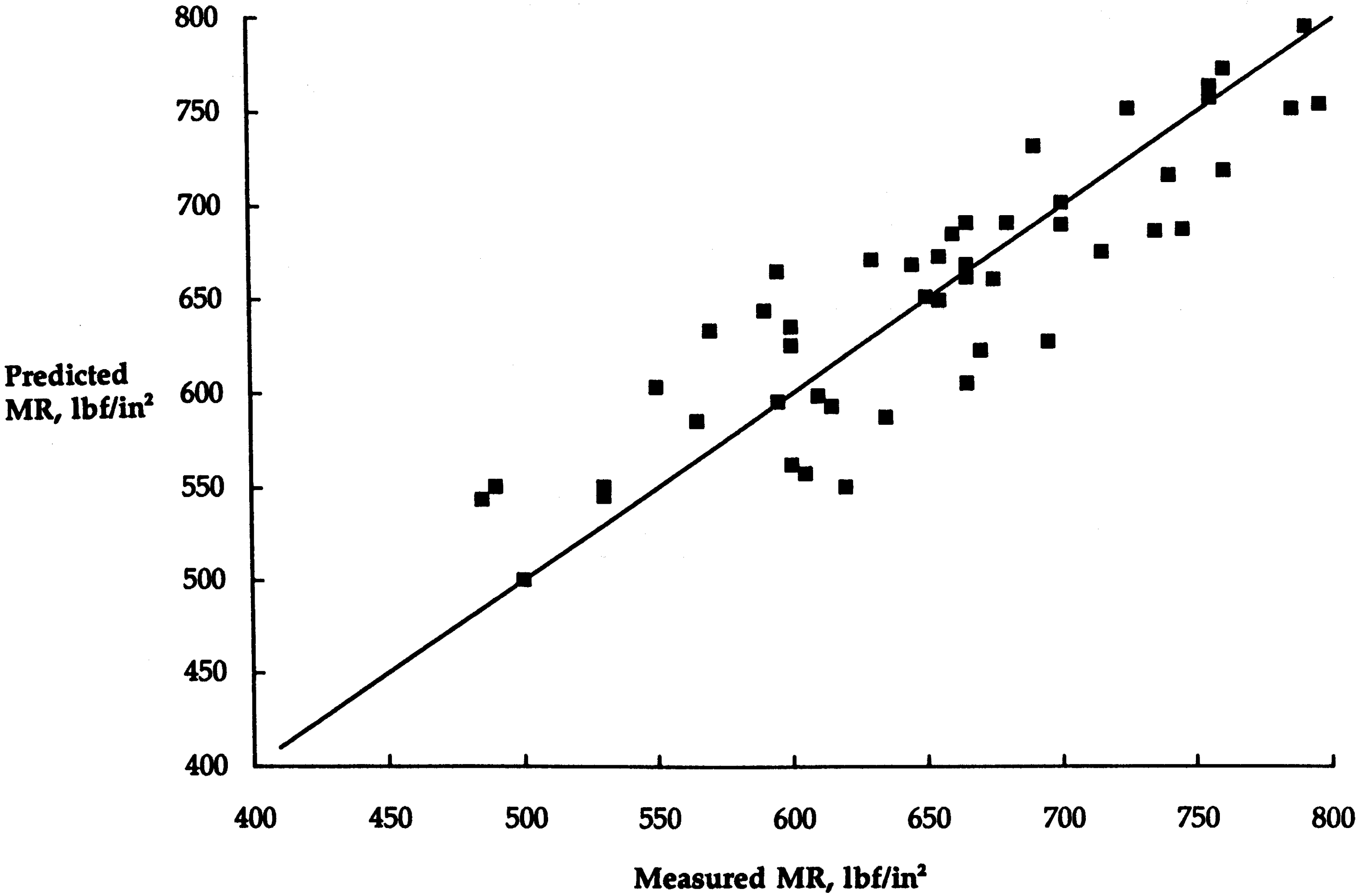
Dependent Variable	Independent Variable	Indep. Coef.	t-stat	Constant	t-stat	R^2	SEE	SEE df	Repl. SD	Repl. df	EMS Ratio	EMS Signif.
Flexural Strength, lbf/in^2	Split Tensile Strength, lbf/in^2	1.2	12.10	88.5	1.89	0.761	39.6	46	29.0	9	1.9	NS
Flexural Strength, lbf/in^2	sqrt (f'c, lbf/in^2)	12.3	12.83	-209	-3.11	0.782	37.9	46	29.0	9	1.7	NS
Splitting Tensile Strength, lbf/in^2	log (f'c, lbf/in^2)	711	12.58	-2155	- 10.33	0.775	27.9	46	21.9	9	1.6	NS
Elastic Modulus, million lbf/in^2	sqrt (f'c, lbf/in^2)	0.062	91.18	****	****	****	0.33	47	0.197	9	2.8	5%

NOTES:
1. Fully consolidated specimens at 7, 14, and 28 d.
2. EMS Ratio = $(SEE/Repl.\ SD)^2$
3. NS = EMS Ratio not significant at 10-percent significance level.
4. Modulus prediction equations forced through zero. Coefficient of determination not computed.
5. 1000 lbf/in^2 = 6.9 MPa



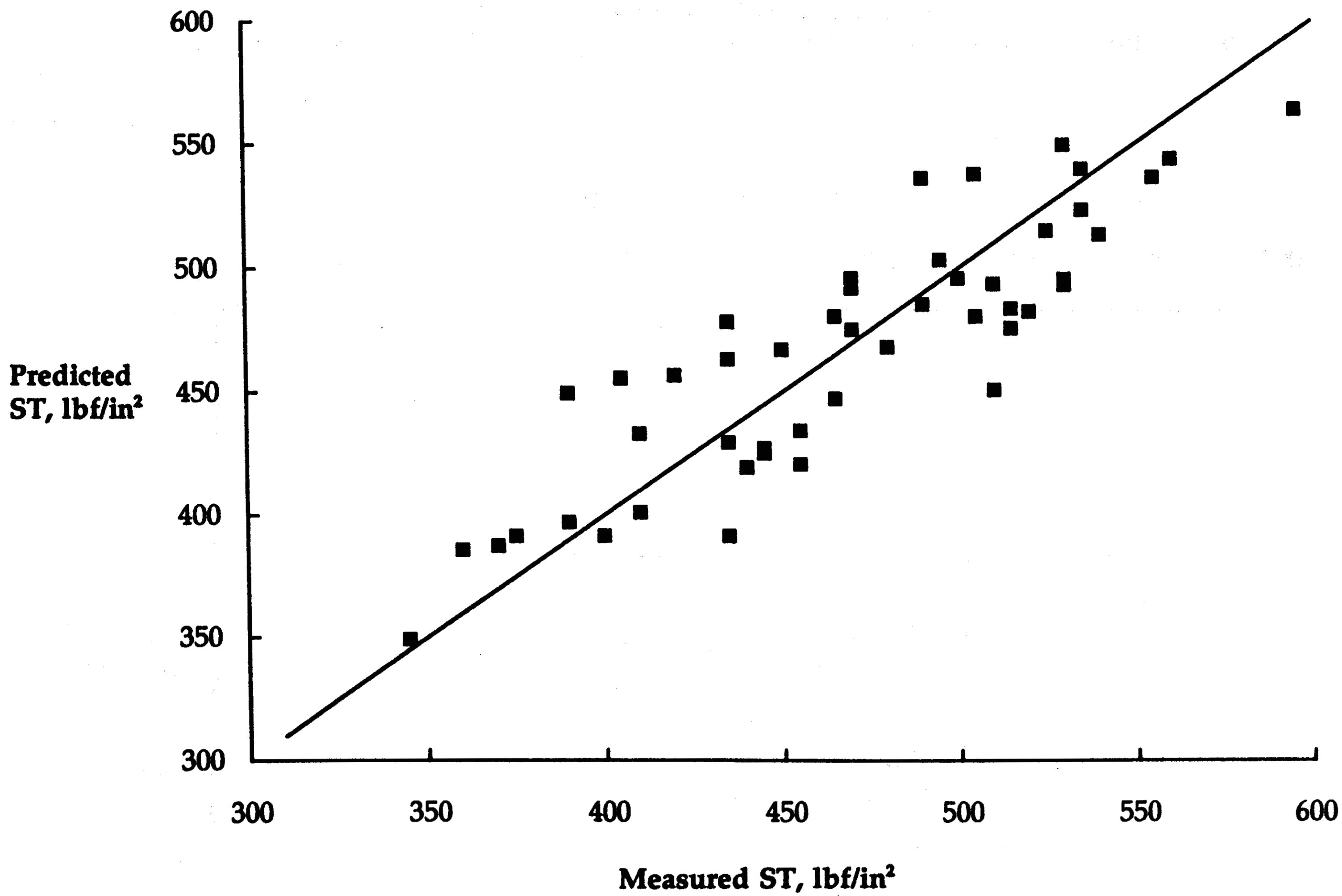
1000 lbf/in² = 6.9 MPa

Figure 24. Predicted vs. measured flexural strength (MR) as a function of splitting tensile strength (ages 7, 14, and 28 days).



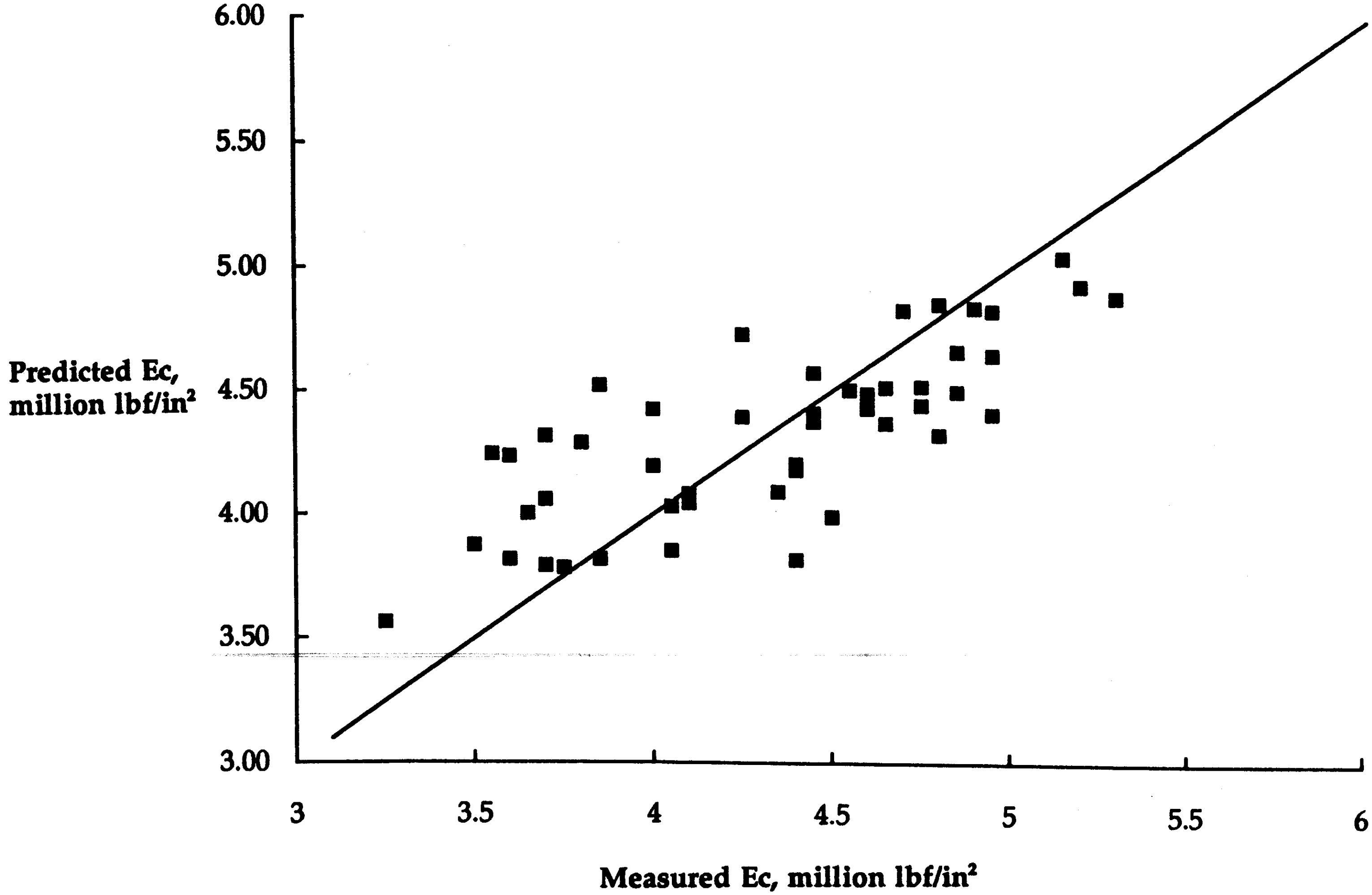
1000 lbf/in² = 6.9 MPa

Figure 25. Predicted vs. measured flexural strength (MR) as a function of compressive strength (ages 7, 14, and 28 days).



1000 lbf/in^2 = 6.9 MPa

Figure 26. Predicted vs. measured splitting tensile strength (ST) as a function of compressive strength (ages 7, 14, and 28 days).



1 million lbf/in² = 6.9 GPa

Figure 27. Predicted vs. measured elastic modulus (Ec) as a function of compressive strength (ages 7, 14, and 28 days).

paving mixes used in the strength and consolidation study, interstrength relationships could be developed with much less variability than the simple general relationships shown in table 14.

The average strength data from the triplicate specimens (shown in tables 7 and 8 of appendix D) were used in developing the various interstrength relationships. Dependent and independent variables were transformed using square root, logarithmic, and inverse transformations. For the prediction of elastic modulus, the square root of compressive strength was used assuming no constant. The interstrength model with the highest R^2 was selected as the most representative interstrength model.

For the 184 models developed, either no variable transformation or the inverse transformation was the most common. Less common transformations were the log and square root transformations. For most mixes, the three types of interstrength relationships were judged to be a good fit. The coefficients of determination for flexural strength as a function of compressive strength, flexural strength as a function of splitting tensile strength, and splitting tensile as a function of compressive strength averaged 0.94, 0.90, and 0.95, respectively. Some relationships for mixes prepared at 97-and 94-percent consolidation were not as good due to lack of linear trends. For instance, some of these mixes did not exhibit an increase in strength with age. Regression equations developed for the 46 mixes for predicting flexural from compressive strength, flexural from splitting tensile strength, splitting tensile from compressive strength, and elastic modulus as a function of the square root of compressive strength are listed in tables 11 and 12 of appendix D.

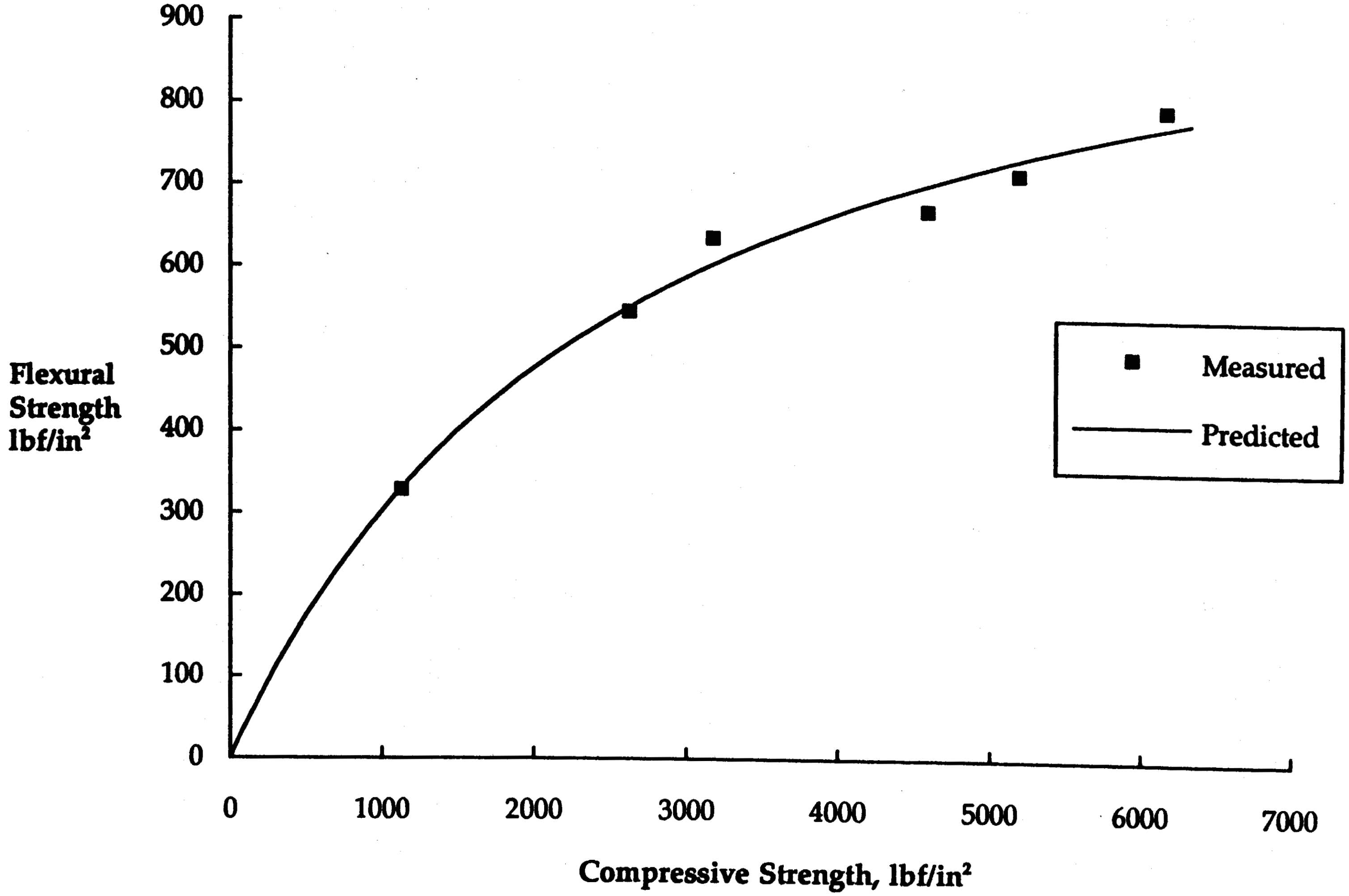
Most of the models developed were based on average 7-, 14-, and 28-d strengths. However, 12 of the 16 different mixes prepared at the three consolidation levels were tested at 1, 3, 5, 7, 14, and 28 d. These 12 mixes were evaluated since mix-specific interstrength equations used in a PRS should be based on several testing ages, not just at 7, 14, and 28 d. Since flexural strength is to be determined for use in a fatigue analysis, the flexural strength (modulus of rupture) models as a function of either compressive or splitting tensile strengths were examined.

The most common flexural strength prediction equation was the inverse-inverse transformation model. Regression equations are listed in table 15, with the coefficients of determination ranging from 0.955 to 0.999. Typical interstrength relationships are shown in figures 28 and 29.

The effects of density are not completely accounted for in predicting flexural strength. The independent variable inherently contains only some of the effects of consolidation on predicted flexural strength. For example, if consolidation affected compressive strength the same as flexural strength, there would be no significant difference between regression equations developed from 100-, 97-, and 94-percent consolidation. Regression equations plotted in figure 30 for mix 19, 26, and 37 (which, as shown in table 15, are the same mix prepared at different levels of consolidation) indicate that interstrength relationships are a function of consolidation.

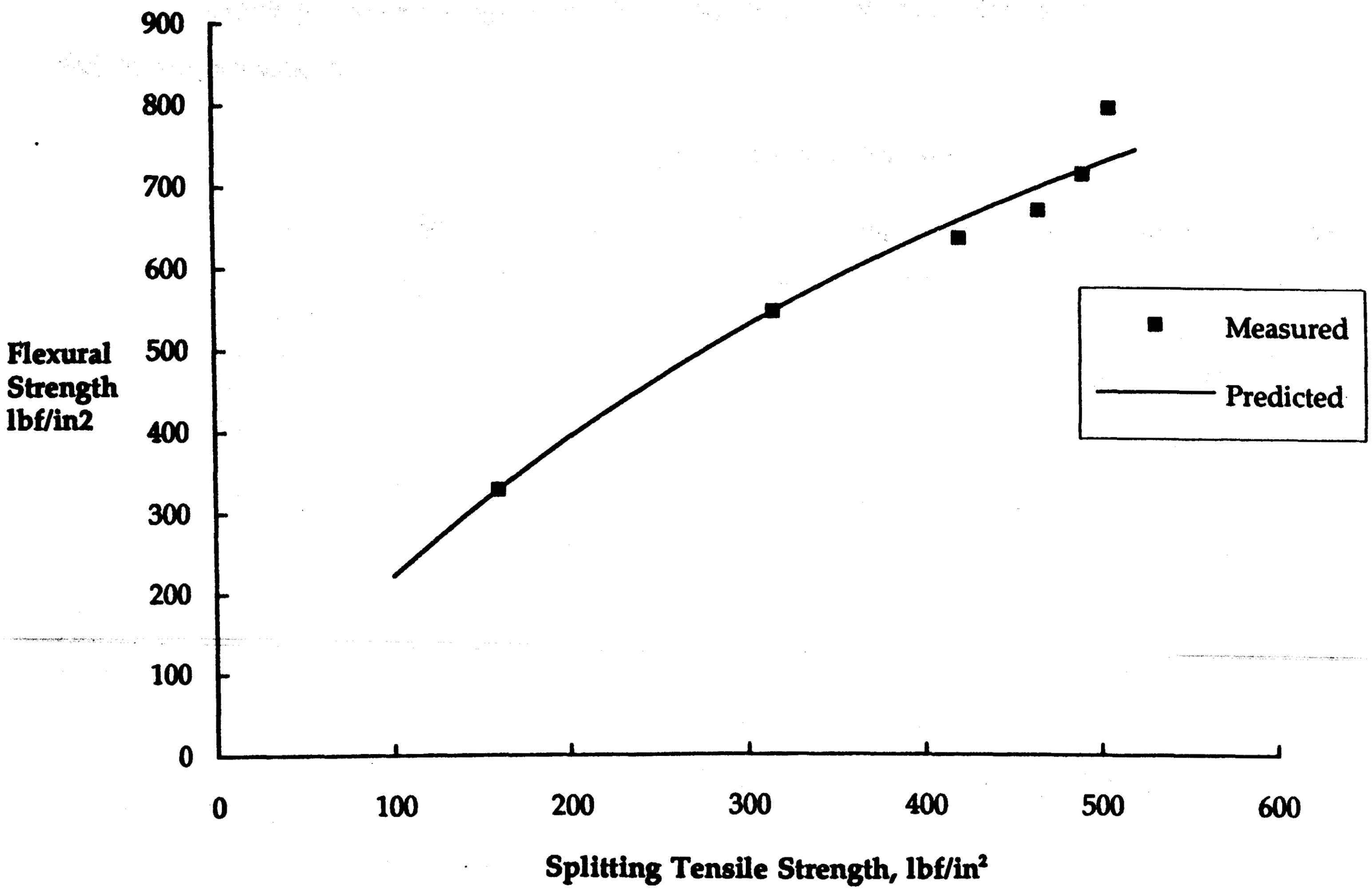
Table 15. Mix-specific regression equations for concrete strengths.

Mix No.	Consol. Level	No. of Data	Depend. Variables	Indep. Variables	Coefficient	Constant	R^2	SEE
19	100	6	1/MR 1/MR	sqrt(f'c) ST	-5.23E-05 -5.45E-06	0.00509 0.00423	0.988 0.955	8.649E-05 0.0003039
37	97	6	1/MR 1/MR	log(f'c) log(ST)	-0.003393 -0.004611	0.014032 0.014174	0.955 0.974	0.0001733 0.0001339
26	94	6	1/MR 1/MR	1/f'c 1/ST	4.030846 0.761758	0.000762 0.000374	0.990 0.984	0.0001303 0.0001118
16	100	6	log(MR) 1/MR	1/f'c ST	-1009.45 -0.003517	3.034825 0.010975	0.979 0.933	0.0146554 6.46E-05
20	100	6	1/MR 1/MR	1/f'c 1/ST	2.395502 0.391151	0.000904 0.000593	0.993 0.991	5.95E-05 2.976E-05
1	100	6	1/MR 1/MR	log(f'c) ST	-0.0029 -5.04E-06	0.012261 0.004119	0.994 0.992	3.325E-05 2.442E-06
5	100	6	sqrt(MR) log(MR)	1/(f'c) 1/ST	-29409.1 -159.713	32.50554 3.1736	0.994 0.992	0.234066 0.0126106
38	97	6	1/MR 1/MR	log(f'c) log(ST)	-0.002521 -0.003605	0.010809 0.011333	0.959 0.954	9.198E-05 2.168E-05
25	94	6	1/MR log(MR)	1/f'c 1/ST	2.993553 -137.8899	0.000878 3.043102	0.995 0.991	5.174E-05 0.0059057
2	100	6	1/MR 1/MR	1/f'c 1/ST	8.231133 1.269285	-0.000215 -0.00151	0.998 0.999	8.701E-05 1.638E-05
21	100	6	1/MR 1/MR	1/f'c log/ST	4.925673 -0.005958	0.000666 0.017416	0.993 0.997	5.354E-05 5.074E-05
6	100	6	log(MR) 1/MR	1/f'c 1/ST	-1023.06 0.545421	3.005526 0.000432	0.985 0.986	0.01363 3.971E-05



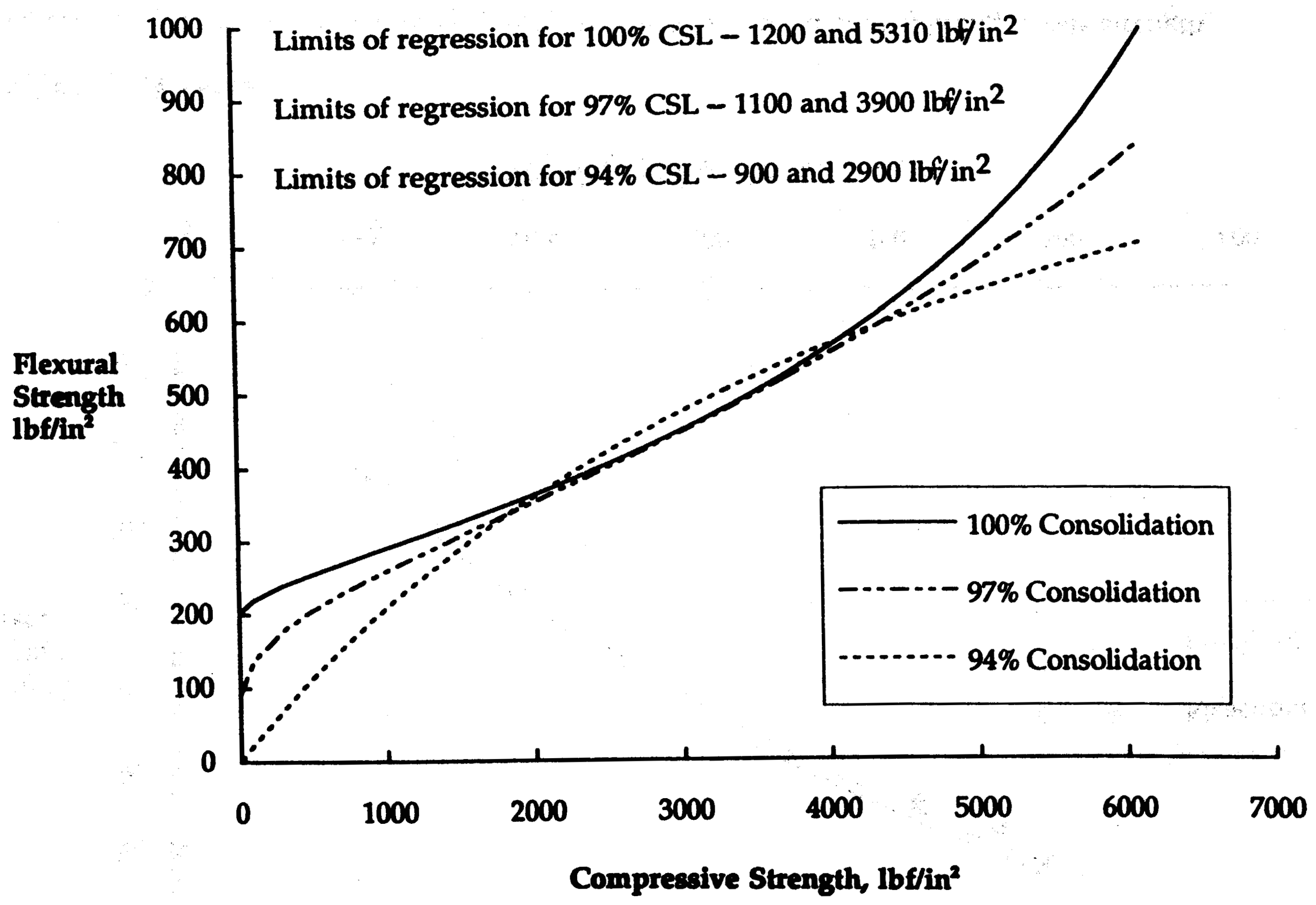
1000 lbf/in² = 6.9 MPa

Figure 28. Predicted and measured flexural strength as a function of compressive strength for mix number 20.



