Incremental Costs and Performance Benefits of Various Features of Concrete Pavements

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FOREWORD

The primary product developed under this study is a software analysis tool that can evaluate the relative costs and performance benefits associated with adding different features to a portland cement concrete (PCC) pavement design. The tool is for pavement designers who are interested in comparing costs versus performance associated with the selection of design features during the PCC pavement design process. This software is only a computational tool. It is not intended to provide absolute answers on the effect of different design features, but rather to offer insight into general cost and performance trends associated with the use of different design features. As such, it also may be of use to State and contractor engineers responsible for fulfilling the design, as well as to educators and students who wish to better understand the PCC pavement design process and its impact on construction costs and pavement performance.

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Director, Office of Infrastructure
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16. Abstract

Various design features (such as dowel bars, tied shoulders, or drainable bases) may be added to a PCC pavement design to improve its overall performance by maintaining a higher level of serviceability or by extending its service life. However, the addition of these features also increases the initial cost of the pavement design, in some cases quite significantly. This then raises the question as to whether the improved performance benefits gained by adding the design features are worth the increase in cost. Furthermore, the effects of adding more and more design features to a PCC pavement design may produce smaller and smaller performance gains, while significantly increasing the overall costs of the pavement structure. Unfortunately, current design practices do not always consider this trade-off between performance benefits and costs when design features are added to a PCC pavement design.

This report presents a methodology for quickly assessing the relative costs and benefits of incorporating various design features in PCC pavements. That methodology has been incorporated into an analytical software tool that can be used by pavement design engineers who are interested in investigating the cost versus performance trade-offs associated with the selection of different design features during the PCC pavement design process. The tool is not intended to provide absolute answers on the effect of different design features, but rather to provide insight into general performance and cost trends associated with the use of those design features.

17. Key Words

concrete pavement, rigid pavement, design features, pavement performance, costs, cost effectiveness

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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 yard	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 degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		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	35.314	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 degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		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 March 2003)

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CHAPTER 1. INTRODUCTION

Introduction

The design of portland cement concrete (PCC) pavements is more than the determination of slab thickness. Other components of a PCC pavement "system"—such as transverse joint design, base type, drainage design, and shoulder type—have a significant impact on pavement performance and therefore must be considered during the structural design process. These components, often referred to as design features or design elements, must be carefully selected for the environment and traffic loading conditions to which a specific PCC pavement is exposed. As shown in figure 1, the careful selection of these features can result in an enhanced design that maintains a higher level of serviceability or extends the performance life of the pavement. Over the years, various studies have documented the importance of many of these design features to overall PCC pavement performance.[1,2,3,4]

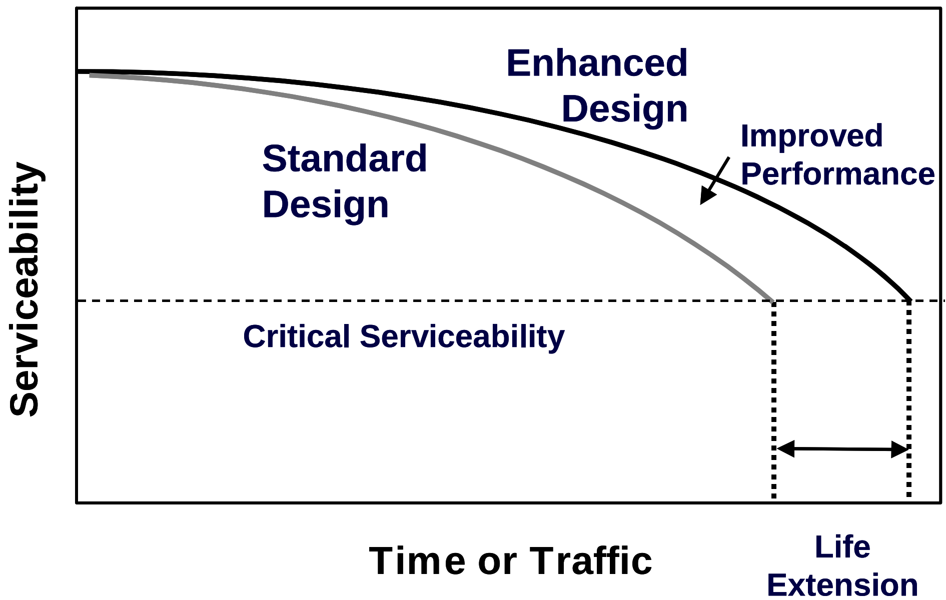


Figure 1. Design feature benefits.

While it is known that the combination of certain design feature choices can have a positive effect on pavement performance, the addition of such features to a PCC pavement also increases the overall cost of the pavement structure. This raises the question of whether the addition of the design features is worth the increase in cost; that is, are the design features cost effective? Although the pavement designer may implicitly believe that the design features are cost effective, this is not necessarily the case for several reasons. For one, the effects of individual design features on pavement performance are not independent from one another, and consequently some level of "performance redundancy" may occur. Furthermore, at some point the inclusion of additional design features may eventually reach a state of diminishing returns, in which fewer performance benefits are gained for ever-increasing costs of new design features. This suggests the need to consider both the performance benefits and the costs of design features when contemplating their use in a PCC pavement design.

Unfortunately, current design practices do not always consider this trade-off between performance benefits and costs when considering PCC pavement design features. In fact, these relationships are poorly understood. Thus PCC pavements are often designed as "premium" pavement structures, incorporating myriad design features without any realistic expectations of the performance benefits to be gained. This suggests that there is a strong need for a methodology and an evaluation tool that can be used to assess the costs and benefits of incorporating design features in PCC pavements.

Project Objectives

This project was initiated by the Innovative Pavement Research Foundation (IPRF) and continued under the auspices of the Federal Highway Administration (FHWA), with the objective of developing a simple methodology for comparing the impact of various PCC pavement design features on cost and performance of PCC pavements. Specifically, the overall objectives of this project are to:

- Document the relative performance benefits of different PCC pavement design features on PCC pavement performance.
- Establish relative construction costs associated with different PCC pavement design features.
- Develop a computer software tool for comparing and evaluating trade-offs in assessing the relative performance benefits and costs of various design features.

The primary product of this project is a computer software application that can be used to evaluate the benefits and costs associated with the addition of different design features to a PCC pavement design. The tool provides insight into general performance and cost trends associated with those modified pavement designs, and can help design engineers develop more cost-effective PCC pavement designs. However, it is emphasized that the tool is not intended for the design of PCC pavements, but rather for the assessment of the cost-effectiveness of new design features by considering the trade-offs between their cost and performance contributions.

Overview of Report

This report has three chapters (in addition to this one) and four appendices. Chapter 2 describes data collection activities conducted under the project, including a summary of the survey results collected from highway agencies and paving contractors. Chapter 3 describes the approach used in evaluating relative performance benefits and costs, and introduces the computer software tool developed under the project. Chapter 4 summarizes the entire project.

Appendix A summarizes the results of the comprehensive literature search conducted to identify information on PCC pavement design features, performance, and costs. Appendix B contains the performance and cost survey forms that were sent to highway agencies and paving contractors, respectively. Appendix C contains a summary of the raw survey