1000 lbf/in² = 6.9 MPa

Figure 29. Predicted and measured flexural strength as a function of splitting tensile strength for mix number 20.



1000 lbf/in² = 6.9 MPa

Figure 30. Predicted flexural strength at different consolidation levels.

An analysis of flexural strength prediction errors was conducted on the 12 mixes tested at 1, 3, 5, 7, 14, and 28 d. Overall, the flexural strength prediction from the compressive strength models was slightly better (lower percent error) than the splitting tensile models. The average prediction errors (absolute values) for all 12 mixes at the different time periods were 2.6 and 2.7 percent for the compressive and splitting tensile strengths, respectively. Results of the interstrength error analysis are presented in table 13 of appendix D. As expected from the observed high coefficients of determination, the flexural strength prediction errors were very small. The interstrength study indicates that under controlled laboratory conditions, flexural strength can be predicted quite well from either splitting tensile or compressive strength.

Effects of Consolidation and Air Content on Interstrength Relationships

The purpose of this analysis was to evaluate the effects of consolidation, air content, and mix design factors on interstrength relationships. The data from all 46 mixes were grouped into one data set combining all of the data at each of the testing ages. An additional main factor variable used in the analysis was the fine aggregate volume (FAP). By definition, the fine aggregate volume is 100 minus the sum of coarse aggregate volume (CAP), cement volume (CEP), water volume (WAP), and air volume (AIR). Two-factor interactions were also used in the strength and elastic modulus prediction analysis. The test time variable (AGE) was excluded.

As previously discussed, to reduce multicollinearity among dependent variables, each main factor (independent variable) was transformed by subtraction of the level mean. The transformation was necessary to reduce intercorrelations between interaction variables. A stepwise regression procedure was used to identify significant main and interaction effect variables. The threshold for adding and deleting variables was set at the 5-percent significance level.

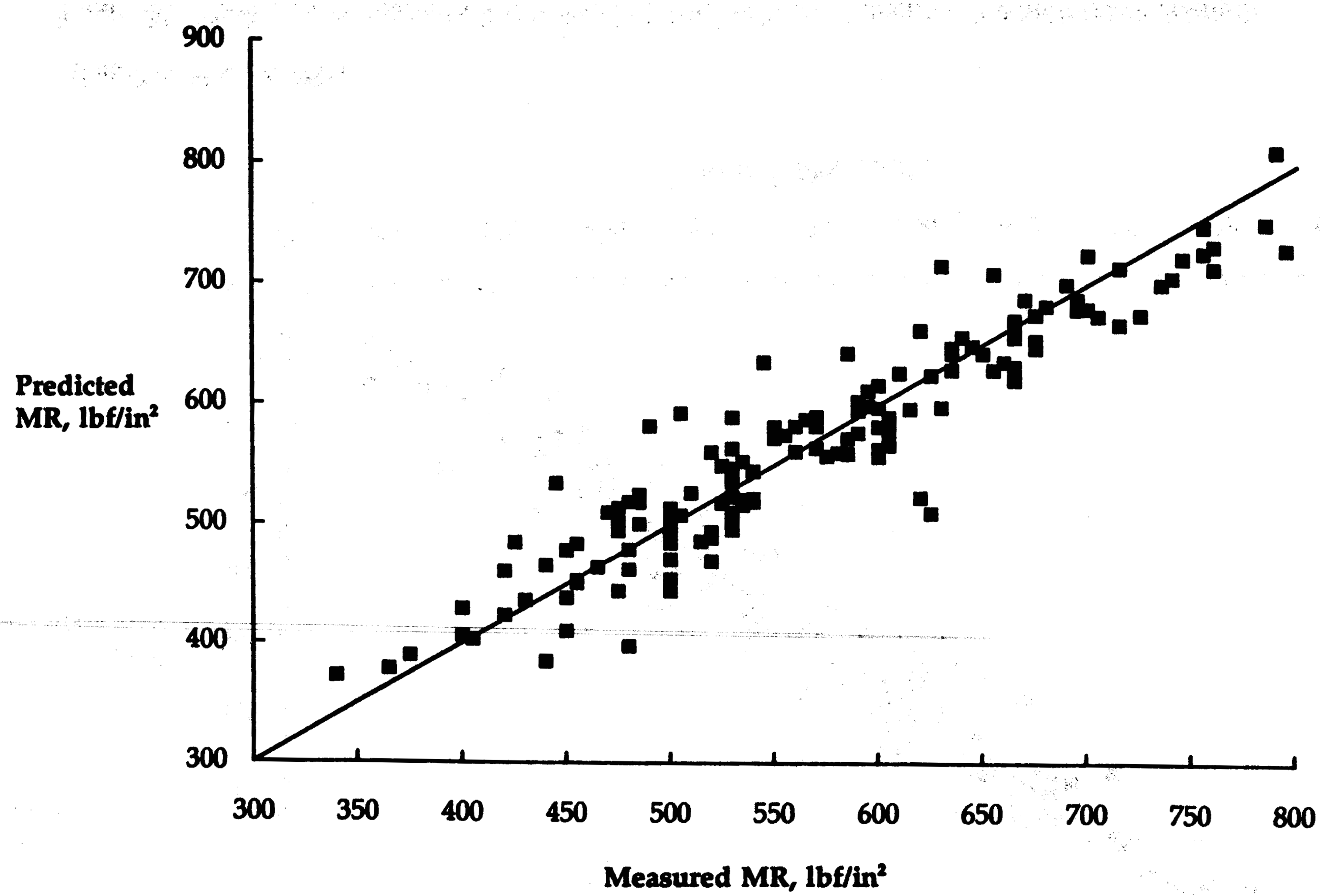
Four prediction equations, similar to those shown in table 14, were developed. The same compressive strength transformations used in the simple interstrength relationships were used. The regression analysis to predict strength and elastic modulus identified between 4 and 11 significant variables in addition to the interstrength independent variable. Only coarse aggregate geometry (CAGD) and consolidation level (CSLD) were significant in all four regression equations. The coefficients of determination (R^2) ranged from 0.83 to 0.94 for the four models developed. Results of the four regression analyses are summarized in table 16, with predicted versus measured variable plots shown in figures 31 through 34.

The independent variable inherently contains only some of the effects of consolidation on predicted properties. For example, if the effects of consolidation affected the square root of compressive strength the same as flexural strength, then consolidation would not be a significant variable when predicting flexural strength from compressive strength. Since consolidation is a significant main effect in all four equations, the effects of consolidation are not completely contained in the independent strength variables.

Table 16. Regression analysis of consolidation and mix design factors on hardened concrete properties.

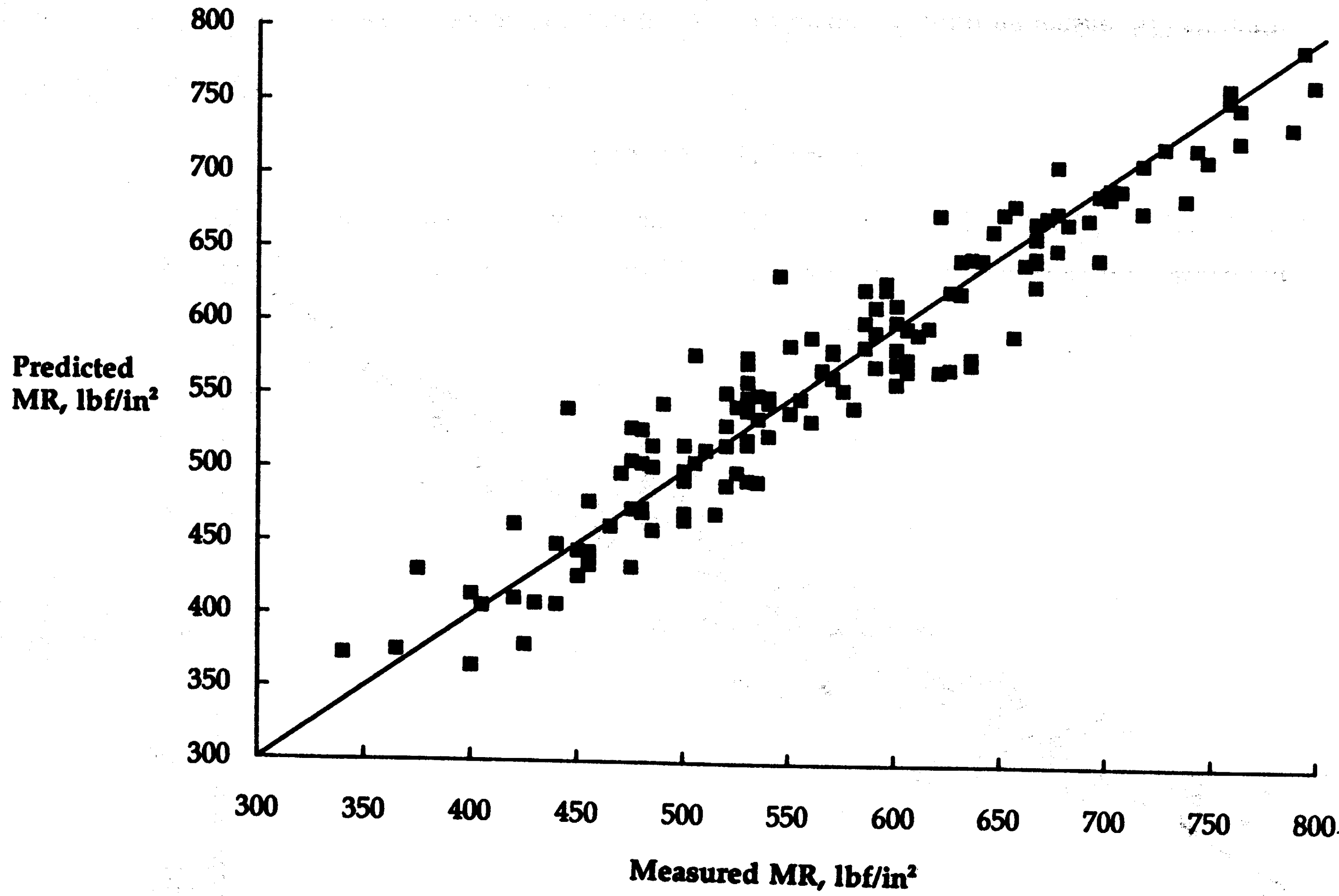
Independent Variable	Flexural Strength, lbf/in²	t-stat.	Flexural Strength, lbf/in²	t-stat.	Splitting Tensile Strength, lbf/in²	t-stat.	Elastic Modulus, million lbf/in²	t-stat.
Split Tensile, lbf/in²	1.05	18.06						
sqrt (f'c, lbf/in²)			8.61	20.46			0.0488	15.20
log (f'c lbf/in²)					451	18.37		
CAHD = (CAH - 0.5)	-26.0	-4.52	-29.2	-5.72			0.166	5.40
CAGD = (CAG - 0.5)	13.3	2.05	47.9	9.16	38.4	8.52	0.500	15.85
AIRD = (AIR - 6)			-14.0	-5.46	-11.9	-5.37		
WCD = (WC - 0.44)							1.11	2.30
CAPD = (CAP - 38)							0.0422	8.25
CSL = (CSL - 97)	15.5	11.11	5.56	3.54	-4.78	-3.61	0.0755	7.01
CAHD * CAGD	-33.5	-2.91	-48.4	-4.73			-0.380	-6.22
CAHD * AIRD			-12.7	-2.48	-8.82	-1.99	0.0766	2.51
CAHD * WCD	-322	-2.23	-512	-3.96			-4.28	-5.52
CAHD * CAPD								
CAHD * CSLD			4.79	2.26			0.0382	2.98
CAGD * CAPD							0.0439	4.28
AIRD * CAPD	-3.80	-3.93	-2.30	-2.69				
CAPD * CSLD	-1.02	-2.56					-0.00629	-2.95
constant	110	4.36	26.8	1.02	-1184	-13.46	0.661	3.29
R^2 adj.	0.886		0.910		0.830		0.936	
SEE	33.70		29.87		25.98		0.1783	
SEE df	129		127		132		125	
Pooled Rep. SD	29.0		29.0		21.9		0.197	
Rep. df	9		9		9		9	
EMS Ratio	1.4		1.1		1.4		0.8	
EMS significance	NS		NS		NS		NS	

NOTES:
1. EMS ratio = $(SEE/Rep.\ SD)^2$
2. NS = EMS ratio not significant at 10-percent significance level.
3. 1000 lbf/in² = 6.9 MPa



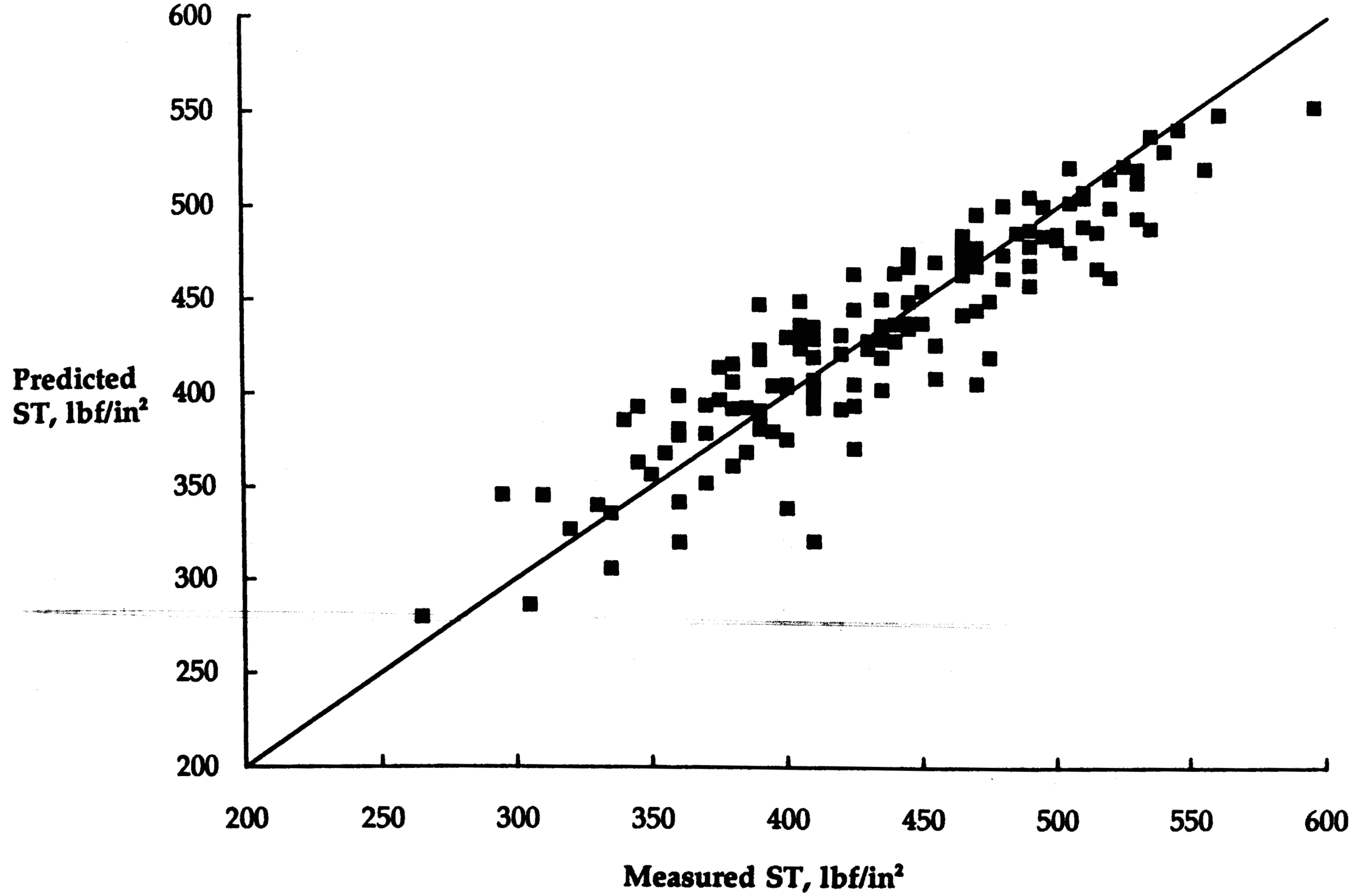
1000 lbf/in² = 6.9 MPa

Figure 31. Predicted vs. measured flexural strength (MR) as a function of splitting tensile (ST) strength (three consolidation levels).



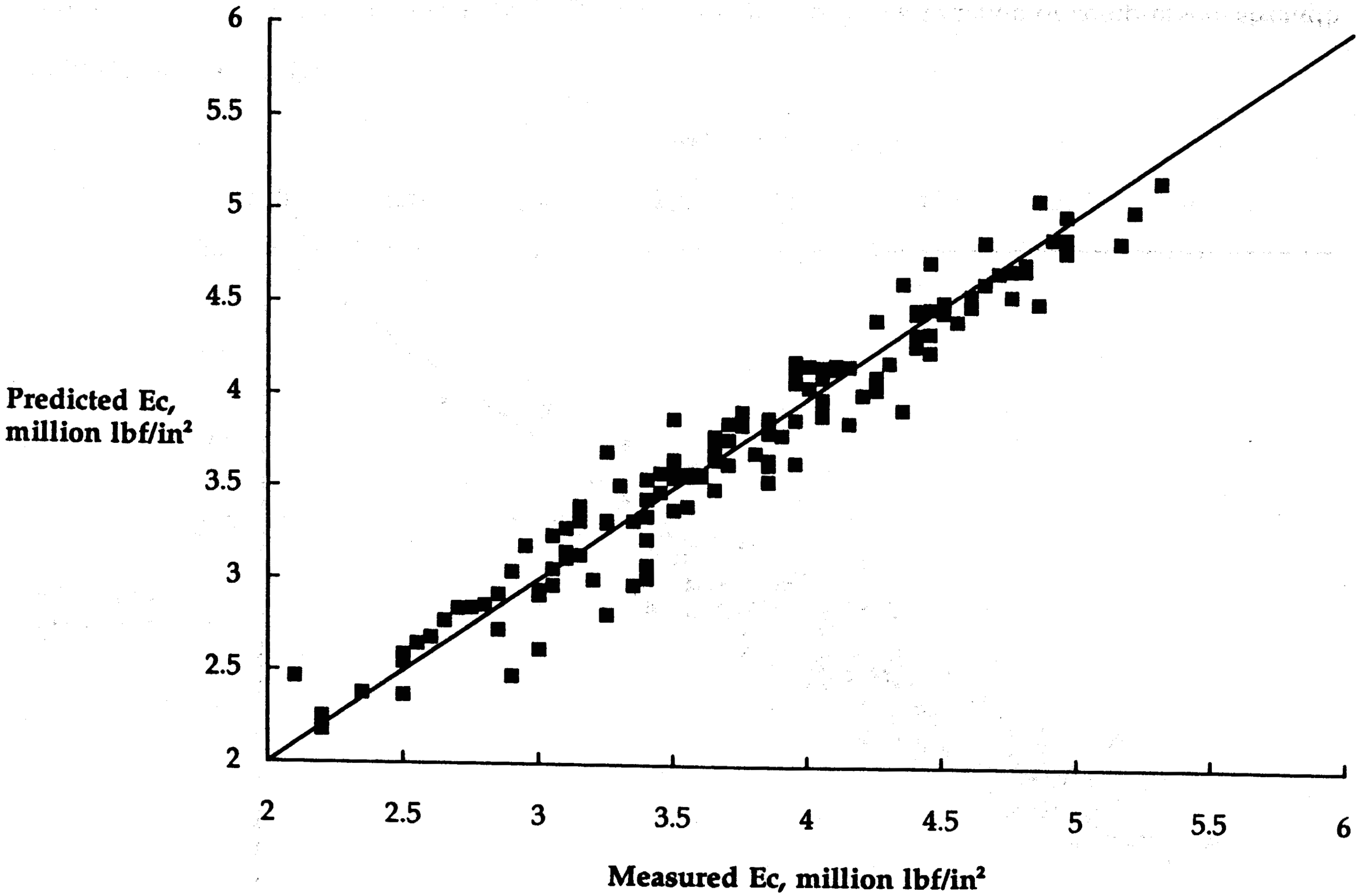
1000 lbf/in² = 6.9 MPa

Figure 32. Predicted vs. measured flexural strength (MR) as a function of compressive strength (three consolidation levels).



1000 lbf/in² = 6.9 MPa

Figure 33. Predicted vs. measured splitting tensile strength (ST) as a function of compressive strength (three consolidation levels).



1 million lbf/in² = 6.9 GPa

Figure 34. Predicted vs. measured elastic modulus (Ec) as a function of compressive strength (three consolidation levels).

For the flexural strength and elastic modulus equations, the coefficient on the consolidation level (CSLD) was positive. This indicates that as consolidation level decreases, flexural strength decreases slightly more than the reduced flexural strength inherent within the reduced compressive strength. This means that the use of the fully consolidated interstrength relationship for a core that was less than 100-percent consolidated would overestimate the *in situ* flexural strength.

The reverse case was observed when predicting splitting tensile strengths from the log of compressive strength. The coefficient on the CSLD variable was negative, indicating that the splitting tensile strength increases with a decrease in consolidation. This indicates that the effects of consolidation are more severe on compressive strength than on splitting tensile strength. In this case, the use of the fully consolidated interstrength relationship for a core that was less than 100-percent consolidated would underestimate the *in situ* splitting tensile strength.

The replicate standard deviations determined from analysis of the nine replicated mixes also is shown in table 16. Since the regression analysis combined all test data at 7, 14, and 28 d, the pooled Rep. RMS was used in evaluating the regression fit. The F-distribution was used to test for the regression variance being less than the replicate variance. All four regression equations were not significant at the 10-percent level of significance and therefore it can be inferred that all SEE are within the expected variation of the corresponding replicate standard deviations.

Since elastic modulus and splitting tensile strength can be directly measured on cores that are less than 100-percent consolidated, only the first two interstrength equations in table 16 are of further interest. It is recommended that unit weight be measured on hardened concrete cylinders during the mix design development stage, since hardened concrete density can be different than wet concrete density. If cores are to be drilled at 72 h, it is recommended that cylinder unit weights also be determined at 72 h. The hardened cylinder unit weight at the targeted air content can then be used as a baseline when evaluating consolidation levels on cores sampled from sublots. If the unit weight of cores are significantly less than the cylinders, the *in situ* consolidation is determined as the unit weight of cores divided by cylinders multiplied by 100 percent. Some engineering judgment will be necessary, since the air content of cores will likely be slightly different than the cylinders fabricated during the mix design stage. Variability in air contents should be considered when determining if there is a consolidation problem.

Once a consolidation problem has been identified, concrete maturity, as determined from *in situ* temperature monitoring, is used to determine an equivalent, laboratory moist-cured, 100-percent consolidated compressive (or splitting tensile) strength at the time of core testing. Concrete maturity is a nondestructive test method developed from data derived during the mix design stage to evaluate the combined effects of curing temperature and time on strength development. Using the less-than-fully consolidated core compressive (or splitting tensile) strength, the less-than-fully consolidated flexural strength is calculated, using relationships from table 16. Similarly, the fully consolidated flexural strength using the maturity-predicted cylinder compressive strength at the time of coring is calculated using the relationships in table 16. Since mix-specific relationships would be developed with project-specific materials, it is likely that the fully consolidated flexural strength, predicted from the cylinder compressive strength, will be significantly different from that predicted from the mix-specific interstrength relationship. It is recommended that a percentage adjustment be made to modify the mix-specific predicted strength or elastic modulus to account for consolidation.

For example, assume using mix design inputs and the following data:

Prediction Equation	Specimen Type	Consol. Level, %	Compressive Strength, MPa	Flexural Strength, MPa
table 16	cylinder	100	28 (4060 lbf/in^2)	4.5 (653 lbf/in^2)
table 16	core	95	24 (3480 lbf/in^2)	3.8 (551 lbf/in^2)
mix-specif.	cylinder	100	28 (4060 lbf/in^2)	4.4 (638 lbf/in^2)

The flexural strength predicted using table 16 decreases from 4.5 to 3.8 MPa (653 to 551 lbf/in^2), with a 5 percent decrease in consolidation level. This corresponds to a 15.6 percent decrease in flexural strength. The mix-specific interstrength relationship developed from mix design strength data predicts a slightly lower flexural strength. The 15.6-percent flexural strength decrease would be applied to the 4.4 MPa (638 lbf/in^2) strength resulting in a 95-percent consolidated flexural strength of 3.7 MPa (538 lbf/in^2). The 95-percent consolidated adjusted flexural strength would then be used in computing the pay factor reduction for the sublot.

A sensitivity analysis was conducted using the flexural strength prediction equations in table 16. For each aggregate type, the average change in flexural strength was computed for changes in consolidation level (CSLD) and in water-cement ratio (WC) for splitting tensile strengths of 2.07, 2.76, and 3.45 MPa (300, 400, and 500 lbf/in^2). The coarse aggregate volume (CAPD) was assumed to remain constant at 35 percent for both models, while the water-cement ratio was set at 0.40 for the compressive strength model. At both the low and medium levels of CSLD and water-cement ratio levels (WCD), the percentage change in modulus of rupture was computed for each of the four aggregate types as splitting tensile strength increased from 2.07 to 3.45 MPa (300 to 500 lbf/in^2). The average percent change is reported in table 17. A similar analysis, shown in table 18, was done as compressive strength was changed from 13.8 to 27.6 MPa (2000 to 4000 lbf/in^2).

As expected, significant decreases in flexural strength occurred as the consolidation level decreased from 100 to 94 percent. For all four aggregate types, the decrease in flexural strength due to a decrease in consolidation ranged from approximately 8 to 25 percent for the splitting tensile model, with larger decreases occurring at lower splitting tensile strengths. For the splitting tensile model, decreases in flexural strength were not sensitive to aggregate type or water-cement ratio. Differences in flexural strength decreases were less than 4 percent.

Table 17. Flexural strength sensitivity to split tensile strength and consolidation level analysis.

Water-Cement Ratio	Split Tensile Strength, lbf/in²	Consol. Level, %	PERCENT CHANGE IN FLEXURAL STRENGTH FROM 100% CONSOLIDATION			
			Crushed Hard Aggregate	Crushed Soft Aggregate	Rounded Hard Aggregate	Rounded Soft Aggregate
0.48	300	97	-12.5	-11.1	-12.4	-11.8
		94	-24.8	-22.2	-24.8	-23.6
	400	97	-10.1	-9.2	-10.1	-9.7
		94	-20.1	-18.4	-20.1	-19.3
	500	97	-8.5	-7.8	-8.5	-8.2
		94	-16.9	-15.7	-16.9	-16.4
0.40	300	97	-12.1	-11.4	-11.9	-12.1
		94	-24.1	-22.6	-23.9	-24.3
	400	97	-9.9	-9.4	-9.7	-9.9
		94	-19.6	-18.6	-19.5	-19.8
	500	97	-8.3	-8.0	-8.2	-8.3
		94	-16.5	-15.8	-16.5	-16.7

NOTES:
1. 1000 lbf/in² = 6.9 MPa
2. Coarse aggregate volume set at 35 percent.
3. Air content set at 5 percent.

Table 18. Flexural strength sensitivity to compressive strength and consolidation level analysis.

Air Content %	Compressive Strength, lbf/in^2	Consol. Level, %	PERCENT CHANGE IN FLEXURAL STRENGTH FROM 100 PERCENT CONSOLIDATION			
			Crushed Hard Aggregate	Crushed Soft Aggregate	Rounded Hard Aggregate	Rounded Soft Aggregate
7	2000	97	-5.6	-2.0	-5.9	-2.6
		94	-11.2	-4.1	-11.8	-4.9
	4000	97	-4.1	-1.5	-4.2	-1.6
		94	-8.0	-2.9	-8.5	-3.5
	6000	97	-3.4	-1.3	-3.5	-1.5
		94	-6.7	-2.6	-7.0	-2.8
5	2000	97	-5.3	-2.2	-5.5	-2.3
		94	-10.5	-4.1	-11.1	-4.9
	4000	97	-3.9	-1.4	-3.9	-1.8
		94	-7.8	-3.1	-7.9	-3.5
	6000	97	-3.2	-1.2	-3.4	-1.3
		94	-6.5	-2.6	-6.7	-2.8

NOTES:
1. 1000 lbf/in^2 = 6.9 MPa
2. Water-cement ratio set at 0.40.
3. Coarse aggregate volume set at 35 percent.

For all four aggregate types, the percent decrease in flexural strength due to a decrease in consolidation ranged from approximately 1 to 12 percent for the compressive strength model. Similar to the splitting tensile strength model, larger flexural strength percentage decreases occurred at lower compressive strengths. Flexural strength decreases due to consolidation decreases were not too sensitive to air content, with differences in strength decreases of less than 2 percent. Larger decreases were noted for the harder coarse aggregates, ranging up to approximately 7 percent higher than decreases for softer coarse aggregates.

As indicated in table 16, air content is a significant variable in predicting flexural strength only as a function of square root of compressive strength. The coefficient on the AIRD variable was negative, indicating that as air content increases, the predicted flexural strength decreases. This suggests that the effect of air content, reflected in compressive strength, is not completely accounted for in the prediction of flexural strength. The AIRD coefficient was not significant when predicting flexural strength as a function of splitting tensile strength. This indicates that the effect of air content on flexural strength is completely reflected when it is estimated based on splitting tensile strength.

It is unlikely that sublot air contents will always be identical to the air content used in the development of the mix-specific, interstrength relationships. To account for the effects of varying air content levels on the interstrength relationships, an analysis similar to that conducted on the effects of varying consolidation levels could be conducted to determine appropriate adjustments to account for the effects of air content.

There is some inherent variability associated with projecting 28-d, standard- cured compressive or splitting tensile strengths from strengths measured at some earlier time. There is also some inherent variability in the mix-specific interstrength predictions. To reduce the sources and magnitude of variability, it is recommended that mix-specific interstrength relationships be developed at several air contents during the mix design process. The predicted sublot flexural strength as a function of the compressive or splitting tensile strength and air content can then be used in determining pay factors.

Core/Cylinder Strength Relationship Study

The PRS recommends drilling cores from each sublot to evaluate the *in situ* thickness, density, air content, and strength. It is a commonly held belief that cores are approximately 85 percent of the compressive strength of cylinders. However, this relationship was developed for mature structural concrete. If interstrength relationships, used to determine *in situ* flexural strength, are based on cylinder data, it is important that relationships between cores and cylinders at ages of less than 28 d be established to account for damage during coring, and differences in specimen size, material proportions, and mix components.

To develop a cylinder and core relationship, cylinders were cast and cured under standard conditions. These cylinders were cored at 7, 14, and 28 d to produce a 102-mm (4-in) diameter by 203-mm (8-in) long core. Cores and cylinders cured to the same maturity were then tested for compressive strength. Since the cylinders and cores were cured to the same maturity, any differences in the test results could be attributed to coring damage and size difference effects.

A total of eight different mixes were evaluated. These eight mixes were obtained using the two levels of coarse aggregate hardness (CAH), two levels of coarse aggregate geometry (CAG), two levels of water-cement ratio (WC), and two levels of coarse aggregate volume (CAP). The half-fractional factorial experimental design for this study is shown in table 19.

Table 19. Factorial design for coring strength-cylinder strength study.

				COARSE AGGREGATE HARDNESS			
				Soft		Hard	
				Coarse Agg. Geometry		Coarse Agg. Geometry	
				Rounded	Crushed	Rounded	Crushed
Water-Cement Ratio	0.40	Coarse Agg. Quantity	Low		5	9	
			High	6			16
	0.48	Coarse Agg. Quantity	Low	14			19
			High		10	8	

Note: Only one level of water volume (14.1 percent) and air content (5 percent) were evaluated.

Similar to the strength and consolidation study, the WC was set at 0.40 and 0.48. The selected WC values correspond to cement volumes (CEP) of 9.3 and 11.1 percent, respectively, and a water volume (WAP) of 14.1 percent. Cement contents were 292 kg/m^3 (492 lb/yd^3) and 350 kg/m^3 (590 lb/yd^3). Air content (AIR) was set at the 5-percent level.

It is assumed that the effects of water and cement quantity are reflected in the age at testing. The effects of consolidation level and air content on core versus cylinder strengths were not investigated, since it was assumed that these effects on strength are similar regardless of specimen type.

The concrete mixes used for the cylinders and cores were made at the same time that the strength and consolidation study specimens were made. A total of three cores and three cylinders were tested at each age for the eight different mixes. Cores were drilled in accordance with ASTM Designation C42, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete." After drilling cores, the cores were sawed to a length of 203 mm (8 in), which was tested at the as-cored moisture condition. Concrete core data and cylinder data are listed in table 14 of appendix D.

The average 28-d cylinder strength ranged from 34.6 to 46.1 MPa (5020 to 6680 lbf/in^2), with a mean of 39.7 MPa (5760 lbf/in^2). The average 28-d core strength ranged from 33.9 to 46.0 MPa (4920 to 6670 lbf/in^2), with a mean of 39.6 MPa (5740 lbf/in^2). Core-to-cylinder compressive strength ratios (average of triplicates) ranged from 0.92 to 1.13, and averaged 1.00 over all testing ages. On average, no significant differences between core and cylinder strengths were noted.

The data from all eight mixes shown in table 19 were grouped into one data set combining all data collected at each age. In addition to the four-mix design property and testing age main effects, two-factor interactions were also used. Each main factor (independent variable) was transformed by subtraction of the level mean to reduce intercorrelations between interaction variables. A stepwise regression procedure was used to identify significant main and interaction effect variables. The threshold for adding and deleting variables was set at the 5-percent significance level. The following prediction equation was developed to predict cylinder compressive strength from core compressive strength:

$$\begin{aligned} \text{Cyl. } f'c \ = \ & 0.997 * \text{Core } f'c + 242.96 * (CAH - 0.5) + 309.56 * (CAG - 0.5) \\ & + 795.59 * (CAP - 38) * (WC - 0.44) \end{aligned} \tag{16}$$

where:

Cyl. $f'c$ = Cylinder compressive strength, lbf/in^2
Core $f'c$ = Core compressive strength, lbf/in^2
CAH = Coarse aggregate hardness (0=soft, 1=hard)
CAG = Coarse aggregate geometry (0=rounded, 1=crushed)
CAP = Coarse aggregate volume, percent
WC = Water-cement ratio

R^2 = 0.999
SEE = 203.9

The constant term was not significant at the 5-percent significance level.

Since the regression analysis combined all test data at 7, 14, and 28 d, the pooled Rep. RMS was used in evaluating the regression fit. The pooled Rep. RMS for compressive strength was 2.40 MPa (348.7 lbf/in^2). The error mean square ratio, computed as the square of the regression standard error of estimate (SEE) divided by the Rep. RMS, was 0.34. The F-distribution was used to test for the regression variance being less than the replicate variance. The regression equation was not significant at the 10-percent level of significance, and it therefore can be inferred that the SEE is within the expected variation of the corresponding replicate standard deviations.

A sensitivity analysis was done on the above regression equation for compressive strengths ranging from 27.58 to 44.82 MPa (4000 to 6500 lbf/in^2) at 3.5 MPa (500 lbf/in^2) increments. This range was approximately that used in deriving the regression equation. The core-to-cylinder ratios ranged from 0.92 to 1.11, and averaged 1.0 for the 96 sensitivity predictions. Of the 96 predictions, 52 percent had a core-to-cylinder ratio of 0.97 to 1.03. The percentage increased to 68 percent between 0.96 and 1.04. The sensitivity analysis indicated that although a statistically significant prediction equation could be developed (as evaluated by the coefficient of determination and standard error of estimate), the ratio of core strength to cylinder strength did not vary significantly from 1.0.

A similar stepwise regression analysis was conducted to determine mix design parameter influences on the core-to-cylinder ratio. In addition to the four-mix design property and testing age main effects, two-factor interactions were also used. Similar to the analysis to predict cylinder strength from core strength, only coarse aggregate hardness (CAHD), coarse aggregate geometry (CAGD), and coarse aggregate volume-water-cement ratio (CAP*WC) were significant in predicting the strength ratio. The coefficient of determination was very low (0.534). It was concluded that a satisfactory regression equation to differentiate between the core and cylinder strengths as a function of mix parameters could not be developed.

Since the cylinder strength regression, the cylinder ratio regression, and the measured data indicated a relatively low sensitivity and appear to be clustered around a ratio of 1.0, the data as a whole (irrespective of mix design levels) were analyzed. A paired t-test was then done on the data to determine if there was a statistically significant difference between matched pairs of core and cylinder strengths. For the 24 sets of matched pairs, the t-value was 0.379. The hypothesis that there is a statistically significant difference between core and cylinder strengths could not be rejected at the 10-percent significance level. The compressive strength of cores and cylinders is shown in figure 35.

Examination of the distribution of the core-to-cylinder ratios showed an approximate bell-shaped distribution around 1.0. The 24-point, relative frequency histogram is shown in figure 36.

Under laboratory controlled conditions, on average, there is no significant difference between cores and cylinders. If *in situ* strength is developed from core strengths, it is important to exercise care in coring. Core rigs should be stabilized to minimize damage to cores, the type of barrel should be matched with the coarse aggregate type, and coring should be done by experienced operators.

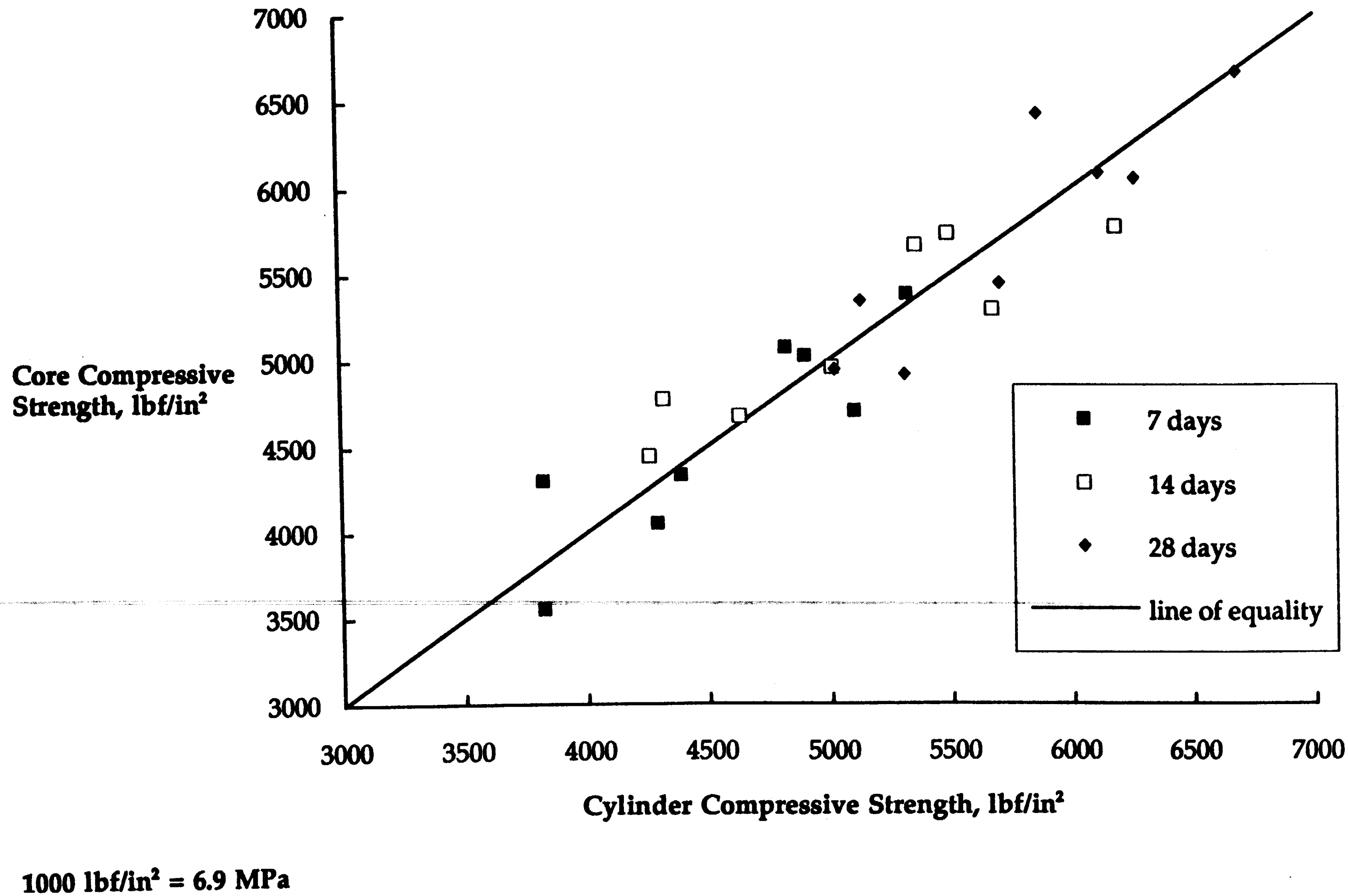


Figure 35. Core vs. cylinder strengths.

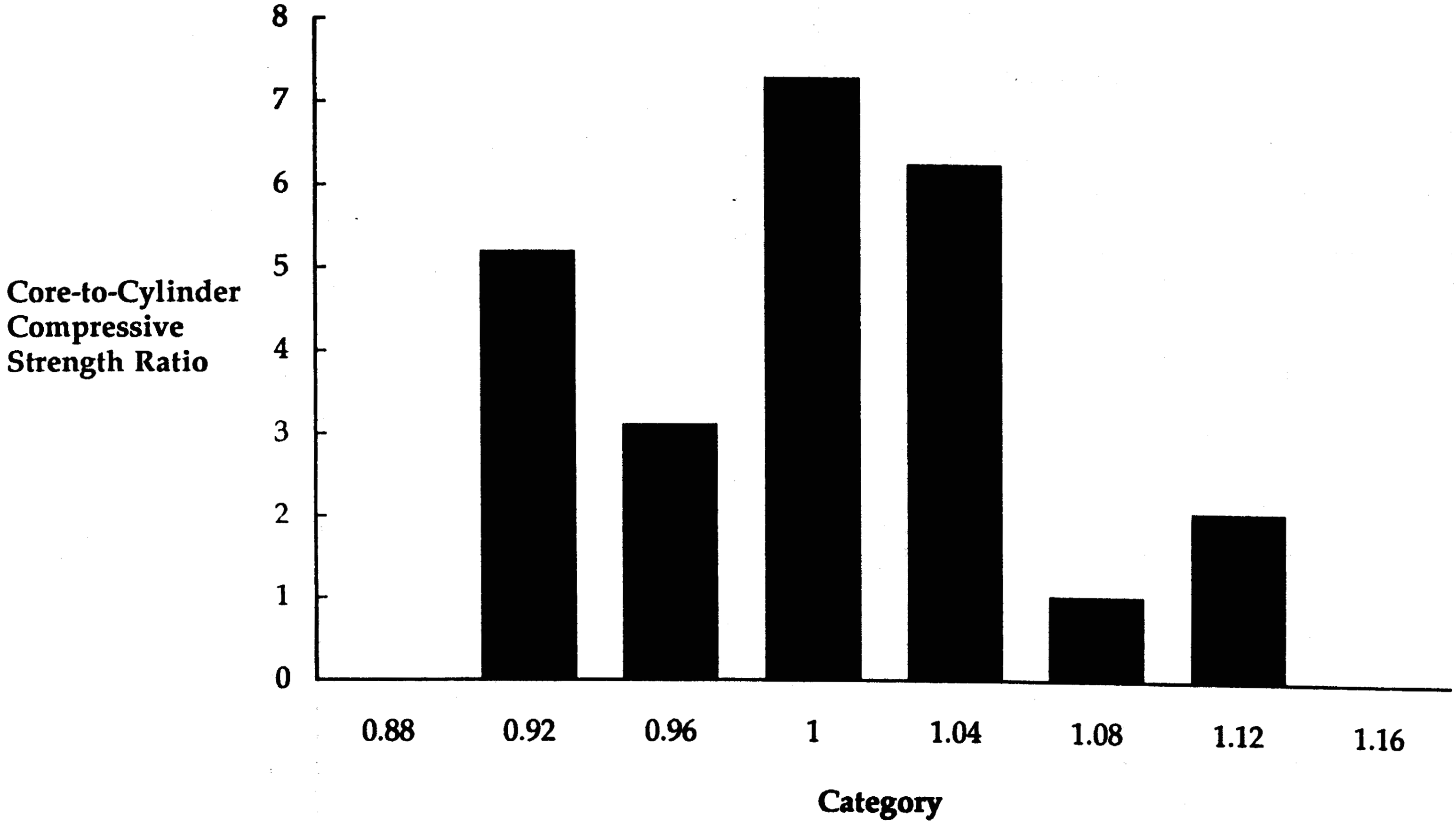


Figure 36. Relative frequency histogram of core-to-cylinder strength ratios.

There is a significant increase in within-test variability of strengths obtained from cores as compared to strengths obtained from cylinders. Standard deviations at all ages were higher for the three cores than the three cylinders. The average standard deviation of cores for the 8 mixes was 83 percent higher than that for cylinders. Based on higher within-test variability, it is recommended that more than one core be used to establish sublot strength. Retesting provisions were also established to provide equitable procedures for establishing sublot strength.

Nondestructive Testing Demonstration

The prototype PRS calls for *in situ* slab strength measurements to be used instead of standard-cured specimens. Whereas standard-cured field specimens are independent of curing temperature and time effects (since both of these factors are set by specifications), *in situ* strengths are primarily affected by these factors. In order to use *in situ* measurements, concrete maturity must be used to extrapolate from the time of core testing to an equivalent 28-d laboratory moist-cured strength. However, an initial adjustment is first needed to convert from a concrete core compressive or splitting tensile strength to a flexural strength using mix-specific relationships developed during the mix design process.

In addition to concrete maturity, another NDT method for monitoring strength development is the use of pulse velocity. With the recent trends toward earlier opening of concrete pavements, these methods can quickly and easily be used to monitor strength development during the initial stages of construction to allow the contractor to make mix design adjustments early on during construction.

Concrete maturity is a nondestructive test for estimating concrete strength based on both curing temperature and time. Maturity concepts have been proposed and used since the 1950's to monitor and estimate strength gain. Once a mix-specific relationship is established between strength gain and the accumulated time-temperature effects, concrete strength gain during construction can be monitored. Two methods of expressing maturity are proposed in ASTM Designation C1074, "Standard Practice for Estimating Concrete Strength by the Maturity Method." Maturity can be expressed in terms of a time-temperature factor or in terms of equivalent age at a specified temperature. Maturity in terms of a time-temperature factor is computed from the temperature history as follows:

$$M = \Sigma (T - T_o) \Delta t \tag{17}$$

where:

M	=	Maturity at age t, in degree-hours or degree-days
T	=	Average concrete temperature during time interval
T_o	=	Datum temperature
Δt	=	Time interval, h or d

Equation 17 is commonly called the Nurse-Saul maturity or the time-temperature factor maturity function. The units used in ASTM C1074 are °C-h (or d). The datum temperature is the temperature below which the concrete ceases to gain strength. For concrete with type I cement without admixtures and a curing range of 0 to 40 °C (32 to 104 °F), a datum temperature of 0 °C (32 °F) is recommended by ASTM. American Concrete Institute (ACI) publication ACI 306R, "Cold Weather Concreting," recommends using -5 °C (23 °F) for mixes with type I cement cured at 0 to 20 °C (32 to 68 °F).[13] The datum temperature is sensitive to the type and quantity of cement, type and quantity of liquid and mineral admixtures, and curing temperatures. Significant prediction errors may occur if datum temperatures are assumed. Since maturity will be used to evaluate 28-d flexural strengths at earlier ages using a maturity-adjusted interstrength relationship, the datum temperature must be experimentally determined over a range of curing temperatures expected during construction. Procedures to experimentally determine the datum temperature for a specific mix are outlined in ASTM C1074.

Based on investigations of mortar specimens, it is recognized that hardening of concrete is not a linear function of curing temperature. The predicted strength of concrete using the linear Nurse-Saul function can deviate from the actual strength for temperatures less than -5 °C (23 °F) and greater than 30 °C (86 °F). In the late 1970's, the equivalent age maturity equation was proposed based upon the Arrhenius equation. The Arrhenius equation is used to quantify cement hydration as a nonlinear acceleration of chemical reactions that increases in temperature. The equivalent age Arrhenius equation is shown below:

$$t_e = \Sigma \Delta t \ \exp(-E / R \times T') \tag{18}$$

where:

t_e = Equivalent age at a specified temperature, days or hours
E = Activation energy, J/mol
R = Universal gas constant
= 8.3144 J/(mol-°K)
T' = $[1 / (273 + T) - 1] / [(273 + T_s)]$
T = Average concrete temperature during time interval Δt, °C
T_s = Reference temperature, °C
Δt = Time interval, h or d

The exponential equation is a function of the absolute temperature. The degree of nonlinearity is dependent on the activation energy (E) that in turn is a function of the temperature, the cement type, and the admixture type and content. For concrete temperatures of 10, 22, and 38 °C (50, 72, and 100 °F), suggested values for the activation energy divided by the universal gas constant (E/R) are 5797, 4029, and 4029 °K, respectively.[14] For type I cement without admixtures or additives, values of the activation energy divided by the gas constant can range from 4811 to 5412 °K. It is suggested by ASTM that the activation energy divided by the gas constant can be reasonably approximated as 5000 °K. Similar to datum temperature calculations, significant prediction errors may occur if the activation energy is assumed. Procedures are outlined in ASTM C1074 to experimentally determine the activation energy.

The pulse velocity method consists of measuring the travel time of a compression wave through concrete. By assuming a direct travel path length, the velocity in m/s (ft/s) can be computed. ASTM C597, "Standard Test Method for Pulse Velocity Through Concrete," describes the test and states that it is intended to be used to assess concrete uniformity and relative quality, and should not be considered as a means of measuring strength or modulus of elasticity. However, under certain circumstances, a velocity-strength or velocity-elastic modulus relationship may be established to serve as a basis of estimating strength or modulus of elasticity.

Pulse velocity testing generates compression waves that are transmitted through the concrete by a transducer held in contact with the surface. The pulses are received by a receiving transducer and the time taken by a pulse to travel through the concrete is accurately measured and digitally displayed in 0.1 µs. Commercially available pulse generators are battery powered and portable, measuring approximately 178 by 114 by 165 mm (7 by 4.5 by 6.5 in). Electromechanical transducers are 50 mm (1.97 in) in diameter by 42 mm (1.65 in) long, with resonant frequencies of 54,000 Hz. Transducer contact is enhanced by using a very thin couplant medium, such as grease, oil, petroleum jelly, flexible sealant, or kaolin-glycerol paste.

Transducers are arranged on concrete surfaces in three basic configurations: direct, semi-direct, and indirect. The direct transmission is the preferred method of testing. The transducers are positioned so that the pulse travels directly through the concrete, such as at either end of a cylinder. The semi-direct method is used when access to geometrically parallel faces of the specimen is not possible, and might be done on a newly placed concrete pavement by positioning one transducer on the surface and the other on the exposed side of the slab. The indirect method or surface transmission places the two transducers on the same surface, but at the prescribed distance from one another. This is the least satisfactory transducer arrangement because the pulse amplitude is only about 1 to 2 percent of that detected for the same path length when direct transmission is used.

Pulse velocity is computed as the measured path length (m or ft) divided by the transit time (seconds). Manufacturer-recommended accuracy in measuring path lengths and travel times is $\pm$ 1 percent.

The compression wave velocity for a homogeneous, isotropic elastic medium is theoretically expressed as:

$$PV = \sqrt{\left(\frac{E}{D}\right) \frac{(1-\mu)}{(1+\mu)(1-2\mu)}} \tag{19}$$

where:

PV	=	Compressional wave velocity, m/s
E	=	Dynamic elastic modulus, Pa
D	=	Unit weight, kg/m^3
μ	=	Poisson's ratio
	=	0.15 (typically)

Since the elastic modulus has been empirically correlated with concrete strength properties and the modulus is related to pulse velocity, strength can be estimated directly from pulse velocity. Several studies have demonstrated that pulse velocity for a specific mix can be used to monitor strength gain.

Twelve of the forty-six mixes prepared for the strength and consolidation study were evaluated using pulse velocity and maturity. Since maturity is a function of admixtures used, cement source, and cement type, the evaluation was only a demonstration of how to develop relationships between strength and NDT and how to use the relationships to adjust core strengths to equivalent 28-d, laboratory-cured strengths. Expected prediction errors in estimating strength were also established. Maturity and pulse velocity were measured at intervals of 1, 3, 5, 7, 14, and 28 d. NDT was done on the same 12 mixes where beams and cylinders were tested to demonstrate developments of relationships between strength types and NDT.

Maturity was calculated from temperatures recorded using a portable temperature logger. Air temperature and concrete specimen temperature were automatically measured with thermocouples and printed every half hour. Monitoring was terminated when specimen temperatures stabilized at the isothermally controlled curing room temperature. Due to specimen size differences causing differing temperature histories for approximately the first 24 h, maturity was calculated for both beams and cylinders. Maturity was greater for beams than cylinders due to higher peak heat of hydration and slower cooling down to isothermal laboratory ambient temperatures. By correlating maturity with corresponding strength for individual mixes, estimates of strength were generated by simply recording the curing time and temperature histories. Maturity was then correlated with the average strength and elastic modulus data listed in tables 7 and 8 of appendix D. Maturity data used in the analysis is listed in table 15 of appendix D.

Pulse velocity was measured longitudinally on every specimen tested. Three pulse velocity tests were conducted at the prescribed ages for the three specimens of each mix. The average pulse velocities were then correlated with the average strength and elastic modulus data listed in tables 7 and 8 of appendix D. Pulse velocity data used in the analysis is listed in table 15 of appendix D.

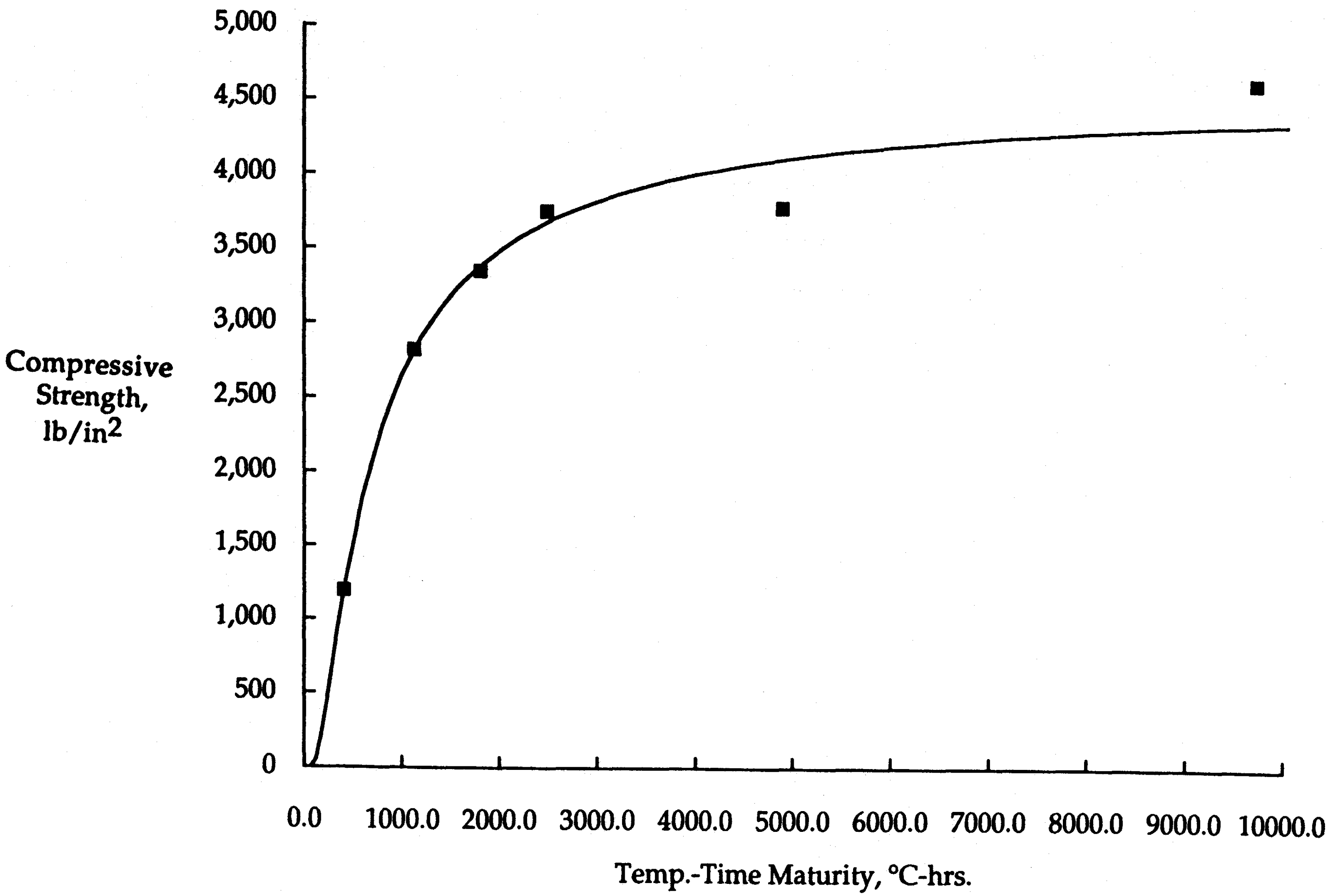
The datum temperatures and activation energies were experimentally determined on mortar cube specimens as described in ASTM C1074. Fine aggregate volumes were a covariable in the experimental mix design, since cement, coarse aggregate, and air volumes were experimental design levels. Water volume was kept constant throughout the laboratory program. The 12-mix average fine aggregate volumes for the lower and upper cement contents were used in the mortar mixes. Air-entraining admixtures were added in dosages to provide an average of 5- and 7-percent air content to the mortar.

Cube tests were conducted to establish datum temperatures and activation energies for the 0.40 and 0.48 water-cement ratio mixes. The cube water bath curing temperatures were 7, 23, and 39 °C (45, 73, and 102 °F). The datum temperatures used in the maturity calculations were 8.0 and 4.5 °C (46 and 40 °F) for the 0.48 and 0.40 water-cement ratio, respectively. The E/R constants (activation energy divided by the universal gas constant) used in the Arrhenius maturity calculations were 6285 and 5940 °K for the 0.48 and 0.40 water-cement ratios, respectively.

Of the eight mixes prepared at 100-percent consolidation, five were at the 0.40 water-cement ratio and three were at the 0.48 water-cement ratio. The remaining four mixes were selected at each of the two lower consolidation and two water-cement ratio combinations. Average data (triplicate specimens) in tables 7 and 8 of appendix D were used in developing strength-NDT relationships. For each combination of the four hardened concrete properties and three NDT types (two maturity and one pulse velocity method), a regression equation was developed. Dependent and independent variables were transformed using square root, logarithmic, and inverse transformations. The interstrength model with the highest R^2 was selected as the most representative interstrength model.

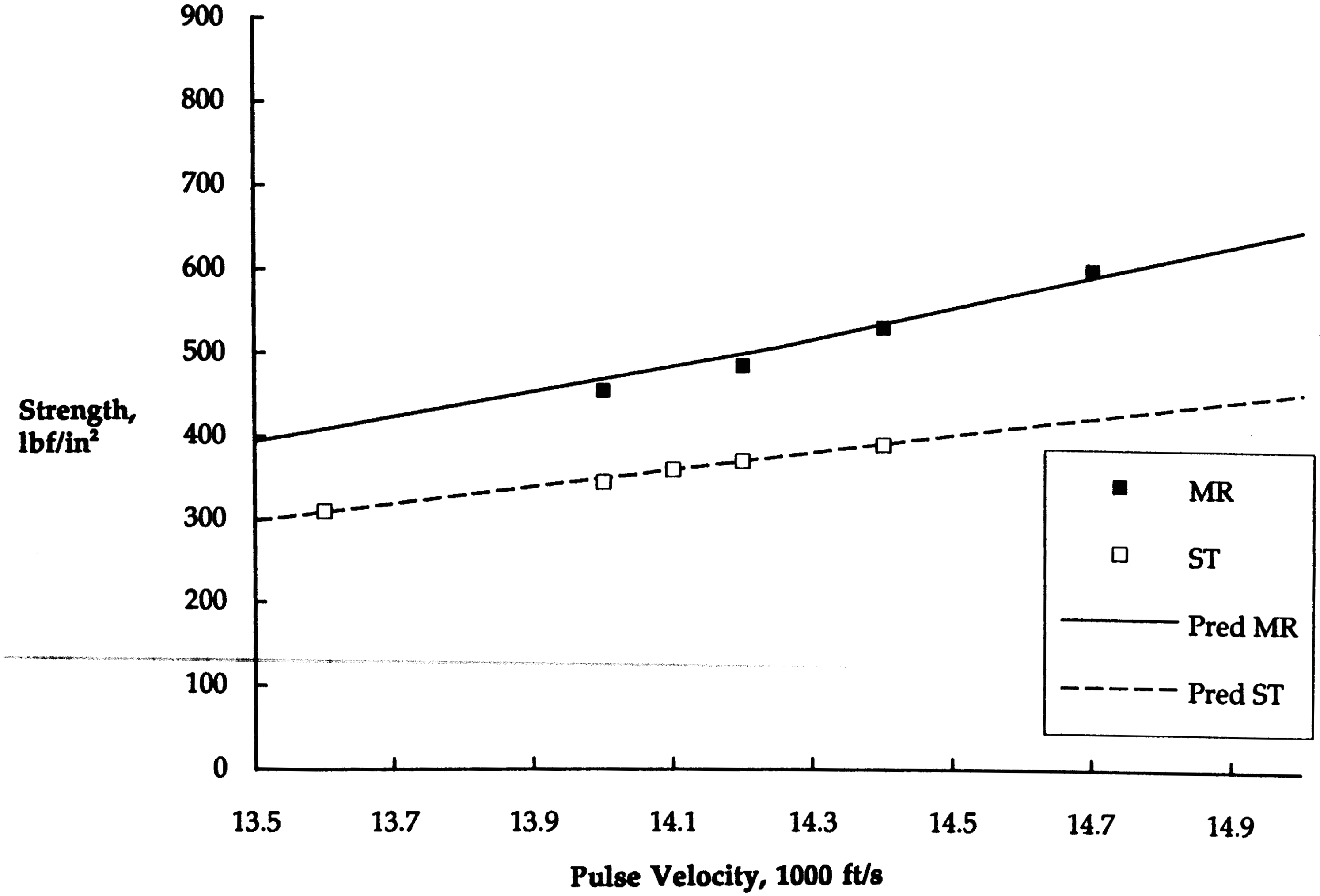
For the 144 strength and elastic modulus prediction models developed, the inverse strength-inverse NDT and log strength-inverse NDT were selected most frequently. Less common were square root transformations or no transformations of variables. For all mixes, the three types of NDT relationships (two maturity and one pulse velocity) predicting flexural, splitting tensile, compressive strength, and elastic modulus were judged to be a very good fit. The coefficients of determination for all of the 144 regression equations developed ranged from 0.953 to 1.0, and averaged 0.987. Regression equations developed for the 12 mixes for predicting flexural strength, splitting tensile strength, compressive strength, and elastic modulus as a function of Nurse-Saul maturity, Arrhenius maturity, and pulse velocity are listed in table 16 of appendix D.

On all four hardened concrete properties, the coefficients of determination for the prediction of strength and elastic modulus averaged 0.989, 0.987, and 0.987 for the pulse velocity, Nurse-Saul, and Arrhenius maturity methods, respectively. For all three NDT types, the average coefficients of determination were 0.990, 0.991, 0.988, and 0.982 for the prediction of flexural strength, compressive strength, splitting tensile strength, and elastic modulus, respectively. Compressive strength as a function of Nurse-Saul maturity is shown in figure 37, illustrating the type of individual mix prediction errors that can be expected from a laboratory-developed maturity curve. An example of prediction errors using the pulse velocity method is shown in figure 38.



1000 lb/in^2 = 6.9 MPa

Figure 37. Predicted and measured compressive strength as a function of measured maturity.



1000 lbf/in^2 = 6.9 MPa
1000 ft/s = 305 m/s

Figure 38. Predicted and measured flexural (MR) or splitting tensile (ST) strength as a function of pulse velocity measurements.

An error analysis was done on the maturity NDT prediction method, since it will be used to standardize the core compressive strength to 28 d. The eight mixes at 100-percent consolidation were used in the error analysis. Since compressive strength will probably be the most common core test method and the coefficients were all very high regardless of the test method or property, compressive strength prediction errors using both the Nurse-Saul and Arrhenius maturity methods were evaluated. For all 12 mixes, the overall average absolute error for all ages was 3.1 percent, with no trend in prediction error with testing age.

Based on the error analysis and the R^2 values for the laboratory-generated maturity equations, it was concluded that the maturity method could be used to adjust core compressive strengths. The error analysis is summarized in table 20.

The maturity functions developed during the mix design stage will be used to adjust core compressive (or splitting tensile) strength to a 28-d (or specified) laboratory moist-cured compressive (or splitting tensile) strength. The standardized core strength will then be converted to a flexural strength using the mix-specific flexural prediction equation also developed during the mix design stage. The procedures for making maturity adjustments are as follows:

1. Maturity relationships are developed during the mix design process. The number of testing ages should be sufficiently large to confidently predict compressive (or splitting tensile) strength from maturity. It is recommended that testing ages be extended past 28 d to ensure that the field maturity at time of coring falls within the range from which the maturity prediction equation was developed.

2. *In situ* maturity is monitored from the start of construction for each sublot with automatic data loggers or commercially available maturity meters. Since cores represent an average slab strength, it is recommended that temperatures be monitored at three elevations (top, middle, and bottom), each located a sufficient distance from slab edges or joints. It is recommended that the average temperature be used to calculate *in situ* maturity.

3. Cores are sampled from sublots following the specified sampling procedures. Compressive or splitting tensile tests are conducted in accordance with prescribed test methods and time limitations. Core strengths are modified in accordance with ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," to account for length-to-diameter ratios of less than 1.8:1. Based on the cylinder and core study, there are no adjustments needed to convert strengths obtained from 102-mm (4-in) diameter cores to an equivalent compressive strength obtained from 152-mm (6-in) diameter cylinders.

4. Compressive strengths at the time of coring are extrapolated to 28 d. For example, assume that the maturity when cored at 7 d is 3733 °C-h (6720 °F-h). From the mix design development, maturity of 28-d laboratory cylinders is 14,933 °C-h (26,880 °F-h). Therefore, all core compressive strengths must be extrapolated an additional 11,200 °C-h (20,160 °F-h).

Table 20. Percent compressive strength maturity prediction error for 100-percent consolidation mixes.

	Percent Error in f'_c Predicted from Nurse-Saul Maturity						Percent Error in f'_c Predicted from Arrhenius Maturity					
Mix No.	1 day	3 days	5 days	7 days	14 days	28 days	1 day	3 days	5 days	7 days	14 days	28 days
1	0.5	3.0	1.7	1.8	5.1	2.3	0.8	3.9	1.1	2.0	5.7	1.1
2	0.1	0.3	0.9	2.1	8.5	6.2	0.0	0.7	0.7	2.1	8.7	5.9
5	0.6	0.3	6.3	0.2	3.0	4.0	0.5	0.1	5.9	0.0	2.7	3.4
6	2.4	8.5	3.0	8.0	2.4	3.0	2.1	8.5	2.9	8.1	2.5	3.2
16	2.3	3.5	2.8	2.8	1.5	2.6	2.8	3.4	3.0	3.0	1.5	2.8
19	1.7	5.7	0.0	0.7	5.6	1.9	1.9	6.2	0.3	0.6	5.8	1.4
20	0.5	4.2	7.3	11.5	4.6	6.8	0.7	4.8	6.8	11.6	5.2	8.2
21	0.4	2.8	0.9	0.2	2.3	0.2	0.3	2.2	1.4	0.0	1.9	1.1
Avg.	1.1	3.5	2.9	3.4	4.1	3.4	1.1	3.7	2.7	3.4	4.2	3.4
Max.	2.4	8.5	7.3	11.5	8.5	6.8	2.8	8.5	6.8	11.6	8.7	8.2
Min.	0.1	0.3	0.0	0.2	1.5	0.2	0.0	0.1	0.3	0.0	1.5	1.1

5. Assume, for this same mix, the maturity equation is:

$$\log f'_c = -278 / NS + 3.7456 \qquad (20)$$

where:

f'_c = Compressive strength, lbf/in^2
NS = Nurse-Saul maturity, °C-hours

Assume also that a core on this project has a compressive strength of 33.1 MPa (4800 lbf/in^2) at a 7-d maturity of 4320 °C-h (7775 °F-h). If the strength development keeps continuing from 33.1 MPa (4800 lbf/in^2) at the same rate as a laboratory-cured cylinder, the strength must be extrapolated out an additional 11,200 °C-h (20,160 °F-h) from 4320 °C-h (7775 °F-h). At 15,520 °C-h (27,936 °F-h), the predicted strength is 36.9 MPa (5340 lbf/in^2).

6. The 28-d compressive strength is converted to a 28-d flexural strength using the laboratory mix design flexural-compressive strength relationships.

The above procedure assumes that *in situ* compressive strength will continue to develop at the same rate as the laboratory-cured cylinder strength from the backcalculated maturity. The backcalculated maturity is defined as that maturity corresponding to the core compressive strength. The laboratory maturity curve must be adjusted to the as-tested core compressive strength. *In situ* strength gain is then assumed to be the same as the adjusted laboratory maturity curve.

Using the above procedure, data from the eight maturity NDT mixes were used to extrapolate 7-d cylinder compressive strengths to equivalent 28-d beam flexural strengths. First, the cylinder compressive strength was extrapolated from 7 to 28 d using mix-specific compressive strength functions. Then, the 28-d flexural strength was estimated from the mix-specific interstrength relationships between compressive and flexural strength. For the time-temperature maturity method, extrapolation errors ranged from 0.1 to 5.0 percent, and averaged 1.8 percent (absolute value). For the equivalent age maturity, the extrapolation errors for the eight mixes ranged from 0.1 to 4.6 percent, and averaged 1.6 percent.

Due to changes in mixes, curing temperatures (or temperature magnitude), solar radiation effects, and numerous other variables, this procedure may not adequately predict extrapolated strength during construction. Therefore, it is highly recommended that field-cured cylinders be used to verify the laboratory-developed, maturity-strength curve. Since maturity is to be monitored on each sublot, the only extra effort is field fabrication and testing of cylinders. If enough cylinders are tested during construction, a new maturity curve can be easily developed, if necessary, to be used on individual paving jobs.

Even if an agency were to require 28-d core strengths, the use of NDT can at least allow the contractor to rapidly estimate standardized strengths to permit changes during construction. Either the pulse velocity or maturity method can be used to monitor strength development. If *in situ* pulse velocity testing is to be done, rigid insulation blockouts staked to the subbase prior to paving and removed after final set provide easy access for direct transmission testing. *In situ* strength is best monitored in the slab interior away from free edges. Since velocity is a function of water content at earlier ages, the blockouts should be spaced such that travel distances are the same as the specimen travel path from which the regression analysis is developed.

INVESTIGATION OF CONCRETE FREEZE-THAW DURABILITY

To address durability problems in the development of PRS, a laboratory testing program was conducted to correlate air-void system parameters with joint spalling. It was assumed that the degree of spalling will directly influence pavement performance. Numerous mixes were evaluated to relate spalling with the critical air-void system parameters shown in table 21. These parameters must meet commonly accepted minimum standards to be durable in F-T environments. The total air content (made up of both entrained and entrapped air) is easily measured in the plastic state using pressure meters. However, for hardened concrete, the four air-void system parameters must be measured using petrographic analysis.

With modern paving equipment, good material control, improved admixtures, and frequent inspection, the problem of inadequate air-void systems are uncommon. However, because of the severe consequences that an inadequate air-void system can have on pavement life, and because the mix design is under the control of the contractor, it was deemed appropriate to include it as part of the PRS. The objective of the laboratory durability investigation was to develop a model relating the degree of spalling (durability) to air-void system parameters and number of F-T cycles.

Laboratory Test Procedure

The procedure used to evaluate spalling was adopted from ASTM C666, "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing." Procedure B was followed, where the specimens are frozen in air and thawed in water. This is more realistic of actual environmental conditions than procedure A, in which specimens are frozen and thawed in water. The procedure was modified by using block sections installed with joints to evaluate the progression of joint deterioration.

Specimens were 305-mm wide by 381-mm long by 152-mm deep (12 in by 15 in by 6 in). Joints, 3.2-mm (0.12-in) wide and 64-mm (2.5-in) deep, were cut transversely across the 305-mm (12-in) wide specimen. Sawcuts were made with a diamond-impregnated sawblade at times when the temperature, monitored with thermocouples at mid-depth, indicated a temperature drop from peak heat of hydration temperature. Since joint sawing is normally done near peak temperatures to avoid random cracking due to thermal contraction and curling restraint stresses, the degree of any unobservable sawing damage was no greater than what would be

Table 21. Critical parameters of the concrete air-void system.

Air content (A)—Total volume of air voids in the cement paste matrix of the concrete, expressed as a percentage by volume of concrete. There is a distinction between entrapped air (> 1 mm [0.04 in]) and entrained air. Required air content is a function of maximum aggregate size.
Voids per inch (η)—Average number of voids per inch of concrete surface. Generally, it is desired that η (in voids per inch) be at least twice the amount of entrained air (in percent).
Specific surface (α)—Calculated average surface area of voids per unit volume of air. Generally, α should be greater than 23 mm^2/mm^3 (600 in^2/in^3).
Void spacing factor ($\bar{L}$)—Average distance between individual air bubbles in the cement paste matrix. Generally, $\bar{L}$ should be less than 0.20 mm (0.008 in).

1 in = 25.4 mm

expected during construction. To simulate field conditions, the surfaces of the specimens prior to sawing were sealed with a clear sealer compound. The joints were widened with an abrasive sawblade at 24 h to form a 9.5- by 9.5-mm (0.38-in by 0.38-in) joint sealant reservoir. Specimens were cured in a moist room for 14 d and then stored at 23 °C $\pm$ 1.7 °C (73 °F $\pm$ 3 °F) and 50 percent relative humidity for an additional 14 d. Rigid insulation dikes were caulked to the specimen perimeter to allow ponding of water on the joint and surface area.

Specimens were alternately frozen in air and thawed in water with concrete interior temperatures ranging from -17.8 to 4.4 °C (0 to 40 °F). Temperatures were monitored with thermocouples positioned in the block interior 51 mm (2 in) from the joint face. Block specimens were cycled approximately two F-T cycles each d. Spalling was photographically logged to establish a spalling and scaling performance model. Specimens were subjected to a total of 300 F-T cycles.

Freeze-Thaw Variables

Previous durability studies indicate that concrete will fail due to the following factors acting separately or in combination:

- Air-void system parameter deficiencies.
- Insufficient strength (low density or high water-cement ratio).
- Poor curing techniques.
- Application of deicer salts.
- Paste and mortar volume.
- Coarse aggregate F-T susceptibility.

Curing techniques are classified more as a construction rather than a material variable and were not investigated in the laboratory study. Since each agency has input into approving aggregate sources, coarse aggregate F-T susceptibility was also not evaluated. To minimize the number of test specimens, one coarse aggregate type with minimal F-T susceptibility was used in the laboratory study.

Air-void parameters, strength, and consolidation are under control of the contractor and were investigated in the durability laboratory materials study. Mortar volume is also under control of the paving contractor, but because of the relatively small range of mortar variation in slipform paving required to achieve lower slump (for a given aggregate source), it was not investigated. To minimize the number of test specimens, the mortar volume was set at 60 percent. Coarse aggregate volume was then adjusted with changes in target air contents.

A half-fractional factorial, shown in table 22, was developed to evaluate the effects of air-void parameters, strength (in terms of the water-cement ratio), and consolidation on joint spalling. Two levels of consolidation (density), water-cement ratio, and deicer solution concentration were evaluated in the experimental design.

Table 22. Factorial design for laboratory investigation of freeze-thaw durability.

				AIR-VOID PARAMETERS					
				Low		Medium		High	
				Consolidation Level		Consolidation Level		Consolidation Level	
				94%	100%	94%	100%	94%	100%
DEICER SOLUTION	None	Water-Cement Ratio	Low	10		8		12	
			High		3		1		5
	Salt	Water-Cement Ratio	Low		4		2		6
			High	9		7		11	

Note: Only one aggregate type and one paste volume were evaluated.

The consolidation levels (CSL) were set at 94 and 100 percent. The lower consolidated specimens, which may be representative of inadequate consolidation that occurs around doweled joints, were carefully rodded to ensure uniform density throughout the specimen. Water-cement ratios were set at 0.42 and 0.50. Two deicer levels were used: none (water only) and salt (4 g of anhydrous calcium chloride per 100 ml). The deicer concentration is that which is commonly used in tests done in accordance with ASTM C672, "Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals." Mix designs used in the laboratory study are summarized in table 17 of appendix D.

Three air content levels were achieved by varying air-entraining amounts and vibration/rodding effort. Hardened concrete air-void parameters were measured in accordance with ASTM C457, "Standard Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete." The measured air systems for the 100-percent consolidated mixes are summarized in table 18 of appendix D.

Durability Data Analysis

Compressive strengths at 28 d (14 d moist cure and 14 d air cure at 50 percent relative humidity [PRH]) ranged from 25.7 to 48.6 MPa (3730 to 7050 lbf/in^2) for the 100-percent consolidated specimens and 17.8 to 32.7 MPa (2580 to 4740 lbf/in^2) for the 94-percent consolidated specimens. Strength significantly increased as either the water-cement ratio or the air content was decreased. Strength of air-cured cylinders remained relatively constant throughout the test period. Average compressive strength increases from 28 to 208 d were 3 and 16 percent for the 100-percent and 94-percent consolidated specimens, respectively.

The distress observed consisted mainly of surface scaling adjacent to the sawed joints. Scaling is defined as surface deterioration to a depth of approximately 3 to 6 mm (0.12 to 0.25 in) resulting in a loss of surface mortar. Scaling in localized spots developed into joint spalling. Spalling is normally defined as chipping, breaking, or fraying of slab edges adjacent to the joint. Due to lack of traffic loading, the definition was increased to include areas where scaling completely exposed coarse aggregate surfaces and areas where coarse aggregate could be dislodged with moderate force applied by a screwdriver blade. With additional loss of mortar under repeated traffic loads and additional F-T cycles, the observed scaling adjacent to joints in the laboratory study would probably develop into true joint spalling. To quantify joint spalling, spalled surface areas adjacent to the joints were periodically measured and normalized by the joint length. Spalling was measured on both sides of the joint and averaged. Spalling distress was then quantified as a percentage of joint length. Scaling distress was also quantified as mm^2 per mm (in^2 per in) of joint.

The presence of calcium chloride (SALT) significantly decreased the number of F-T cycles to initial scaling and spalling. With the exception of the higher (0.50) water-cement ratio at the nominal (4.5-percent) air content and 94-percent consolidation, none of the specimens thawed in plain water exhibited any scaling or spalling. The block thawed in plain water developed 258 mm^2/mm (0.4 in^2/in) at 98 cycles and did not increase for the remaining 202 F-T cycles. The scaled area did not develop into joint spalling.

All six block specimens subjected to the calcium chloride solution exhibited scaling adjacent to the joint at less than 60 F-T cycles. Scaling was first observed between 55 and 59 cycles for the 100-percent consolidated blocks and between 32 and 35 cycles for the 94-percent consolidated blocks. Scaling was observed along the entire 305-mm (12-in) long joint for all blocks within 65 cycles. Distribution of surface scaling was approximately equal on both sides of the joint. Scaling tended to significantly increase between approximately 150 and 200 cycles.

All six block specimens subjected to the calcium chloride solution exhibited spalling adjacent to the joint at less than 70 F-T cycles. Distribution of spalling was approximately equal on both sides of the joint. With the exception of the fully consolidated specimen with a WC of 0.42 and 5-percent air content, once spalling was initiated it increased rapidly up to approximately 100 F-T cycles, then significantly tapered off from 100 to 300 cycles. Spalling for the fully consolidated specimen with a WC of 0.42 and 6-percent nominal air content did not rapidly increase until after approximately 180 F-T cycles.

For the 100-percent consolidated specimens, the degree of scaling and spalling was significantly reduced with improvements in air-void system parameters. Terminal scaling areas for the fully consolidated specimens were 84, 69, and 30 mm^2/mm (3.3, 2.7, and 1.2 in^2/in) for nominal air contents of 3, 4.5, and 5.5 percent, respectively, after 300 F-T cycles. The reverse trend was true for the 94-percent consolidated specimens, in that the degree of scaling increased with improvements in air-void system parameters. For 94-percent consolidation, terminal scaling areas were 58, 84, and 97 mm^2/mm (2.3, 3.3, and 3.8 in^2/in) for nominal air contents of 3, 4.5, and 5.5 percent, respectively, after 300 F-T cycles. With the exception of the 3-percent air content, the terminal degree of scaling increased with decreases in consolidation and corresponding increases in water-cement ratio.

Terminal spalling percentages for the fully consolidated specimens were 52, 68, and 30 percent for nominal air contents of 3, 4.5, and 5.5 percent, respectively, after 300 F-T cycles. Terminal spalling percentages for the 94-percent consolidated specimens were 84, 72, and 43 percent for nominal air contents of 3, 4.5, and 5.5 percent, respectively, after 300 F-T cycles. The degree of spalling decreased with improvements in air-void system parameters. The terminal degree of spalling increased with decreases in consolidation and corresponding increases in water-cement ratio. Joint scaling and spalling data are given in table 19 of appendix D.

Models relating the number of freeze-thaw cycles to joint spalling were developed. The hardened air content variables selected were the air content and the spacing factor ($\bar{L}$). The spacing factor and air contents used in the analysis are based on <u>fully consolidated</u> concrete. Entrapped air and honeycombing encountered in the less than fully consolidated specimens will significantly alter the air-void system parameters. Plots of deterioration with time indicated a nonlinear increase with cycles. The logarithm of cycles (N) was used as the predictor variable. Since specimens with no calcium chloride exhibited no significant scaled or spalled areas, the interaction effects of SALT*AIR and SALT*log(N) were incorporated into the regression models. The average air-cured compressive strength (28 to 208 d) was used as a covariable for WC. The logarithm of compressive strength was also examined as a transformed independent variable. Other independent variables examined were air content (AIR), consolidation level (CSL), calcium chloride (SALT), water-cement ratio (WC), and void spacing factor ($\bar{L}$).

Similar to other regression analysis, a stepwise regression procedure was used to identify significant main and interaction effect variables. Three different air models were developed in the durability spalling study. The first model did not include the spacing factor variable since not all agencies will utilize hardened air content data in a PRS. This model would be used to set a performance criterion when fresh concrete is sampled behind the slipform paver and tested in the plastic state. The second model developed incorporated both the air content and spacing factor, which can be used if *in situ* concrete air-void systems are evaluated. A third model was developed incorporating only the void spacing factor to evaluate uncommon occurrences where the air content and the void spacing factor are both large.

The first model, giving joint spalling as a function of the air content only, is:

$$\begin{aligned} SPALL = {} & 22.6 + 75.1 * SALT * \log(N) - 78.0 * SALT \\ & - 11.7 * AIR * SALT - 0.00478 * f'_c \end{aligned} \tag{21}$$

where:

SPALL	=	Joint spalling, percent of joint length
SALT	=	0 if no calcium chloride, 1 if calcium chloride
N	=	Number of freeze-thaw cycles
AIR	=	Air content of fully consolidated specimen, percent
f'_c	=	Compressive strength, lbf/in^2

R^2_{adj} = 0.855
SSE = 9.0

The spalling model as a function of only air content is shown in figure 39. As the air content increased from 3 to 5 percent, the percentage of joint that was spalled significantly decreased. Compressive strength, a significant variable, was much less sensitive to scaling/spalling.

The second model, giving joint spalling as a function of both air and spacing factor, is:

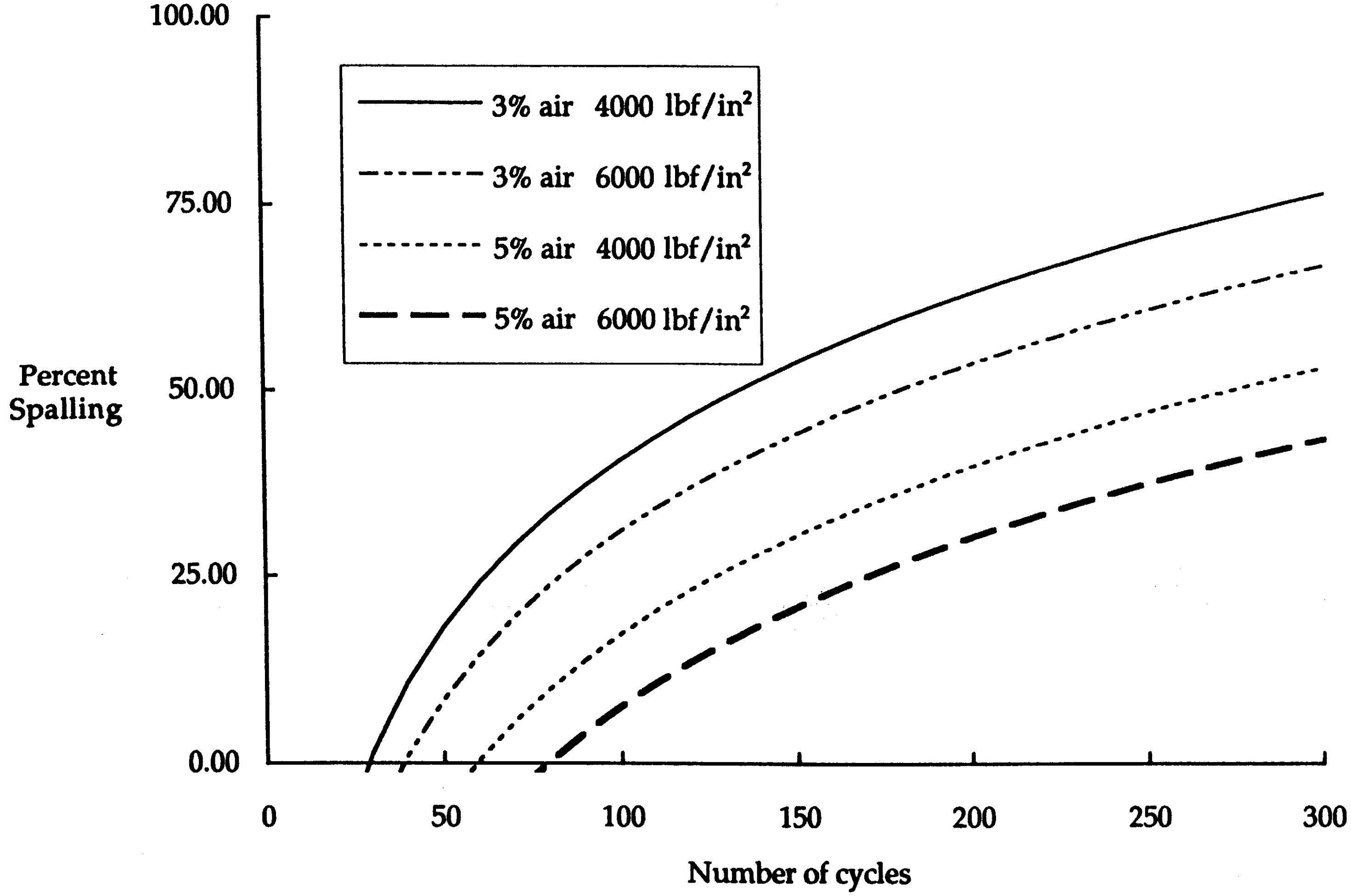
$$\begin{aligned} SPALL = {} & 45.0 + 77.0 * SALT * \log(N) - 29.3 * AIR * SALT \\ & - 0.001 * AIR * f'_c - 1955 * \bar{L} * SALT - 0.439 * \bar{L} * f'_c \end{aligned} \tag{22}$$

where:

SPALL	=	Joint spalling, percent of joint length
SALT	=	0 if no calcium chloride, 1 if calcium chloride
N	=	Number of freeze-thaw cycles
AIR	=	Air content of fully consolidated specimen, percent
f'_c	=	Compressive strength, lbf/in^2
$\bar{L}$	=	Void spacing factor, 1/in

R^2_{adj} = 0.890
SSE = 7.9

The void spacing factor only entered the model as an interaction effect with air, since void spacing factor is correlated with air content. In the laboratory study, the correlation between air content and spacing factors for the six mixes was -0.826. The increase in the coefficient of variation is therefore relatively small when the spacing factor is added to the air content model. Past experience indicates that air content and spacing factor are negatively correlated. As the number of air bubbles increases, the space between them decreases. There may be uncommon instances where the air-void analysis indicates a high air content, but because of the relatively large air bubbles, the spacing factor is larger than the recommended 0.2 mm (0.008 in). Since there was a relatively high degree of correlation between the air content and the spacing factor, there are not significant predicted joint spalling differences between the models presented in equations 21 and 22.



1000 lbf/in² = 6.9 MPa

Figure 39. Predicted percent spalling as a function of the number of freeze-thaw cycles.

The third model, relating joint spalling to the spacing factor, is:

$$SPALL = 14.1 + 74.9 * SALT * \log(N) - 137.1 * SALT + 1727 * \bar{L} * SALT - 0.003 * f'_c \tag{23}$$

where:

SPALL	=	Joint spalling, percent of joint length
SALT	=	0 if no calcium chloride, 1 if calcium chloride
N	=	Number of freeze-thaw cycles
$\bar{L}$	=	Void spacing factor, 1/in
f'_c	=	Compressive strength, lbf/in^2

R^2_{adj} = 0.833
SSE = 9.7

To account for the severity of the laboratory tests where concrete was kept in a saturated state, a percentage factor adjustment is recommended for use in a PRS. For example, if an agency specifies a minimum air content of 5 percent, the expected joint spalling after 300 cycles could be predicted using equation 21. Similarly, other joint spalling percentages could be computed over a range of air contents. The increase or decrease from the baseline 5-percent air content would be used in a PRS to predict an increase or decrease in joint maintenance.

SUMMARY

Summary of Laboratory Testing

Laboratory testing was conducted to investigate key relationships between concrete material quality characteristics and two pavement distress indicators:

- Transverse cracking caused by repeated loading and thermal curling.
- Joint spalling caused by an inadequate air-void system.

The first part of the laboratory materials study investigated factors that affect concrete strength and modulus of elasticity—factors that are under the control of the paving contractor and can significantly influence concrete pavement performance in terms of the development of transverse cracking. Within-batch and between-batch standard deviations were determined for use in the mix design as well as to evaluate the results of the strength/stiffness portion of the laboratory program. The overall flexural strength standard deviation (within- and between-batch standard deviation) ranged from 0.23 to 0.36 MPa (34 to 52 lbf/in^2) between 7 and 28 d. Guidance was provided on the use of overall standard deviations in targeting mix design strengths.

To assist in the mix design process, the various mix design factors were analyzed to evaluate influences on strength and elastic modulus due to changes in coarse aggregate type, cement content, air content, and so on. As expected, flexural strength was most sensitive to changes in water-cement ratio.

Since core strengths will be used to predict *in situ* flexural strengths, simple interstrength relationships were derived for fully consolidated mixes. Although significant relationships could be developed between flexural and splitting tensile strength, between flexural and compressive strength, between splitting tensile and compressive strength, and between elastic modulus and compressive strength, no satisfactory general relationships independent of mix components could be established.

Mix-specific relationships were examined to evaluate errors in predicting one hardened concrete property from another. Interstrength relationships were developed for all 46 mixes used in the strength/stiffness portion of the study. Tests were done at 1, 3, 5, 7, 14, and 28 d for 12 of these mixes. The average flexural strength prediction error (absolute value) from compressive strength was 2.6 percent and that predicted from splitting tensile strength was 2.7 percent. Plots of best fit regression equations indicated that consolidation effects were not completely accounted for within the strength relationships. For example, if consolidation affected flexural strength to the same degree as compressive strength, the regression lines for different consolidation levels extrapolated over the same strength range should coincide. Examination of interstrength relationships at different consolidation levels indicated that significant differences are present. If compressive strength of a less than fully consolidated core was used to predict flexural strength with an interstrength relationship developed from fully consolidated specimens, significant errors can be introduced.

Since it is unlikely that consolidation will be investigated in the mix design process and evaluation of consolidation level effects is somewhat difficult, the 46-mix data base developed in this study was used to evaluate effects of consolidation on interstrength relationships. Several significant mix design parameters and interaction effects were identified to aid engineers in evaluating effects of consolidation on interstrength relationships. Since it is unlikely that the relationships developed in this study at 100-percent consolidation will predict the same flexural strength as that of a mix-specific interstrength equation, a percentage adjustment to the predicted flexural strength was recommended. Details on the use of an adjustment to mix-specific, predicted flexural strength were discussed.

The effects of air content on flexural strength were not completely accounted for in estimating flexural strength from compressive strength. Increases in air content decrease flexural strength more than compressive strength. Adjustments to account for the effects of air content on flexural strength could be made using data developed in the laboratory study. It is recommended that in order to reduce variability in predicting *in situ* flexural strength, mix-specific interstrength relationships at several different air contents should be established during the mix design process. *In situ* core strength and measured air content could then be used to estimate the 28-day, standard-cured flexural strength.

Cores and cylinders cured under identical conditions (same maturity) were tested for compressive strength. Since the cylinders and cores were cured under identical conditions, any differences would be attributable to coring damage and size difference effects. No significant differences were observed between average core strengths and cylinder strengths tested from eight different mixes at 7, 14, and 28 d. Based on analysis of core data, it cannot be statistically inferred that there is a difference between the average strengths obtained from cores and cylinders. The data do not suggest that cores tested in a PRS need to be adjusted for size or coring effects.

There is a significant increase in the within-test variability of cores compared to that of cylinders when both are cured under ideal laboratory curing conditions. Because of this higher variability, it is recommended that more than one core be used to establish sublot strength and that appropriate retesting provisions be made available.

Trends toward earlier opening of concrete pavements emphasize the use of NDT methods to monitor strength development. The pulse velocity and maturity NDT methods are two ways of monitoring *in situ* slab strength and elastic modulus development. These methods can quickly and easily be used to monitor strength development during the initial stages of construction to allow the contractor to make mix design adjustments early in the construction process.

Twelve of the forty-six mixes prepared for the strength and consolidation study were evaluated using pulse velocity and maturity. Since maturity is a function of admixtures used, cement source, and cement type, the evaluation was only a demonstration of how to develop relationships between strength and NDT and how to use those relationships to adjust core strengths to equivalent laboratory-cured strengths. Maturity and pulse velocity were measured at intervals of 1, 3, 5, 7, 14, and 28 d. Strength and elastic modulus mix-specific prediction equations were developed and it was shown that an excellent prediction model could be developed. Using the Nurse-Saul and Arrhenius maturity methods, the average absolute compressive strength prediction error at ages ranging from 1 through 28 d was 3.1 percent.

Steps to adjust core compressive strength (at any maturity) to a standard laboratory-cured compressive strength were outlined. The *in situ* slab maturity and core compressive or splitting tensile strength can be used in mix-specific strength-maturity relationships. Once a standard cured compressive strength is established, mix-specific relationships are used to predict standard cured flexural strength.

Using data generated in the laboratory nondestructive testing study, the 28-day flexural strength was predicted from 7-day cylinder strengths. Cylinder compressive strengths were projected to 28 d using concrete maturity. Flexural strengths at 28 d were then estimated from compressive strength using mix-specific interstrength relationships. Prediction errors averaged 1.8 percent for the eight mixes used in the

NDT study. It must be emphasized that the low prediction errors were determined under controlled, laboratory curing conditions.

Field testing under variable curing conditions is necessary to establish maturity projection and interstrength prediction errors. Based on field-testing data, some minimum maturity for coring could be established to minimize projection errors and to provide contractor feedback as soon as possible.

To address durability problems in the development of PRS, a laboratory testing program was conducted to correlate air-void system parameters with joint spalling. It was assumed that the degree of spalling would directly influence pavement performance. The procedure used to evaluate spalling was adopted from ASTM C666, "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing." Procedure B was followed, where the specimens are frozen in air and thawed in water. The procedure was modified by using block sections with joints to evaluate the progression of joint deterioration. The block specimens were subjected to a total of 300 freeze-thaw cycles.

Consolidation level, water-cement ratio, use of calcium chloride, and air-void parameters were investigated in the development of a joint scaling/spalling model. Without calcium chloride ponded on surfaces during thawing cycles, no significant joint deterioration was observed after 300 cycles. Significant scaling and spalling occurred on the blocks subjected to deicing solutions.

Three different air models were developed in the durability spalling study. The first model did not include the spacing factor variable since not all agencies will utilize hardened air-content data in a PRS. This model would be used to set a performance criterion when fresh concrete is sampled behind the slipform paver and tested in the plastic state. The second model developed incorporated both the air content and spacing factor, which can be used if *in situ* concrete air-void systems are evaluated. A third model was developed incorporating void spacing factor only (no air content variable) to evaluate uncommon occurrences where the air content is relatively high with a large spacing factor.

Summary of Laboratory Results Incorporated into PRS

Several results of the laboratory investigation were incorporated directly into the prototype performance-related specification given in appendix A. That specification currently considers four quality characteristics: flexural strength, slab thickness, air content, and roughness. The laboratory data developed in this investigation were used to address the flexural strength and durability materials factors.

The specification requires 28-d flexural strength for use in many of the predictive models. The concrete flexural strength is determined indirectly via compressive or splitting tensile testing of cores taken from the *in situ* pavement. No satisfactory relationship between flexural and compressive or splitting tensile strength could be developed from the 46 mix data base developed in the laboratory study. Therefore, mix-specific interstrength relationships should be developed to minimize prediction errors when estimating *in situ* flexural strength. Mix-specific interstrength relationships demonstrated that significant relationships predicting flexural strength from compressive or splitting tensile strength could be developed for the given project materials.

The analysis of the core and cylinder compressive strength data indicated that, on average, there is no significant difference between the strength obtained from a 152- by 305-mm (6-in by 12-in) cylinder and that obtained from a 102- by 203-mm (4-in by 8-in) core. Therefore, based on the laboratory data, no provisions are required to convert core compressive strengths to equivalent cylinder compressive strengths. There is, however, a larger variability associated with strengths obtained from cores than strengths obtained from cylinders. To minimize the effects of within-test variability, it is recommended in the specification that a minimum of two cores be tested per sublot for strength. Provisions for retesting were also included to ensure equitable procedures for establishing sublot strength.

Significant equations predicting strength as a function of maturity were developed in the laboratory study. Similar to interstrength relationships, compressive or splitting tensile strength as a function of maturity should be developed using project-specific materials and proposed mix designs. Procedures were presented for using a concrete maturity adjustment to estimate the 28-d flexural strength based on the strength of a core taken at an earlier time. This procedure allows rapid feedback during construction and permits the contractor to make any needed adjustments.

Results from the laboratory durability study were incorporated into the durability performance model. Data were used to modify a joint spalling performance model as a function of air content, the presence of deicer solution, the compressive strength, and number of freeze-thaw cycles. The modified performance model incorporated the performance effects due to air content and strength, both of which are under control of the contractor.

CHAPTER 5. EXPERIMENTAL PLAN FOR THE ACCELERATED TESTING OF MATERIALS AND CONSTRUCTION VARIABLES

INTRODUCTION

It has been asserted in chapter 2 that performance-related specifications must be driven by the development of key pavement distress indicators. That is, the performance of a pavement is measured by the amount of distress and roughness that it develops over time, and the effect of this distress on future rehabilitation and on pavement life is expressed in terms of a life-cycle cost. It is this life-cycle cost that determines the amount of the incentive or disincentive earned by the contractor.

Pavement distress indicators deemed critical to the performance of concrete pavements were identified in chapter 2. For each of these critical distress indicators, key quality characteristics were determined. These quality characteristics are defined as the fundamental materials or construction variables that play a significant role in the development of the associated distress. For example, low concrete flexural strength is a major material factor in the development of load-induced transverse cracking.

Although the key quality characteristics were identified for the primary pavement distress indicators, the actual effect of these quality characteristics on the development of pavement distress and pavement roughness is uncertain. While these effects may be well known for a few of the quality characteristics, they are less certain for many of the others.

In order to better assess the effects of the quality characteristics on the development of pavement distresses, field studies are needed. Ideally, concrete pavement sections of specified design and varying quality characteristics could be constructed and monitored over time so that the resulting distresses and roughness could be recorded and their impact on pavement performance could be determined. However, there are several limitations to that approach. One problem is that certain distresses take a long time to develop, meaning that results will not be immediately available for implementation in a PRS. This is a problem inherent in the study of pavements, where performance is assessed in terms of years, not months. Although the development of distresses could be accelerated through the use of extreme design values (for example, leaving all joints unsawed to determine the effect of inadequate sawing on concrete pavement performance), there invariably and understandably would be a reluctance on the part of most highway agencies to build concrete pavement sections that are destined to fail prematurely in such a catastrophic manner.

The use of an accelerated loading facility was considered for evaluating the performance of concrete pavement sections. An accelerated loading facility could produce up to 1 million load applications per month—the equivalent of 10 or 20 years worth of traffic loading in a year. However, only quality characteristics that are associated with load-induced distresses could be evaluated in this way, and these are characteristics that are fairly well defined. Furthermore, the development of pavement distresses caused by environmental and other factors (such as spalling due to an inadequate air-void system) cannot be accelerated since they require regular and cyclic temperature and moisture variations.

More immediate results could be obtained through the monitoring of selected inservice concrete pavements. In this approach, inservice concrete pavements would be sought out that display distresses that are known to be the result of the particular quality characteristic of interest. For example, jointed concrete pavement sections that are displaying joint spalling or joint deterioration due to dowel misalignment could be identified and evaluated to assess the effect of that quality characteristic on pavement performance. While this approach would require the solicitation and identification of sections displaying distresses that are known to be the result of a particular quality characteristics, this approach does offer the following advantages:

- The cost of constructing new pavements is eliminated.
- The likelihood of obtaining more immediate results is increased.
- The potential for more active participation of highway agencies is increased.

However, there are several drawbacks to the use of inservice pavements, including loss of control over some of the design variables, difficulty in determining the exact construction conditions, and uncertainty over the exact traffic loadings that the pavement has sustained to date.

Nevertheless, it is believed that the use of inservice pavements provides the most practical alternative for quantifying the effect of the quality characteristics on pavement performance and providing more immediate results. Based on that assertion, experimental plans that emphasize the use of inservice pavements have been developed for the evaluation of key distress types. However, alternative plans for specially constructed sections are included for some of the key distress types. Each plan is presented as an independent, stand-alone experiment so that only the most promising ones may be implemented without necessarily implementing the others. Because not every conceivable combination of design, subgrade, traffic, and environmental factors are accounted for, the results of these experiments will need to be extended to other conditions using theory or engineering judgment.

PROPOSED EXPERIMENTS

Three key distress indicators are included in the proposed PRS presented in chapter 3: transverse cracking caused by loading, joint spalling caused by an inadequate air-void system, and initial roughness caused by poor construction practices. However, several distress indicators were identified in chapter 2 that require further investigation so that they may be incorporated into a PRS. These distress indicators are:

1. Transverse cracking due to inadequate or late sawing.
2. Longitudinal cracking due to inadequate or late sawing.
3. Transverse or longitudinal spalling due to early sawing.
4. Transverse joint deterioration due to improper consolidation at joints.
5. Spalling/scaling due to improper air system.
6. Transverse joint spalling due to dowel misalignment.
7. Initial pavement roughness.
8. Longitudinal spalling due to improper tiebar placement.
9. Crack deterioration and punchouts due to improperly placed reinforcing steel.

It was felt that two quality characteristics (air system and roughness) currently included in the PRS require additional investigation, and are therefore also included for the field evaluation.

The proposed individual experiments for the evaluation of these distress indicators—including design factors, variables, and layout—are presented in the next sections. Again, the use of inservice concrete pavements is emphasized, although alternative plans using specially constructed sections are also included for a few of the distress indicators.

Joint Sawing Study (Distress Indicators 1, 2, and 3)

Transverse and longitudinal joints are typically created in a new concrete pavement through diamond-blade sawing. If the transverse and longitudinal joints are sawed too late, or if they are sawed to an inadequate depth, uncontrolled, random cracking can develop as the young slab responds to stresses caused by drying shrinkage, temperature contraction, and thermal curling. If the joints are sawed too soon, spalling or ravelling of the concrete can occur along the joint.

The two primary factors influencing these types of distresses that are under the control of the contractor are the depth of sawing and the timing of sawing. While it is generally accepted that the depth of sawing should be one-fourth to one-third of the slab thickness, the determination of the earliest or latest time for sawing is not so easily established because it is a function of curing conditions and mix design parameters, which influence the rate of strength gain of the newly placed concrete pavement. Recent research has shown that, shortly after placement, surface cooling of about 8 °C (15 °F) from the maximum slab surface temperature resulted in the development of cracking in young concrete.[15] Thus, the slab surface temperature can be monitored and used as an indicator of the latest time for sawing.

The same study on joint sawing indicated that the earliest time for joint sawing could be determined by monitoring the concrete strength.[15] The minimum compressive strength at which sawing should be initiated was a function of the type of aggregate used in the mix:[15]

- Crushed, soft coarse aggregate: 3.7 MPa (530 lbf/in^2).
- Crushed, hard coarse aggregate: 7.0 MPa (1010 lbf/in^2).

- Rounded, hard coarse aggregate: 4.8 MPa (690 lbf/in^2).
- Rounded, soft coarse aggregate: 2.1 MPa (310 lbf/in^2).

Thus, knowing the type of aggregate in the mix, an early joint sawing time can be identified by monitoring concrete strength after construction, using either maturity methods or pulse velocity testing.

An experimental plan using inservice concrete pavements is proposed for this study and is presented below. However, an experimental plan has also been prepared using specially constructed pavements and is provided as an alternative.

Experimental Plan Using Inservice Pavements

Inservice pavements may be used to determine the effect of inadequate sawing on the performance of concrete pavements. However, considerable effort may be needed to identify candidate sections that not only have experienced cracking and spalling due to inadequate joint sawing, but also have detailed documentation on the sawing and curing activities during construction (depth of sawing, timing of sawing, sawing patterns, weather conditions, type of curing, and so on).

Site-Specific Information

It is recommended that three nearby or adjacent sections be selected in each of the four main climatic zones (wet-freeze, dry-freeze, wet-nonfreeze, dry-nonfreeze). One of the three sections in each climatic zone should be free of cracking and spalling, one should exhibit transverse or longitudinal cracking (or both) that is known to be due to inadequate joint sawing, and one should exhibit spalling that is believed to be the result of sawing the joints too early. The distresses should appear over at least a 600-m (2000-ft) segment of the pavement.

While it is preferred that the sections be adjacent to one another on the same roadway, this may not always be possible. It is desirable that the pavement sections be exposed to traffic levels within the range of 250,000 to 500,000 80-kN (18-kip) equivalent single-axle load (ESAL) applications per year.

Pavement Design

It is recommended that the inservice pavements selected for this experiment have certain characteristics, including the following:

- Conventional JPCP designs are recommended because of their widespread use.

- Pavement sections should be relatively new (less than 2 years old) and subjected to moderate traffic volumes.

- Pavements should be constructed on non-swelling and non-frost susceptible soils.

- The pavements should not exhibit any D-cracking or alkali-reactivity distress, nor should they exhibit any significant structural deterioration.
- At least three sections are needed in each of the four climatic zones: one free of distress, one with transverse or longitudinal cracking due to inadequate joint sawing, and one with joint spalling due to early joint sawing. While it is preferable that the sections be adjacent to one another to account for traffic effects, this may not be possible.
- If possible, it is recommended that about half of the sections be constructed over a granular base and the other half be constructed on a stabilized base.
- The distresses in the sections should be located over a minimum length of 600 m (2000 ft).
- Detailed documentation on the construction of the pavement sections must be available.

The experimental design matrix for this study is shown in table 23.

Table 23. Experimental design matrix for joint sawing study of inservice pavements.

		CLIMATIC ZONE			
		Dry-Freeze	Wet-Freeze	Dry-Nonfreeze	Wet-Nonfreeze
TYPE OF DISTRESS IN SECTION	No Distress	Section 1	Section 4	Section 7	Section 10
	Transverse/ Longitudinal Cracking Due to Inadequate Sawing	Section 2	Section 5	Section 8	Section 11
	Joint Spalling Due to Early Sawing	Section 3	Section 6	Section 9	Section 12

Note: If possible, six sections should be constructed on a granular base and six sections should be constructed on a stabilized base.

Variables

As observed from table 23, there are no design or construction variables that are <u>directly</u> incorporated into the experimental plan. Instead, distresses due to inadequate sawing are being targeted in the belief that a range of critical variables (depth of sawing, timing, base type, and so on) will be encountered that will allow a thorough evaluation of the data.

Special Data Needs

The availability of complete construction records and documentation is essential to this study. Information on the timing of sawing, the depth of sawing, curing conditions, and so on, must be available in order to perform a valid analysis. Roughness information would also be helpful to assist in documenting the effect of distress on the roughness of the pavement.

Layout

For the inservice joint sawing study, it is recommended that a minimum length of 600 m (2000 ft) is needed for each section. It is preferable that the sections within each climatic zone be adjacent to one another on the same roadway, although this may not always be possible.

Length of Test Period

It is expected that results from this study should be available immediately upon evaluation of the sections. However, it may be desirable to continue monitoring the development of slab cracking, joint spalling, and the ensuing pavement roughness of the sections for an additional 2 to 5 years so that ultimate levels of distress can be determined.

Products for PRS

It is believed that this study will lead to the development of several models for incorporation into a PRS. One model will predict the development of transverse and/or longitudinal cracking based on critical construction factors, such as joint sawing depth and timing of joint sawing. Another model can be developed that will predict the development of joint spalling due to early joint sawing operations.

<u>Alternative Experimental Plan Using Specially Constructed Pavement Sections</u>

As an alternative to the use of inservice pavements in the evaluation of joint sawing, the following experimental plan describes pavement sections that could be specially constructed to evaluate the effect of inadequate joint sawing. Although expensive to construct, more control over the variables is obtained.

Site-Specific Information

While this experiment could be conducted in any environment, it is suggested that it be conducted as a "demonstration" in the wet-freeze environment. The experiment will be conducted on only one subgrade type and for only one traffic level. It is recommended that it be subjected to a moderate level of traffic (between 250,000 and 500,000 80-kN [18-kip] ESAL applications per year).

Pavement Design

It is recommended that the concrete pavement constructed for this experiment have the following design characteristics:

- Conventional jointed plain concrete pavement.

- Slab thickness designed for expected traffic level.

- Short (say, 5-m [16.4-ft]) joint spacing, at regular intervals.

- A conventional concrete mix design that achieves adequate short- and long-term strength should be used. Only high-quality aggregates not susceptible to D-cracking or alkali-reactivity should be used.

- Dowel bars as dictated by the expected traffic level. Dowel bars should be 32 mm (1.25 in) in diameter, 457-mm (18-in) long, and placed at mid-depth.

Variables

As previously stated, the two variables of interest in this experiment are the depth of sawing and the timing of sawing. Although the optimum timing of joint sawing is a function of the mix design and curing conditions, it is believed that both early and late sawing times can be identified by monitoring the concrete strength and temperature data as previously described. In this way, sawcutting can be performed "early" (to evaluate joint spalling), within specifications, and "late" (to evaluate uncontrolled slab cracking).

In addition to the factors described above, it is known that base type can have a significant effect on the development of transverse cracks. Greater friction between the slab and base is produced by stiffer base courses (such as cement- or asphalt-treated bases), which can lead to the development of transverse and longitudinal cracks if the joints have not been sawed deeply enough or in a timely fashion.

Although evaluating the same variables, separate experiments are proposed for transverse joint sawing and longitudinal joint sawing to prevent the confounding effects of random transverse and longitudinal cracking. The experimental design matrix for the transverse joint sawing study is shown in table 24, while the experimental design matrix for the longitudinal joint sawing study is shown in table 25. The variables, the recommended number of levels, and the way that the different levels can be obtained for both experiments are described in the sections that follow.

Table 24. Experimental design matrix for transverse joint sawing study using specially constructed sections.

		Depth of Transverse Joint Sawing (percent of slab thickness)		
		0	17	33
Stabilized Base	No Sawing	Section 1	N/A	N/A
	"Early" Joint Sawing	N/A	Section 3	Section 9
	"Optimal" Joint Sawing	N/A	Section 4	Section 10
	"Late" Joint Sawing	N/A	Section 5	Section 11
Granular Base	No Sawing	Section 2	N/A	N/A
	"Early" Joint Sawing	N/A	Section 6	Section 12
	"Optimal" Joint Sawing	N/A	Section 7	Section 13
	"Late" Joint Sawing	N/A	Section 8	Section 14

Table 25. Experimental design matrix for longitudinal joint sawing study using specially constructed sections.

		Depth of Longitudinal Joint Sawing (percent of slab thickness)		
		0	17	33
Stabilized Base	No Sawing	Section 1	N/A	N/A
	"Early" Joint Sawing	N/A	Section 3	Section 9
	"Optimal" Joint Sawing	N/A	Section 4	Section 10
	"Late" Joint Sawing	N/A	Section 5	Section 11
Granular Base	No Sawing	Section 2	N/A	N/A
	"Early" Joint Sawing	N/A	Section 6	Section 12
	"Optimal" Joint Sawing	N/A	Section 7	Section 13
	"Late" Joint Sawing	N/A	Section 8	Section 14

Depth of Sawing

Three levels of the depth of sawing (expressed as a percentage of the slab thickness) are proposed for both longitudinal and transverse joint sawing experiments: 0, 17, and 33. Sawing at a depth of one-third the slab thickness, which is in accordance with many specifications, should create a sufficiently deep weakened plane to ensure the formation of the crack beneath the sawcut. One-sixth of the slab depth is suggested as a good mid-value between 0 and one-third. The final level, zero percent, or not sawing joints, will serve as an accelerated evaluation of sorts, since the effect of random, uncontrolled cracking (both transverse and longitudinal) will be readily apparent. Under these conditions, transverse cracks will occur at large intervals shortly after construction. It is expected that longitudinal cracking over the length of the section will also occur shortly after construction. The effect of the uncontrolled cracking on pavement performance can be assessed in terms of the amount of cracking that develops and its impact on pavement roughness.

Timing of Sawing

The timing of the sawing operations of both the longitudinal and transverse joint sawing experiments will be investigated at four levels: none, early, optimum, and late. The timing for these operations can be identified through concrete strength and temperature monitoring. The no sawing option, as described above, will cause the development of a great deal of random cracking and should prove useful in evaluating the effect of the cracking on pavement performance. The early joint sawing will create joint ravelling or spalling, whose effect on concrete pavement performance can then be investigated. The optimum and late joint sawing times, taken together, should prove useful in assessing how the timing of joint sawing operations impacts the development of cracking.

Base Type

Two base types are recommended: a stabilized base and a granular base. This will allow the effect of base friction to be considered in the evaluation. The design of these base types will be in accordance with the sponsoring agency's specifications.

Special Construction Needs

During the construction of this experimental project, it is important that sufficient joint sawing equipment be available so that all of the joints, both longitudinal and transverse, can be sawed at the prescribed times. It is also important during construction to try and minimize large differences in curing and environmental conditions.

Layout

Tables 24 and 25 indicate that a total of 28 sections are needed for the investigation of joint sawing (both longitudinal and transverse). Assuming a recommended minimum length of 300 m (1000 ft), a total project length of 8500 m (28,000 ft) is required.

Length of Test Period

It is believed that this experiment can yield useful information within a minimum of 2 years, because most of the cracking should develop within the first year (or even the first 6 months). However, it is desirable to provide sufficient time for all of the cracking to initiate. If the effects of random cracking on roughness are sought, a minimum period of 5 to 10 years is probably needed so that the cracks can be allowed to progress and deteriorate.

Products for PRS

Two models, one for transverse cracking and one for longitudinal cracking, can be developed from this study that will predict the amount of cracking based on the depth of the sawcut and the timing of the sawing. While these models would be limited to the materials, environment, and curing conditions of the experiment, it is believed that they would provide considerable insight on the topic of joint sawing. It may be possible to extend these results to other conditions, since sawcut depth and timing are objective measurements that could be used in a PRS.

A third model will be developed that predicts joint spalling as a function of early sawing. Again, this model would be limited to the conditions inherent in the experimental study.

Air System/Consolidation Study (Distress Indicators 4 and 5)

The provision of an inadequate air-void system in a concrete pavement can create severe spalling, scaling, and disintegration of both transverse and longitudinal joints. This is particularly true for concrete pavements located in a freeze-thaw environment and subjected to deicing materials. For example, a recent evaluation of a section of the I-88 Tollway in Illinois revealed that the cause for the severe transverse and longitudinal joint deterioration in the 16-year-old, 356-mm (14-in) slab was an inadequate air-void system.[16] Although such occurrences are uncommon, the consequences are quite severe when it does occur.

Inadequate or insufficient consolidation is another phenomenon that, although uncommon, can cause severe deterioration, particularly at doweled transverse joints. Less than fully consolidated concrete has a lower strength than fully consolidated concrete, and is more susceptible to damage from both traffic and environmental effects. While these topics have been investigated in the laboratory studies described in chapter 4, a field study is proposed to assess the effect of inadequate air and insufficient consolidation on actual concrete pavement performance.

Experimental Plan Using Inservice Pavements

An adequate number of inservice concrete pavements are believed to exist that are exhibiting distress due to an inadequate air-void system or to inadequate consolidation. However, some effort may be needed to identify such sections, including a limited amount of initial testing to verify that the distresses exhibited in the candidate sections are indeed due to the quality characteristics of interest in this study. Once such sections have been identified, the degree of consolidation and the air-void content can be determined and correlated with the joint spalling.

Site-Specific Information

This study can be adequately performed by considering inservice pavements in each of the four main climatic zones. This will allow the effects of freeze-thaw and the presence of excess moisture to be considered. The effects of deicing chemicals can also be evaluated in the freeze climates.

In order for the objectives of the study to be achieved, two types of pavement sections must be identified in each climatic zone. First, sections that are exhibiting spalling or joint deterioration due to an inadequate air-void system must be identified. Second, separate sections that are exhibiting joint spalling or deterioration that are due to inadequate consolidation of the joints must be located. It is desirable for the pavement sections to be exposed to traffic levels within the range of 250,000 to 500,000 80-kN (18-kip) ESAL applications per year.

Pavement Design

It is recommended that the inservice pavements selected for this experiment have certain characteristics, including the following:

- Conventional JPCP designs are recommended because of their widespread use.
- Pavement sections should be at least 15 years old and subjected to moderate traffic volumes.
- Sections should be constructed on non-swelling and non-frost susceptible soils.
- The pavement sections should not exhibit any D-cracking or alkali-reactivity distress, nor should they exhibit any significant structural deterioration.
- Within each climatic zone, one section with an inadequate air-void system and one section with inadequate consolidation must be identified. For each study, sections should possess similar design features (thickness, base type, joint spacing, and so on).
- The consolidation study should employ doweled concrete pavements, where joint consolidation problems are most common.

- A minimum length of 600 m (2000 ft) is needed for each of the sections. For the air-void study, at least 20 joints in each section should be exhibiting distress due to an inadequate air-void system; for the consolidation study, at least 20 joints should be exhibiting distress due to inadequate consolidation. Tests should be done on all joints within the section to determine the values of air and consolidation so that correlations between those values and joint spalling can be established.

The experimental design matrix for this study is shown in table 26.

Table 26. Experimental design matrix for air-void/consolidation study of inservice pavements.

		CLIMATIC ZONE			
		Dry-Freeze	Wet-Freeze	Dry-Nonfreeze	Wet-Nonfreeze
TYPE OF DISTRESS	Spalling Due to Inadequate Air Content	Section 1	Section 3	Section 5	Section 7
	Spalling Due to Inadequate Consolidation	Section 2	Section 4	Section 6	Section 8

Variables

The variables considered are the air content for the air-void study, and the degree of consolidation for the consolidation study. These variables must be identified prior to the inclusion of inservice pavements through destructive coring and linear traverse. To facilitate the analysis, it is desirable that the various sections within each study possess similar design features (thickness, base type, joint spacing, and so on).

Special Data Needs

Verification of an inadequate air-void system or of inadequate consolidation must be made prior to the inclusion of a candidate section into the study. This can be accomplished through destructive coring and linear traverse. Some original design and construction information—such as target air contents and densities—will be needed to assist in the verification and selection process. Roughness information over the life of each of the sections would also be helpful to assist in documenting the effect of distress on the roughness of the pavement.

Once suitable sections are identified, tests should be conducted on both the distressed and nondistressed joints within the section. This will provide a wider range of data for use in developing correlations between the quality characteristic and pavement distress.

Layout

It is recommended that each section included in the study have a minimum length of 600 m (2000 ft). For the air-void system study, at least 20 joints in each section should be exhibiting distress due to an inadequate air-void system. For the consolidation study, at least 20 joints in each section should be exhibiting distress due to inadequate consolidation.

Length of Test Period

It is expected that results from this study would be available immediately upon evaluation of the sections. However, it may be desirable to continue monitoring the joint spalling and the ensuing pavement roughness of the sections for an additional 5 years so that ultimate levels of distress can be determined and so that the effect of the distresses on long-term roughness can be identified.

Products for PRS

This study should provide two products for use in a PRS. The first is a model that can be used to predict the development of joint spalling as a function of the air content (or other air-void system parameters), certain climatic indicators (available moisture and number of freeze-thaw cycles), and the application of deicing chemicals. The second product is a model that will predict the development of joint spalling or deterioration as a function of the degree of consolidation at the joints.

<u>Alternative Experimental Plan Using Specially Constructed Pavement Sections</u>

As an alternative to the use of inservice pavements in the evaluation of concrete air-void systems and the degree of consolidation, the experimental plan given below is provided. This experimental plan describes pavement sections that could be specially constructed to evaluate the effects of the air-void system and consolidation on concrete pavement performance. However, as noted earlier, such an undertaking is expensive to construct and many of the sections are designed for early failure.

Because deterioration caused by an inadequate air-void system may take several years to manifest itself, several low levels of air content are proposed so that the deterioration may be "accelerated" to a certain degree. Two levels of consolidation are proposed, obtained by activating and deactivating the mechanical vibrators.

Site-Specific Information

It is imperative that this experiment be located in a freeze-thaw environment so that the concrete with inadequate air entrainment will be exposed to freeze-thaw conditions. The experiment will be conducted on only one subgrade type and for only one traffic level. Again, it is suggested that the pavement be subjected to between 250,000 and 500,000 80-kN (18-kip) ESAL applications per year.

Pavement Design

It is recommended that the concrete pavement constructed for this experiment have the following design characteristics:

- Conventional jointed plain concrete pavement.

- Slab thickness designed for expected traffic level.

- Short (say, 5-m [16.4-ft]) joint spacing, at regular intervals.

- For all cases, the concrete mix should be designed to achieve adequate strength. Only high-quality aggregates not susceptible to D-cracking or alkali-reactivity should be used.

- One base type (in accordance with sponsoring agency's specifications).

- Dowel bars of sufficient diameter for the slab thickness and projected traffic. Dowel bars should be 32 mm (1.25 in) in diameter, 457-mm (18-in) long, and placed at mid-depth. Alignment of the dowel bars is critical to eliminate any confounding effects.

- Joints sealed in accordance with the agency's standard practice.

- Tied concrete shoulders, to allow an evaluation of the effects of lower traffic on joint spalling (since the shoulders will be subjected to a lower traffic level than the mainline pavement).

Variables

A total of four design variables are suggested for the evaluation of joint spalling due to inadequate air/consolidation. The experimental design matrix for this study is shown in table 27, while the variables, the recommended number of levels, and the way that the different levels can be obtained are described in the sections that follow.

Air Content

Different air contents can be obtained by varying the amount of air-entraining agent. Four levels of air content are recommended: none, low, medium, and high.

Table 27. Experimental design matrix for air-void/consolidation study using specially constructed sections.

MAINLINE PAVEMENT STUDY

		High Strength	Low Strength
Full Vibration	No Air	Section 1	Section 9
	Low Air	Section 2	Section 10
	Medium Air	Section 3	Section 11
	High Air	Section 4	Section 12
No Vibration	No Air	Section 5	Section 13
	Low Air	Section 6	Section 14
	Medium Air	Section 7	Section 15
	High Air	Section 8	Section 16

SUPPLEMENTARY ROADSIDE STUDY

		High Strength	Low Strength
Full Vibration	No Air	Section 1	
	Low Air		
	Medium Air	Section 2	
	High Air		
No Vibration	No Air		
	Low Air		
	Medium Air		
	High Air		

Strength

It is proposed that the effect of relative strength on the development of joint spalling be investigated. Different strengths can be achieved by altering the cement factor. Two levels of strength (cement factor) are suggested (low and high).

Consolidation

Consolidation can have a significant effect on concrete strength. It is proposed for inclusion as a variable in this study to determine its effect on spalling. Two levels of consolidation are proposed: conventional paving with vibration, and conventional paving with the vibrators turned off. It is expected that the latter procedure will produce about a 90-percent level of consolidation.

Other Factors

It is proposed that the outer tied shoulder be monitored to determine the effect (if any) of traffic loading on the development of joint spalling, since the shoulder will be subjected to fewer traffic applications. Furthermore, because the application of salt cannot be controlled as a variable, it is suggested that a short concrete section (say, 100-m [300-ft] long) be constructed off the roadway. This section will not carry traffic and will not be subjected to deicing chemicals, and therefore should serve as a control. Only the amount of air will be varied, since information on the effects of consolidation and strength will be provided from the mainline study.

Special Construction Needs

This experiment requires the construction of a tied concrete shoulder and an offroad pavement section. The tied concrete shoulder should be paved integral with the mainline pavement. The vibrators of the paver must be turned off in certain locations to produce less than fully consolidated concrete. The offroad pavement section should have joints cut at the same interval and at the same time as the mainline pavement sections. The joints of both the mainline and the offroad sections should be sealed in accordance with the agency's standard practice.

Layout

A full factorial design is proposed for this study. According to the factors selected for this study, a total of 16 sections will be needed for the investigation. If the sections are 150 m (500 ft) in length, then the entire project length needed is about 2450 m (8000 ft). In addition, a short, 100-m (300-ft) section will be constructed on the roadside to isolate the effects of traffic and deicing chemicals.

Length of Test Period

It is recommended that this experiment be left inservice for a minimum of 10 years (approximately 5 million ESAL applications). This should be sufficient time for many of the sections to exhibit spalling due to an inadequate air-void system. In fact, the sections containing no air-void system may begin exhibiting deterioration within a few years, depending upon the harshness of the climate and the application of deicing chemicals. It is expected that a period of up to 20 years is needed to fully evaluate the effects of the factors in the experiment.

Products for PRS

Based on the conduct of this experiment, a model will be developed that predicts joint spalling or deterioration as a function of air, concrete strength, consolidation, and number of freeze-thaw cycles. Because of the tied concrete shoulder and the supplementary roadside study, the effects of traffic and deicing chemicals on joint spalling will also be considered. The quality characteristics that are to be measured are the amount of entrained air, the strength, and the consolidation (unit weight).

Dowel Misalignment Study (Distress Indicator 6)

The potential for misaligned dowel bars to adversely affect the performance of concrete pavement has long been recognized, and, in fact, there have been several studies on how misaligned dowel bars affect concrete pavement performance.[(17,18,19)] However, these have generally been analytical treatises or small-scale laboratory studies and not actual field investigations.

There are several problems in evaluating the effects of dowel misalignment on concrete pavement performance. One problem is measuring the actual amount of dowel misalignment in the slab, although this can be overcome by using ground penetrating radar (GPR).

Another problem is that dowel bars may be misaligned in many ways, such as:[(19)]

1. Horizontal translation, where a level dowel bar is not located at its proper location along the length of the joint.
2. Longitudinal translation, where a level dowel bar is not centered over the joint.
3. Vertical translation, where a level dowel bar is not located at the slab mid-depth.
4. Horizontal skew, where a level dowel bar is skewed to either the right or the left.
5. Vertical skew, where a dowel bar is not level but skewed pointing up.

It is also possible that a dowel bar could exhibit a combination of these types of misalignment. However, it is believed that misalignments 4 and 5 are most important to the study under consideration. In addition, there are typically 12 dowel bars at a joint, any number of which could be misaligned. The more misaligned dowel bars, the greater the restraining stresses as the joint undergoes movement.

<u>Experimental Plan Using Inservice Pavements</u>

Inservice pavements may be used to evaluate the effect of dowel misalignment on concrete pavement performance. These inservice pavements must be more than 10 years old and must be exhibiting signs of spalling or joint deterioration that is suspected of being the result of misaligned dowels. Some effort will be needed to identify suitable sections, including a limited amount of initial testing to verify that the pavement joint distresses are caused by misaligned dowels.

Site-Specific Information

This study can be adequately performed by considering inservice pavements in each of the four main climatic zones. This will allow the effects of seasonal temperature and moisture effects to be considered. It is essential that sections be identified that are exhibiting spalling or joint deterioration due to dowel misalignment. It is desirable that the pavement sections are exposed to the annual traffic levels in the range of 250,000 to 500,000 80-kN (18-kip) ESAL applications.

Pavement Design

It is recommended that the inservice pavements selected for this experiment have certain characteristics, including the following:

- Both doweled JPCP and JRCP designs should be included to evaluate the effect of joint spacing on the development of spalling.
- Pavement sections should be at least 10 years old and subjected to moderate traffic volumes.
- Sections should be constructed on non-swelling and non-frost susceptible soils.
- The pavement sections should not exhibit any D-cracking or alkali-reactivity distress, nor should they exhibit any significant structural deterioration.
- Where possible, it is recommended that about half of the sections be constructed over a granular base and the other half be constructed on a stabilized base.
- Within each climatic zone and for each pavement type (JPCP and JRCP), one section exhibiting joint distresses due to dowel misalignment must be identified.
- A minimum length of 600 m (2000 ft) is needed for each of the sections, with at least 20 joints in each section exhibiting distress due to misaligned dowel bars. Tests should be done on all joints within the section to determine the type and amount of misalignment so that correlations between those values and joint spalling can be established.

The experimental design matrix for this study is shown in table 28.

Table 28. Experimental design matrix for dowel misalignment study of inservice pavements.

		CLIMATIC ZONE			
		Dry-Freeze	**Wet-Freeze**	**Dry-Nonfreeze**	**Wet-Nonfreeze**
JPCP (Short-Jointed)	**Deficient Dowel Alignment**	Section 1	Section 3	Section 5	Section 7
JRCP (Long-Jointed)	**Deficient Dowel Alignment**	Section 2	Section 4	Section 6	Section 8

Note: If possible, four sections should be constructed on a granular base and four sections should be constructed on a stabilized base.

Variables

The presence of distress due to dowel bar misalignment, joint spacing, and base type are the variables that are considered for this study. Once a pavement section is identified as exhibiting distress due to dowel bar misalignment, measurements of the location of the dowel bars must be conducted through the use of GPR. This will identify the amount of misalignment, which can then be used in the development of models for the prediction of joint deterioration.

Special Data Needs

A critical aspect of this proposed field study is the verification that the joint distresses of the candidate sections are the result of dowel bar misalignment. This can be quickly determined through the use of GPR, although some discretionary coring may also be required for verification. A minimum of 20 joints suffering from dowel misalignment is needed within each section in order for a section to be considered for inclusion. In addition, roughness information over the life of each of the sections would also be helpful to assist in documenting the effect of distress on the roughness of the pavement.

Once suitable sections are identified, tests should be conducted on both the distressed and nondistressed joints within the section. This will provide a wider range of data for use in developing correlations between dowel misalignment and joint spalling.

Layout

It is recommended that a minimum length of 600 m (2000 ft) is needed for each section. At least 20 joints in each section should exhibit distress due to dowel misalignment.

Length of Test Period

It is expected that results from this study would be available immediately upon evaluation of the sections. The type and amount of dowel misalignment will be measured and related to the development of joint spalling. However, it may be desirable to continue monitoring the joint spalling and the ensuing pavement roughness of the sections for an additional 5 years so that the ultimate levels of distress can be determined and so that the effect of the distresses on long-term roughness can be identified.

Products for PRS

This study should lead to the development of a model that predicts the development of joint spalling as a function of the type and amount of dowel misalignment, pavement design factors (base type, slab thickness, joint spacing), and climatic factors (annual maximum and minimum temperatures). In addition, by evaluating all joints within the section, it is believed that tolerances on the acceptable amount of dowel misalignment can be determined.

<u>Alternative Experimental Plan Using Specially Constructed Pavement Sections</u>

As an alternative to the use of inservice pavements for the evaluation of dowel misalignment on concrete pavement performance, the experimental plan given below is provided. This experimental plan describes pavement sections that could be specially constructed to evaluate the effects of dowel misalignment on concrete pavement performance. Again, while more control over the variables is obtained, such an undertaking is expensive to construct and many of the sections will fail early.

From a practical perspective, the major factors to be investigated in a study of dowel misalignment are the type of misalignment, the amount of misalignment, and the amount of movement to which the joint will be subjected. To produce a useful experiment of reasonable scope and size, the following assumptions are made:

- For the purposes of this study, it is believed that misalignment due to a vertical skew is a more critical problem (cages crushed downward during concrete deposition), so only that type of misalignment will be investigated.

- Four levels of misalignment will be investigated: 0, 13 mm (0.5 in), 25 mm (1 in), and 51 mm (2 in) per 457 mm (18 in).

- The number of dowel bars misaligned at a joint will be investigated at three levels: none, four, and eight. This is believed to cover the severe case when many dowel bars are misaligned, as well as the more common case where only a few dowel bars are misaligned.

- Different amounts of joint movement will be obtained by using two different base types (granular and stabilized) and joint spacings (JPCP and JRCP).

Site-Specific Information

This experiment could be conducted in any environment or location where dowel bars are commonly used. However, it is suggested that this experiment be conducted in a wet-freeze environment, where large temperature differences (which cause horizontal slab movements) and the application of deicing chemicals will be encountered. The experiment will be conducted on only one subgrade type and for only one traffic level. It is recommended that it be subjected to a moderate level of traffic (say, between 250,000 and 500,000 80-kN [18-kip] ESAL applications per year).

Pavement Design

It is recommended that the concrete pavement constructed for this experiment have the following design characteristics:

- Conventional jointed plain concrete pavement with short (say, 5-m [16-ft]) joint spacing at regular intervals.

- Conventional jointed reinforced concrete pavement with joint spacings less than 12 m (40 ft).

- Slab thickness designed for expected traffic level.

- A conventional concrete mix design that achieves adequate short- and long-term strength should be used. Only high-quality aggregates not susceptible to D-cracking or alkali-reactivity should be used.

- Dowel bars of sufficient diameter for the slab thickness and projected traffic. Dowel bars should be 32 mm (1.25 in) in diameter, 457-mm (18-in) long, and placed at mid-depth.

Variables

The experimental design matrix for this study is shown in table 29. The variables, the recommended number of levels, and the way that the different levels can be obtained, are discussed in the sections that follow.

Table 29. Experimental design matrix for dowel misalignment study using specially constructed sections.

			Amount of Horizontal Dowel Misalignment: 0	13 mm (0.5 in)	25 mm (1 in)	51 mm (2 in)
JPCP	Granular Base	No misalignment	Section 1	N/A	N/A	N/A
		4 bars misaligned	N/A	Section 5	Section 13	Section 21
		8 bars misaligned	N/A	Section 6	Section 14	Section 22
	Stabilized Base	No misalignment	Section 2	N/A	N/A	N/A
		4 bars misaligned	N/A	Section 7	Section 15	Section 23
		8 bars misaligned	N/A	Section 8	Section 16	Section 24
JRCP	Granular Base	No misalignment	Section 3	N/A	N/A	N/A
		4 bars misaligned	N/A	Section 9	Section 17	Section 25
		8 bars misaligned	N/A	Section 10	Section 18	Section 26
	Stabilized Base	No misalignment	Section 4	N/A	N/A	N/A
		4 bars misaligned	N/A	Section 11	Section 19	Section 27
		8 bars misaligned	N/A	Section 12	Section 20	Section 28

Number of Dowel Bars Misaligned

The number of dowel bars misaligned at a joint will affect the amount of resistance (stress) developed at the joint. Three levels will be investigated: no dowel bars misaligned, four dowel bars misaligned, and eight dowel bars misaligned. This setup covers the extreme case of multiple misaligned dowel bars, as well as the more common cases of no dowel misalignment or only a few dowel bars misaligned.

It is recommended that for the four-dowel misalignment case, two adjacent dowel bars be misaligned in each wheelpath to evaluate the effects of dowel socketing under loading. The dowel bars should be tilted in opposite directions (one tilted upwards and the adjacent one tilted downwards) to maximize joint lockup. For the eight-dowel misalignment case, it is suggested that four dowel bars in each wheelpath be misaligned. Again, the adjacent dowel bars should be misaligned in opposite directions.

Amount of Dowel Misalignment

As observed from table 29, the amount of dowel misalignment has been set to four levels: 0, 13 mm (0.5 in), 25 mm (1 in), and 51 mm (2 in) per 457 mm (18 in). It is believed that this covers the range of misalignments that would most commonly be encountered in actual construction operations. To ensure that the desired misalignments are obtained, it is recommended that dowel baskets reinforced with steel ties be used and rigidly secured to the base. Close monitoring during the paving operations is needed to ensure that the baskets are not moved. The actual amount of misalignment will be measured after construction using GPR.

Base Type

Both a granular and a stabilized base are proposed for inclusion in the study. Since granular bases allow more movement than stabilized bases, this will create different amounts of joint openings that will be resisted by the misaligned dowel bars. Larger openings are expected to create greater stresses in the slab, which can result in the development of spalling or cracking.

Joint Spacing

Joint spacing is suggested as a variable for evaluation in the experiment. Longer joint spacings (i.e., longer slab lengths) create larger joint openings and movements. The amount of these joint openings will determine the amount of stress that is developed in the slab. Joint spacing will be considered as a variable through the construction of a JPCP (with slab lengths generally less than 5 m [16 ft]) and a JRCP (with slab lengths generally less than 12 m [40 ft]).

Special Construction Needs

To ensure the precise misalignments prescribed in this experiment, dowel baskets reinforced with steel ties should be used. The exact number of dowel bars must be misaligned to the prescribed amount prior to paving, and the dowel baskets must be rigidly affixed to the base. During paving, the baskets must be closely monitored to ensure that they are not moved.

So that sufficient slab movements will be developed, it is recommended that only dowel bars at alternate joints be misaligned. Furthermore, it is believed that at least a 75-m (250-ft) transition area between design sections must be provided to eliminate adjacent effects.

Transverse joint sawing must be performed to the specified depth and in a timely manner. This is to ensure the formation of the joints at the dowel location and to eliminate any mid-panel cracking that could reduce the amount of slab movements. Sawcuts should be made exactly over the center of the dowels.

To maximize the development of distress, it is suggested that no tiebars be used between lanes. This will also eliminate or reduce lane width effects.

Layout

Table 29 indicates that a total of 28 sections are needed for the investigation of dowel misalignment. Assuming a minimum of 150 m (500 ft) is needed for each section, then a total project length of 4250 m (14,000 ft) is needed. If a 75-m (250-ft) transition area is placed between design sections, then the total project length becomes 6300 m (20,750 ft).

Length of Test Period

In order to obtain as much useful information as possible from this experiment, it is recommended that it be left in service and monitored for at least 10 years. This will allow sufficient time for joint spalling and cracking from dowel misalignment to develop. However, it is expected that some useful data will be available on a few of the sections in as early as 2 or 3 years.

Products for PRS

It is expected that a performance prediction model will be developed that will predict the amount of joint spalling or cracking as a function of dowel misalignment, the number of dowel bars misaligned, and the amount of joint movement (opening/closing). The former two variables could easily be measured on an actual construction project using random sampling procedures and GPR.

Pavement Roughness Study (Distress Indicator 7)

It is a commonly held belief that new pavements constructed with a rougher profile will deteriorate more rapidly than smoother pavements. This concept has perhaps been most widely encouraged by the AASHTO pavement design models, which predict that a pavement with a higher initial serviceability rating will last longer than an otherwise equivalent, but rougher, pavement.[5] While the reason for rougher pavements deteriorating faster than smoother ones is often attributed to greater dynamic loading effects of truck traffic on rougher pavements, this theory has never really been confirmed or verified in an actual field investigation.

Many current concrete pavement specifications offer smoothness incentives to contractors if they achieve a smoothness at or below a specified value. The specified value varies somewhat from agency to agency, but generally a maximum value of 0.11 m/km (7 in/mi), as measured by the California profilograph, is specified for the contractor to receive full pay. However, this initial smoothness value is apparently subjective in that it accounts for the short-term benefits of smoothness to the user, but does not account for any potential long-term benefits (increased service life and postponement of rehabilitation) of a smoother pavement.

Thus, it appears that an experiment on initial pavement roughness should achieve two purposes. The first is to determine if initially smooth pavements do indeed last longer than initially rough pavements, and the second is to identify critical levels of initial smoothness that can ensure the long-term benefits of initially smooth pavements. It is believed that inservice pavements can be used for this study.

<u>Site-Specific Information</u>

This experiment recommends the use of inservice roadways for which profile records are available for every year since construction. It is recommended that two nearby sections—one relatively smooth and the other relatively rough—be selected in each of the four main climatic zones. Ideally, the sections will be adjacent to one another and therefore would have been exposed to the same traffic levels (preferably within the range of 250,000 to 500,000 80-kN [18-kip] ESAL applications per year). It is further recommended that pavements constructed on subgrades susceptible to frost heave or soil swelling be avoided, so that the actual, long-term roughness effects can be identified without any confounding caused by severe soil movements.

<u>Pavement Design</u>

It is recommended that the inservice pavements selected for this experiment have certain characteristics, including the following:

- Conventional JPCP designs are recommended because of their widespread use.

- Pavement sections should be at least 15 years old and be subjected to moderate traffic volumes.

- Sections should be constructed on non-swelling and non-frost susceptible soils.

- Each pavement section should contain an adequate structural design and should contain dowel bars at the transverse joints to minimize roughness due to faulting.

- The pavement sections should not exhibit any D-cracking or alkali-reactivity distress, nor should they exhibit any significant structural deterioration.

- At least two pavement sections are needed in each of the four climatic zones. It is desirable that a section of "smooth" pavement and a section of "rough" pavement be located within each section and adjacent to one another within that project so that traffic effects are constant.

- A minimum length of 300 m (1000 ft) is needed for each of the sections.

- Periodic profilograph traces and roughness data, including that obtained at initial construction, must be available for each of the sections.

The experimental design matrix for this study is shown in table 30.

Table 30. Experimental design matrix for roughness study of inservice pavement sections.

		CLIMATIC ZONE			
		Dry-Freeze	Wet-Freeze	Dry-Nonfreeze	Wet-Nonfreeze
HIGHWAY PROJECT NO. 1	Currently Smooth Section	Section 1	Section 5	Section 9	Section 13
	Currently Rough Section	Section 2	Section 6	Section 10	Section 14
HIGHWAY PROJECT NO. 2	Currently Smooth Section	Section 3	Section 7	Section 11	Section 15
	Currently Rough Section	Section 4	Section 8	Section 12	Section 16

Variables

It is observed from table 30 that only one variable, pavement roughness, is being evaluated. The intent is to determine if the roughness of a pavement as measured today is in any way related to its roughness at the time of construction. The availability of historical roughness data and records are essential to this study so that the effect of initial roughness on later roughness can be determined. The most effective approach is believed to be identifying smooth and rough segments on an existing pavement and then, by inspecting historical records, determining if that roughness is in any way related to the initial roughness.

It is important that each section have approximately the same design and design features (thickness, spacing, base type, and so on). The design should be structurally adequate and should contain dowel bars at the transverse joints to minimize the development of significant roughness due to faulting.

Special Data Needs

The availability of historical roughness data is essential to this study. Preferably, initial roughness data should be available, and it is highly desirable that roughness data be available for those sections on a yearly basis since their construction. The availability of actual surface profiles would greatly supplement the analysis.

Layout

For the roughness study, a minimum length of 300 m (1000 ft) is needed for each section. Therefore, assuming 300 m (1000 ft) per section, a total of 600 m (2000 ft) is needed within each project. Where needed, at least a 150-m (500-ft) transition segment should be provided between adjacent sections so that dynamic effects from section to section are minimized.

Length of Test Period

It is expected that results from this study would be available immediately upon evaluation of the initial and current roughness of the sections. However, it may be desirable to continue monitoring the pavement roughness of the sections for a short period of time (say, 5 years) so that the exact trends are identified. The evaluation of two sections within each region will increase the validity of the results, while the evaluation in each region will indicate if climatic effects have any impact on the rate that roughness develops.

Products for PRS

The primary product from the roughness experiment will be the acceptance or rejection of the assumption that smoother pavements deteriorate more slowly and last longer than rougher pavements of otherwise equivalent design. It may be possible to develop a model that predicts future roughness as a function of the initial roughness for a given subgrade and environmental loading. The development of such a model (if possible) will quantify the effects of an initially rough pavement on subsequent pavement roughness. It will help to identify those critical levels of initial roughness that may prolong pavement life, which, in turn, can be incorporated into a PRS.

Tiebar Placement Study (Distress Indicator 8)

During construction, deformed tiebars, typically No. 4 (13-mm [0.5-in]) or No. 5 (16-mm [0.62-in]) bars, are placed between adjacent traffic lanes or between a traffic lane and an adjacent tied concrete shoulder. Most tiebars are installed at mid-depth of the slab and at about 762- to 914-mm (30- to 36-in) intervals using a mechanical implanting device attached to the paver. The tiebars are needed to keep adjacent slabs from separating and to maintain load transfer across the joint.

Occasionally, the tiebars may not be placed to the specified depth. If the tiebar is placed too deep in the slab, it will not be as effective in keeping the adjacent slabs together and maintaining load transfer. If the tiebar is placed too shallow, it can create spalling along the longitudinal joint due to insufficient cover. It is these effects of improper tiebar placement that are to be investigated under this study.

Site-Specific Information

It is believed that this study can be accomplished using inservice concrete pavements. This is because sufficient sections with a range in tiebar depths are believed to exist. Two sections that are exhibiting spalling/scaling distress due to high tiebar placement should be selected from all four climatic regions. It is recommended that the sections be subjected to a moderate level of traffic (between 250,000 and 500,000 80-kN [18-kip] ESAL applications per year).

Pavement Design

The inservice pavements used in this study must meet certain criteria in order to be considered for inclusion:

- Pavement sections should be exhibiting some spalling/scaling distress due to high tiebar placement. It is desirable that the spalling/scaling distress be prominent throughout the length of the section. Measurements of tiebar depth will be taken using GPR and limited coring.

- Pavements may be either JPCP or JRCP.

- All pavement sections should be multilane facilities.

- The pavement sections should not exhibit any D-cracking or alkali-reactivity distress, nor should they exhibit any significant structural deterioration.

- At least two sections that exhibit spalling/scaling distress should be selected in each of the four climatic zones.

- A minimum length of 900 m (3000 ft) is needed for each of the sections.

- All pavement sections should be at least 10 years old and subjected to a moderate level of traffic.

The experimental design matrix for this study is shown in table 31.

Table 31. Experimental design matrix for tiebar placement study of inservice pavement sections.

	CLIMATE			
	Dry-Freeze	Wet-Freeze	Dry-Nonfreeze	Wet-Nonfreeze
Inservice Projects Exhibiting Spalling/Scaling Due to High Tiebar Placement	Section 1 Section 2	Section 3 Section 4	Section 5 Section 6	Section 7 Section 8

Variables

The primary variable of interest in this study is the depth of tiebar placement and how it relates to spalling or scaling. Since inservice pavements are recommended for this study, the actual depth of tiebar placement will not be controlled, but it is believed that a sufficient range of depths will be encountered on suitable sections included in the study.

During the evaluation of each section, the tiebar depths and associated levels of spalling will be noted for every tiebar. It is expected that this should provide information on the critical tiebar depth at which spalling develops or greatly increases. Roughness measurements will be made to determine the effect of the spalling/scaling on the performance of the pavement.

The use of GPR is recommended for determining the actual depth of the tiebars. This is a rapid and reliable means of measuring the depth of the tiebars over the length of the section.

It may be desirable to conduct falling weight deflectometer (FWD) testing across some of the longitudinal joints to determine load transfer efficiencies. It will also be necessary to retrieve some cores to calibrate the radar and to verify the tiebar depth in some of the particular cases. Cores, when taken, can be used for petrographic examination to look for cracking damage around high steel.

Layout

Within each climatic zone, two sections will be evaluated that are exhibiting spalling/scaling due to high tiebar placement. As previously indicated, a minimum section length of 900 m (3000 ft) is needed, meaning that a total length of at least 2750 m (9000 ft) will be monitored in each climatic zone.

Length of Test Period

Because inservice pavements are being used in this investigation, it is expected that results from the study would be available almost immediately after initial monitoring of the sections. However, it is suggested that monitoring of the sections continue for at least 5 years, so that all spalling/scaling due to high tiebar placement has sufficient time to fully develop.

Products for PRS

The main product from this study will be a model that predicts the occurrence of longitudinal joint spalling/scaling as a function of tiebar depth. It is expected that tiebars placed to a shallow depth will cause spalling/scaling of the longitudinal joint, whereas those placed at mid-depth will not create any distresses. The model developed under this study would incorporate the effects of different climates and different subgrades, since it will be based on inservice pavements.

Reinforcing Steel Placement Study (Distress Indicator 9)

Reinforcing steel is placed in both JRCP and CRCP to maintain slab integrity across transverse cracks. JRCP typically have transverse joints spaced at about 9- to 12-m (30- to 40-ft) intervals and a small amount of reinforcing steel (0.1 to 0.2 percent of the pavement cross section) to hold mid-slab cracks tight and prevent them from deteriorating. CRCP has no transverse joints and contains a much larger percentage of steel (typically 0.6 to 0.8 percent). This steel is designed to create regularly spaced transverse cracks and hold them tightly together to maintain aggregate interlock.

Reinforcing steel for JRCP generally consists of welded-wire fabric (WWF). During construction, the WWF may be placed between layers of concrete or may be depressed in the plastic concrete using a mesh depressor. A minimum cover of 51 to 76 mm (2 to 3 in) is generally specified.

Deformed bars are most often used for reinforcement in CRCP. The longitudinal reinforcement is considered most critical to the performance of the pavement, although some agencies place transverse reinforcement as well. The most common sizes of deformed bar that are used are No. 4 (13-mm [0.5-in]), No. 5 (16-mm [0.62-in]), and No. 6 (19-mm [0.75-in]) bars. The longitudinal reinforcing steel may be placed on chairs prior to paving or may be fed through tubes at the back of the paver. However, the tube feeders provide less control over the depth of placement of the reinforcing steel. Again, a minimum cover of 76 to 102 mm (3 to 4 in) is generally specified.

For both JRCP and CRCP, control of the depth of the steel placement is critical to ensuring that the transverse cracks are held tight and prevented from deteriorating under traffic loading. One study of CRCP showed that steel that was too deep in the slab caused extensive crack deterioration and punchouts.[20] This is because at too great a depth, the steel is ineffective in holding the crack tight and preventing it from deteriorating. On the other hand, if the steel is placed too high, spalling or scaling at the slab surface may result due to insufficient concrete cover.

In order to assess the effect of the depth of steel placement on pavement performance, it is proposed that a study be conducted of inservice pavements. Both inservice JRCP and CRCP should be examined. It is believed that there are a sufficient number of inservice JRCP and CRCP sections with steel located at different depths, so that a specially constructed experiment is not needed.

Site-Specific Information

Inservice JRCP and CRCP sections must be solicited from interested agencies for the study. These should be pavements that exhibit spalling distress due to high steel or ones that may be exhibiting severe crack deterioration or punchouts due to low steel. CRCP sections constructed using a tube feeder are prime candidates for inclusion in the study because the depth of steel under that type of construction is known to vary widely. During the pre-selection process, the depth of the reinforcing steel can be quickly determined using GPR to verify the appropriateness of a section.

Two sections that are exhibiting distresses that are known to be caused by improperly placed steel should be selected from each of at least two different climatic regions. A nearby section not exhibiting such distresses should be included as a "control" for comparison. By using inservice pavement sections, the effects of different climates and subgrades may be considered. It is recommended that the sections be subjected to a moderate level of traffic (between 250,000 and 500,000 80-kN [18-kip] ESAL applications per year).

Pavement Design

The inservice pavements to be used in this study must meet certain criteria to be considered for inclusion:

- All sections should be reinforced. It is desired that two distressed JRCP sections and two distressed CRCP sections be selected in each of two climatic zones. One control JRCP section and one control CRCP section ("control" indicating sections that are of similar design but are not exhibiting distresses that are known to be caused by improperly placed steel) should also be selected in each of the two climatic zones. Only two climatic zones are recommended because of the absence of reinforced pavements in certain regions of the country.

- Sections should be exhibiting distresses that are known to be caused by improperly placed steel. These can be projects suggested by the agencies, and the steel depth verified through the use of radar. It is desirable that the distress be prominent throughout the length of the project.

- All pavement sections should be multilane facilities.

- The pavement sections should not be exhibiting any D-cracking or alkali-reactivity distress, nor should they be exhibiting any significant structural deterioration.

- A minimum length of 900 m (3000 ft) is needed for each of the pavement sections.

- All pavement sections should be at least 10 years old and have been subjected to a moderate level of traffic.

The experimental design matrix for this study is shown in table 32.

Table 32. Experimental design matrix for reinforcing steel placement study of inservice pavement sections.

	Climatic Zone 1		Climatic Zone 2	
	JRCP	CRCP	JRCP	CRCP
Inservice Projects Exhibiting Distress Due to Improper Placement of Reinforcing Steel	Section 1 Section 2	Section 4 Section 5	Section 7 Section 8	Section 10 Section 11
Nearby Inservice Project of Similar Design Not Exhibiting Distress	Section 3	Section 6	Section 9	Section 12

<u>Variables</u>

The primary variable of interest in this study is the depth of the reinforcing steel. This must be examined for both JRCP and CRCP. Since inservice pavements are recommended for this study, the actual depth of reinforcing steel will not be controlled, but it is believed that a sufficient range of depths will be encountered on suitable pavement sections included in the study. The inclusion of "control" sections will be used to establish a baseline for the study. Distress and roughness measurements will be made on all sections to determine the effect of the improperly placed reinforcement on the performance of the pavement.

The use of GPR is recommended for determining the actual depth of the reinforcing steel. The depths to reinforcement would be measured along the entire section and locations of spalling noted so that correlations could be made. In addition to this testing, it may be desirable to conduct some FWD deflection testing across some of the cracks to determine load transfer efficiencies. It will also be necessary to retrieve some cores to calibrate the radar and to verify the reinforcing steel depth in specific cases.

Layout

Within each climatic zone, there will be six sections evaluated: two JRCP sections and two CRCP sections that are exhibiting distresses from improperly placed reinforcing steel, and one JRCP section and one CRCP section (of designs similar to their respective counterparts) that are not exhibiting any such distress. As previously indicated, a minimum project length of 900 m (3000 ft) is needed, meaning that a total length of at least 5500 m (18,000 ft) will be monitored in each climatic zone.

Length of Test Period

Because inservice pavements are used in this investigation, results from the study should be available almost immediately after initial monitoring of the sections. However, it is suggested that monitoring of the sections continue for at least 5 years so that the full effects of improperly placed reinforcing steel may be determined.

Products for PRS

This study will provide models that predict the development of crack deterioration and of surface spalling/scaling as a function of the depth of steel. Two such models will be developed, one for JRCP and one for CRCP. This will allow the consideration of depth of steel to be incorporated into a PRS.

DATA COLLECTION ACTIVITIES

Monitoring of the experimental pavement sections is needed to assess the effect of each of the variables being investigated. Extensive data collection activities are proposed to enable a complete and valid analysis. Various data collection activities are to be conducted on the sections at three different times: during initial project selection (or construction), during the evaluation and monitoring period, and after the section is taken out of service (*post mortem* evaluation).

Initial Data Collection

A certain amount of initial data collection efforts are needed once suitable sections have been identified. This information will serve as the basis for much of the later analysis. If the alternatives using specially constructed sections are selected, this information represents data that should be collected during their construction.

Environmental Data

Detailed weather information should be obtained from the construction records or recorded during any new construction. Environmental information to be collected includes:

- Maximum daily air temperature.
- Minimum daily air temperature.
- Daily precipitation.
- Humidity.
- Wind Speed.
- Solar conditions (sunny, cloudy, etc.).

It may be useful to obtain historical weather information for use in the data analysis. Such historical weather data may include:

- Average maximum daily temperature (by month).
- Average minimum daily temperature (by month).
- Average monthly and annual precipitation.
- Thornthwaite Moisture Index.
- Freezing Index.
- Average number of freeze-thaw cycles.
- Average percent sunshine.
- Maximum and minimum average solar radiation (by month).
- Average monthly wind speed.

Subgrade Data

Subgrade data for each of the sections should be obtained to identify the properties of the material. Test information on the subgrade should include:

- Gradation.

- Atterberg limits (liquid and plastic limits).

- Moisture-density relationships (maximum dry density and optimum moisture content).

- Strength tests (California Bearing Ratio [CBR], R-value, resilient modulus, or perhaps plate load tests).

- Shrink/swell potential or frost susceptibility tests.

For any new construction, sufficient sampling and testing should be conducted to account for soil variability throughout the proposed construction site.

Pavement Construction Data

Pavement construction data may be informative for several of the data analyses. Certainly, any process control data (air content, slump, strength, and so on), if available, would be useful. However, not all of this information is expected to be available.

If any of the alternatives for specially constructed pavement sections are exercised, certain construction factors must be carefully supervised. The following items should be noted during construction:

- The depths and alignment of all steel (dowel bars, tiebars, reinforcing steel) should be checked prior to paving. After construction, the depths and alignment of the steel can be verified by using GPR. Since the sections to be monitored are relatively short, it is recommended that 100-percent sampling be done to identify steel locations.

- Joint sawing activities should be carefully planned so that both early and late sawing is conducted, and to ensure that the proper depths of sawing are made. Actual time and depths of sawing should be recorded.

- The initial roughness should be measured on all sections before being opened to traffic. The California profilograph (or equivalent roughness measuring device) should be used. For consistency, it is recommended that the same device be used on the section throughout the monitoring period.

- Temperatures at both the bottom and the top of the concrete slab should be monitored during the first 24 h of the joint sawing study. Concrete strengths should also be monitored during this period through either maturity or pulse velocity testing.

- Immediately after construction, two cores should be retrieved from all sections to verify thickness and strength. These cores could be inspected to verify depth to reinforcement on JRCP. A separate core should be obtained from each of the sections in the air-void/consolidation study to determine the percent consolidation and to determine the air content and related parameters (spacing factor, specific surface, and voids) of the hardened concrete; the latter testing will require a linear traverse. Joint cores could also be taken to verify location of dowels and tiebars.

- Cores of 102-mm (4-in) diameter should be sufficient for most coring activities.

Data Collection During Test Period

Distress Surveys

It is recommended that distress surveys be conducted on all experimental sections at least annually. It is envisioned that the most efficient way of accomplishing this will be through the use of automated methods, similar to what is being used in the Long-Term Pavement Performance (LTPP) program. Data that will be collected in this manner include:

- Transverse cracking.
- Longitudinal cracking.
- Joint spalling.
- Joint seal damage.
- Corner breaks.
- Pumping (visible staining).

Current automated survey equipment is not capable of measuring transverse joint faulting. This would have to be measured manually for each of the sections. At that time, joint openings should also be obtained, along with representative photographs or videotapes of each section to provide a permanent record of its condition. An evaluation can also be made of the drainage conditions of the roadway.

Nondestructive Testing

Falling Weight Deflectometer (FWD) Testing

A limited amount of FWD testing is proposed for the monitoring of these sections. This is because the focus of the experiments is assessing the way that quality characteristics affect pavement performance as measured by key distress indicators (cracking, spalling, roughness, and so on). Information provided by the FWD relates to a pavement's structural capacity (load transfer and backcalculated modulus values), which is not really needed in developing relationships between the quality characteristic and the pavement performance. However, FWD testing may be of interest for the following experiments:

- Dowel misalignment experiment to evaluate the load transfer conditions at the transverse joints.

- Consolidation study and air-void study to detect imminent deterioration brought about by insufficient consolidation or an inadequate air-void system.

- Tiebar placement study to monitor load transfer efficiencies over time. Tiebars that are placed too deep or too shallow may not provide adequate load transfer across the joint.

The testing for these experiments could be conducted on an annual basis, with the first evaluation done before the sections are opened to traffic. Deflection testing should be conducted twice during the day of testing so that slab curling effects may be taken into account.

Ground Penetrating Radar Testing

Ground penetrating radar testing is needed for three of the proposed studies: dowel misalignment, tiebar depth, and steel depth. For each of these studies, GPR first must be used to identify candidate sections for inclusion in the study, and then it can be used to determine the actual depths of the steel. A limited amount of coring may be needed in conjunction with the GPR testing for verification purposes.

Coring, Boring, and Material Sampling

Additional destructive testing of the pavement sections may be required during the test period. Possible examples of such testing include:

- Coring of cracks in the slab to assist in identifying their cause.
- Coring of inservice JPCP to determine and verify tiebar steel depths (as needed).
- Coring of inservice JRCP and CRCP to determine and verify reinforcing steel depths (as needed). These cores can also be examined to identify any cracking damage around the high steel.
- Boring of base and subgrade material to investigate frost depth.
- Coring of joints to investigate extent of deterioration, both beneath the slab and around dowel bars.
- A linear traverse must be conducted on joint cores for the air-void system/ consolidation study to determine the critical air-void system parameters.

Roughness

Roughness measurements are needed on all experimental sections. This is to provide a basis for assessing the development of the distresses on pavement performance. For any new sections constructed, initial roughness must be measured on all sections immediately after construction and before being subjected to traffic loading. In the case of the inservice sections, an initial roughness value should be recorded when the monitoring of the section begins. Thereafter, it is expected that annual or semi-annual measurements will be required for all of the experimental sections.

There are several different means of obtaining pavement roughness, perhaps the most efficient is the collection of profile using equipment and procedures employed in the LTPP program. The profile data collected in this manner can be expressed in a number of ways, including in International Roughness Index (IRI) units. This can be correlated to the present serviceability index, which is used in many of the pavement prediction models.

Traffic Loading

An accurate representation of the actual traffic loading is required for the experimental sections, so that traffic effects can be accounted for in the progression of the pavement distress. Although this can be accomplished in a number of ways, the most effective is the use of weigh-in-motion (WIM) technology. This technology obtains vehicle classification and axle weight data and distribution for traffic as it is moving, thereby eliminating delays. It is recommended that WIM data be collected and used to supplement computations of traffic loadings.

Post Mortem Data Collection

The experimental pavement sections should be monitored for the prescribed time period, or until they have reached a critical level of service (e.g., a PSI of 2.5). At that time, the section can be considered to be out of service since it is in need of some sort of rehabilitation. A final distress survey should be conducted at that time for use in the development of prediction models.

There are several alternatives for treating a section that is "out of service." One alternative is to repair or overlay the section to keep it in service. Another alternative is to remove and replace the section, after first performing a *post mortem* analysis. This is probably the preferred approach since some valuable information could be gained by doing a *post mortem* analysis at that time. Among the types of data that could be collected from *post mortem* analyses are:

- Determination of depth of transverse and longitudinal cracking.
- Determination of depth of tiebars and reinforcing steel (where appropriate).
- Investigation of deterioration at the bottom of transverse joints.
- Inspection of misaligned dowel bars.
- Examination of transverse joints for loss of support.

During the course of the study, it is expected that additional items will be identified that should be inspected during the *post mortem* analysis.

DATA ANALYSIS PLAN

Data Base Development

It is important that a computerized data base be developed for the data collected from each study so that the analysis of the data collected from the various studies produces useful results. To make the most use of the data collected, it is essential that the data are incorporated into a data base management system that will allow storage, retrieval, and analysis in a user-friendly, systematic, and efficient manner. It is expected that the data base currently used in the FHWA Long-Term Pavement Performance (LTPP) program could be easily adapted for use on these studies. Like the data collected under the FHWA LTPP program, the data collected for these studies could be classified into seven modules:

- Inventory.
- Materials Testing.
- Climate.
- Maintenance.
- Rehabilitation.
- Traffic.
- Monitoring.

Each module is, in turn, made up of a number of tables that contain specific information or data elements on a particular aspect of the pavement sections. Once pavement monitoring begins, that module will receive updates under each study.

Data Analysis

The primary data analyses performed under this study will be the development of concrete performance prediction models for the previously identified distress indicators. Each of the proposed studies is set up in such a way so as to facilitate the development of prediction models for use in a PRS. This is because the prediction models that will be developed will be used for predicting future pavement deterioration so that future maintenance and rehabilitation costs can be estimated.

The data analysis work must first begin with an evaluation of the data base developed to ensure the integrity of the data assembled for each study. The examination of the data will involve the application of various statistical procedures to the data to examine the ranges of the various variables, their distribution characteristics, and any subtle anomalies.

After the examination of the data base, the effects of the key factors (and any significant interactions) on the progression of pavement distress and performance should be evaluated. Analysis of variance and regression techniques should be used to determine the significant deteriorative effects of the main factors and their interactions. In addition, any current prediction models should be considered to provide insight into model functional forms, independent and dependent variables, and interaction of variables.

Regression techniques should be used to examine all possible linear relationships between the independent variables. The models developed from such linear regression analysis will be useful in selecting the variables that should be considered in the main model formulation. Nonlinear regression techniques can then be applied to determine the unknown constants in the form of the models identified through linear regression. A powerful statistical software program, such as the SAS statistical package, will be needed for the data analysis and model development.

If all of the studies are fully implemented, it is expected that the following prediction models could be developed for the specific pavement designs investigated:

- A model predicting the amount of transverse cracking based on the depth and timing of transverse joint sawing operations.
- A model predicting the amount of longitudinal cracking based on the depth and timing of longitudinal joint sawing operations.
- A model predicting the amount of transverse joint spalling/ravelling based on the timing of transverse joint sawing operations.
- A model predicting the amount of longitudinal joint spalling/ravelling based on the timing of longitudinal joint sawing operations.
- A model predicting the amount of joint deterioration (spalling) based on the amount of air (or some other air-void parameter), the concrete strength, the number of freeze-thaw cycles, and the use of salt.
- A model predicting the amount of joint deterioration based on the degree of consolidation.
- A model predicting the amount of joint deterioration (spalling or cracking) based on the type and amount of dowel misalignment.
- A model predicting the progression of pavement roughness based on the initial roughness of the pavement.
- A model predicting the amount of longitudinal joint spalling based on the depth of the tiebars between lanes.
- A model predicting the amount of deteriorated cracks (JRCP) or punchouts (CRCP), based on the depth of the reinforcing steel.

Since these models quantify the effect of these key quality characteristics on concrete pavement performance, they can be directly incorporated into a PRS.

SUMMARY

Several key quality characteristics (such as dowel misalignment or depth of steel) that are under the control of the contractor and that are known to affect the performance of the pavement are not represented in a current PRS. This is because relationships between those quality characteristics and the ensuing pavement performance has not been quantified. For example, dowel misalignment is generally regarded as a critical construction item that can influence the performance of the pavement, yet there are no relationships that predict the development of joint distress as a function of the amount of dowel misalignment.

Experimental plans are presented for the evaluation of key distress indicators not currently considered in a performance-related specification. These experimental plans consist of six studies intended to either fill in missing areas in a PRS or supplement existing ones so that a comprehensive PRS may be developed. Each of the experiments includes a summary of the experimental design, a description of the recommended pavement design characteristics, a description of the variables being investigated, a discussion of any special construction requirements, a description of the test section layout and length of test period, and a summary of the expected products that will be available from the study. A summary of the six studies is provided in table 33.

All of the studies emphasize the use of inservice pavements for evaluating the key quality characteristics. This approach is less expensive than using specially constructed sections and should provide more immediate results. However, some control over the various factors is lost.

For some of the studies, several experimental plans for the use of specially constructed sections are provided as an alternative. These types of studies provide for the most control over the many different variables, but tend to be very expensive and may not produce immediate results. In addition, there may be a reluctance to construct such pavements that are destined to fail prematurely.

The various experiments are presented as independent studies so that interested agencies could select those experiments that they feel are most important to their concrete pavements. In this way, those distress indicators of interest may be evaluated by an agency and the results implemented in its specifications.

In addition to the proposed experimental plans, recommended data collection and data analysis plans are presented. These plans summarize the recommended data to be collected in the field and the suggested approach for the analysis of the data.

Table 33. Summary of proposed field studies.

Study	Distress Indicators Being Evaluated	Associated Quality Characteristics	Type of Field Study	Results Available
Joint Sawing Study	• Transverse and longitudinal cracking due to late sawing • Transverse and longitudinal joint spalling due to early sawing	• Depth of joint sawing • Timing of joint sawing	• Inservice pavements (JPCP only) • *(New Construction Alternative)*	• 0 to 5 years • *(2 to 10 years)*
Air System/ Consolidation Study	• Joint deterioration due to inadequate air system • Joint deterioration due to inadequate consolidation	• Air-void distribution • Percent consolidation • Freeze-thaw cycles • Salt application	• Inservice Pavements (JPCP only) • *(New Construction Alternative)*	• 0 to 5 years • *(10 to 20 years)*
Dowel Misalignment Study	• Joint deterioration due to misaligned dowel bars	• Amount of misalignment • Number of bars misaligned • Joint movement	• Specially constructed (JPCP only) • *(New Construction Alternative)*	• 0 to 5 years • *(2 to 10 years)*
Pavement Roughness Study	• Progression of pavement roughness as a function of initial roughness	• Initial pavement roughness	• Inservice pavements (JPCP only)	• 0 to 5 years
Tiebar Placement Study	• Longitudinal joint spalling due to high placement of tiebars	• Depth of tiebars	• Inservice pavements (JPCP and JRCP)	• 0 to 5 years
Reinforcing Steel Placement Study	• Surface spalling due to high steel • Crack deterioration and punchouts due to low steel	• Depth of reinforcing steel	• Inservice pavements (JRCP and CRCP)	• 0 to 5 years

CHAPTER 6. SUMMARY AND RECOMMENDATIONS

SUMMARY

Summary of Prototype PRS

A comprehensive, prototype performance-related specification (PRS) for concrete pavements is presented. This specification is based in part on previous specifications and research, particularly the pioneering groundwork laid by the New Jersey DOT. The specification considers the life-cycle cost of the as-constructed pavement as the overall measure of quality, and compares that to the life-cycle cost of the as-designed pavement to develop appropriate pay adjustments. The pay factor is computed using equation 1, repeated here for convenience:

$$\text{Pay Factor} = 100\,(\text{BID} + \text{DIFF}) \,/\, \text{BID} \tag{24}$$

where:

BID	=	Contractor's bid price for the lot, $
DIFF	=	$LCC_{des} - LCC_{con}$
LCC_{des}	=	As-designed life-cycle cost for lot, $
LCC_{con}	=	As-constructed life-cycle cost for lot, $

It is observed from this equation that both positive and negative pay adjustments are possible. The approach is in accordance with the legal principle of liquidated damages, as advocated by Weed and others.[2,3,4] The liquidated damages are computed at the time of construction based on the projected increase or decrease in future costs.

By using the life-cycle cost as the overall quality measure, the specification is able to address many of the limitations of current specifications. For instance, the new specification offers the following advantages:

- The specification is driven by key distress indicators that control the performance, and hence the LCC, of the pavement. Currently, only four variables (strength, thickness, air content, and roughness) are accounted for, although other variables can easily be added as prediction models become available.

- Multiple quality characteristics are rationally considered in the development of pay adjustments. Virtually an unlimited number of quality characteristics can be considered provided that there exists a prediction model that relates the quality characteristic to pavement performance, and that a suitable maintenance/ rehabilitation program exists for responding to all important distresses.

- Within-lot variability of the quality characteristics is directly considered. Many specifications only use the mean value, while ignoring the variation associated with the quality characteristics. The proposed specification directly considers the within-lot variability and accounts for it in the determination of the pay schedule.

- A rational procedure is presented for computing pay adjustments based on the legal principle of liquidated damages. The procedure provides incentive to the contractor to provide high-quality work by allowing positive as well as negative pay adjustments.

- The specification requires testing of the *in situ* concrete pavement through coring and testing to provide a true assessment of its as-constructed properties and its expected performance.

Summary of Laboratory Studies

In support of the development of the prototype PRS, extensive laboratory testing was conducted to fill several gaps in the materials area. Specifically, the laboratory testing was conducted to investigate key relationships between concrete material quality characteristics and two pavement distress indicators: transverse cracking caused by repeated loading and thermal curling, and joint spalling caused by an inadequate air-void system.

The following is a summary of the results from the laboratory study:

- The first part of the laboratory materials study investigated factors that affect concrete strength and modulus of elasticity, factors that are under the control of the paving contractor and can significantly influence concrete pavement performance in terms of the development of transverse cracking. Several mix design variables (coarse aggregate type, cement content, air content, and so on) were evaluated to determine their effect on concrete strength and elastic modulus. As expected, flexural strength was most sensitive to changes in water-cement ratio.

- Simple interstrength relationships were derived for fully consolidated mixes. Although significant relationships could be developed between flexural and splitting tensile strength, between flexural and compressive strength, between splitting tensile and compressive strength, and between elastic modulus and compressive strength, no general relationships independent of mix components could be established. This emphasizes the need for project-specific strength interrelationships.

- Mix-specific relationships were examined to evaluate errors in predicting one hardened concrete property from another. The average flexural strength prediction error from compressive strength was 2.6 percent and that predicted from splitting tensile strength was 2.7 percent. Plots of best fit regression equations indicated that consolidation effects were not completely accounted for within the strength relationships.

- A study comparing the compressive strengths of cores to the compressive strengths of cylinders was conducted. For cores and cylinders cured under identical conditions (same maturity), no significant differences were observed between core strengths and cylinder strengths of eight different mixes tested at 7, 14, and 28 d.

- The use of maturity and pulse velocity for monitoring *in situ* slab strength and elastic modulus development was demonstrated in a portion of the laboratory study. Steps to adjust core compressive strength (at any maturity) to a standard, laboratory-cured compressive strength were outlined. The *in situ* slab maturity and core compressive or splitting tensile strength can be used in mix-specific, strength-maturity relationships. Once a standard-cured compressive strength is established, mix-specific relationships are used to predict the standard-cured flexural strength.

- To address durability problems in the development of PRS, a laboratory testing program was conducted to correlate air-void system parameters with joint spalling. Block specimens with joints were monitored to evaluate the progression of joint deterioration over a total of 300 freeze-thaw cycles. The presence of salt in the ponding solution was noted to have a tremendous impact on spalling and scaling, whereas those samples without salt in the ponding solution exhibited no significant joint deterioration observed after 300 cycles. Three different air models were developed in the durability spalling study, each a function of either air content or the void spacing factor.

Several of the results of the laboratory investigation were incorporated into the prototype performance-related specification given in appendix A. For example, the use of cores is recommended in the specification, and the laboratory work showed that no adjustments are needed to convert core compressive strengths to equivalent cylinder compressive strengths. However, more cores are required to minimize the effects of the larger variability associated with core strengths. Also, the results from the laboratory durability study were used to modify a joint spalling performance model as a function of air content, the presence of deicer solution, the compressive strength, and the number of freeze-thaw cycles.

Summary of Proposed Accelerated Field Studies

Test plans for the evaluation of various construction variables have been developed. Experiments have been established for the evaluation of the following distress indicators:

- Transverse cracking due to late sawing.
- Longitudinal cracking due to late sawing.
- Joint spalling due to early sawing.

- Transverse joint deterioration due to poor consolidation.
- Spalling/scaling due to an inadequate air-void system.
- Transverse joint deterioration due to dowel misalignment.
- Effect of initial roughness on subsequent pavement performance.
- Longitudinal joint spalling due to tiebar misalignment.
- Spalling/scaling due to high steel.

These studies are needed so that the effect of other quality characteristics that are under the control of the contractor may be quantified and eventually included in the specification.

RECOMMENDATIONS FOR FUTURE PRS DEVELOPMENT

The prototype specification produced under this study represents a major step in the continued development and evolution of a comprehensive performance-related specification for concrete pavements. However, the specification is by no means complete, as a great deal of work remains to be conducted to continue its development, extend its applicability, and improve its capabilities. Some of this work includes:

- Extensive testing, validation, and verification of the specification under simulated and actual construction conditions are needed.

- Additional research is needed on values of material and testing variability that contractors are currently able to achieve for all of the quality characteristics.

- Although the PaveSpec computer program has been developed to assist in simulation and in generating pay factors, software for use with the specification needs to be developed in which testing results can be directly entered and the corresponding pay factors produced.

- The specification, currently developed only for jointed pavements, needs to be expanded for continuously reinforced concrete pavements.

- Improved prediction models are needed that relate the quality characteristics of a mix to pavement performance.

- Additional construction-related variables need to be included in the specification, as appropriate.

- The development and use of rapid tests for the *in situ* quality characteristics must be encouraged.

- The specification should be expanded to include all parts of concrete highway construction, including joints, reinforcement, base, subbase, subgrade, and shoulders. The basis for the PRS developed in this study can incorporate these additional elements.

- The specification should be flexible to allow individual agencies the freedom to include their own performance criteria and rehabilitation strategies.

- The approach outlined for concrete pavements is also applicable to other pavement types. Efforts would be needed to define the key distress indicators and key *in situ* quality characteristics for other pavements, and also to identify or develop suitable performance prediction and cost models.

- Industry must continue to play a key role in the development of performance-related specifications to ensure the development of a rational and equitable specification.

IMPLEMENTATION CONCERNS

The prototype PRS developed in this study is a complicated procedure that requires numerous inputs, many of which are currently difficult to estimate for a given contract. In addition, the PRS is based on estimating the LCC of a pavement that in itself has several advantages and disadvantages.

The main advantage of the LCC approach is that it is possible to realistically consider any number of quality characteristics (both means and variations) in the rational calculation of a single pay factor for the lot. This approach can be extended to include all aspects of the pavement/subgrade system. There are no judgments required as to how to combine several different pay factors into a single pay factor for the lot. The primary disadvantage is that the calculation procedures for LCC are very controversial, and of themselves raise many questions. In addition, the computation of LCC for a lot that included variation of quality characteristics is a very difficult technical problem that is only solved approximately in the prototype specification.

The implementation of this prototype PRS will require further testing and evaluation of the technical and practical aspects of the specification. Further sensitivity and evaluation of the prototype PRS and the PaveSpec computer program may show that it can be simplified, without great loss of accuracy, to make it far easier to use in the field. This would involve the development of pay factor equations through regression analyses based upon many runs of PaveSpec for a range of project conditions. Future work should focus on this important aspect of PRS implementation.

REFERENCES

1. Afferton, K. C., J. Freidenrich, and R. M. Weed, *Managing Quality: Time for a National Policy*, Transportation Research Record 1340, Transportation Research Board, Washington, DC, 1992.

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

3. Anderson, D. A., D. R. Luhr, and C. E. Antle, *Framework for Development of Performance-Related Specifications for Hot-Mix Asphaltic Concrete*, NCHRP Report No. 332, Transportation Research Board, Washington, DC, 1990.

4. Irick, P. E., S. B. Seeds, M. G. Myers, and E. D. Moody, *Development of Performance-Related Specifications for Portland Cement Concrete Pavement Construction*, FHWA-RD-89-211, Federal Highway Administration, Washington, DC, 1990.

5. *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, DC, 1986.

6. Darter, M. I., J. M. Becker, M. B. Snyder, and R. E. Smith, *Portland Cement Concrete Pavement Evaluation System (COPES)*, NCHRP Report No. 277, Transportation Research Board, Washington, DC, 1985.

7. Chamberlin, W. P., *Performance-Related Specifications*, Draft Report, NCHRP Project 20-5, Topic 23-05, Transportation Research Board, Washington, DC, December 1992.

8. *Standard Recommended Practice for Acceptance Sampling Plans for Highway Construction*, AASHTO Designation R9-90, American Association of State Highway and Transportation Officials, Washington, DC, 1990.

9. Barros, R. T., R. M. Weed, and J. H. Willenbrock, *Software Package for Design and Analysis of Acceptance Procedures Based on Percent Defective*, Transportation Research Record 924, Transportation Research Board, Washington, DC, 1983.

10. Willenbrock, J. H. and P. A. Kopac, *Development of Price-Adjustment Systems for Statistically Based Highway Construction Specifications*," Transportation Research Record 652, Transportation Research Board, Washington, DC, 1977.

11. Smith, K. D., A. L. Mueller, M. I. Darter, and D. G. Peshkin, *Performance Of Jointed Concrete Pavements, Volume II—Evaluation and Modification of Concrete Pavement Design and Analysis Models*, FHWA-RD-89-137, Federal Highway Administration, Washington, DC, 1990.

12. *ACI Standard Recommended Practice for Evaluation of Strength Test Results of Concrete*, ACI 214-77 (re-approved 1989), American Concrete Institute, 1992.

13. *Cold Weather Concreting*, ACI 306-88, American Concrete Institute, 1992.

14. Carino, N. J., "The Maturity Method," Chapter 5 of *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, Boca Raton, FL, 1991.

15. Okamoto, P. A., P. J. Nussbaum, K. D. Smith, M. I. Darter, T. P. Wilson, C. L. Wu, and S. D. Tayabji, *Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements*, FHWA-RD-91-079, Federal Highway Administration, Washington, DC, October 1991.

16. Johnson, K. D., E. H. Rmeili, and M. I. Darter, *Development of Maintenance and Rehabilitation Strategies—Northern Illinois Toll Highway, East-West Tollway Extension*, Phase II Final Report, Illinois State Toll Highway Authority, December 1990.

17. Smith, A. R. and S. W. Benham, "Effect of Dowel Bar Misalignment Across Concrete Pavement Joints," *Transactions*, American Society of Civil Engineers, Volume 103, 1938.

18. Friberg, B. F., "Design of Dowels in Transverse Joints of Concrete Pavements," *Transactions*, American Society of Civil Engineers, Volume 105, 1940.

19. Tayabji, S. D., *Dowel Placement Tolerances for Concrete Pavements*, Transportation Research Record 1062, Transportation Research Board, Washington, DC, 1986.

20. Roman, R. J. and M. I. Darter, *Pavement Distress Study, Interstate Route 77, Fairfield and Chester Counties, South Carolina*, South Carolina Department of Transportation, March 1988.

* U.S. G.P.O.:1993-301-717:80311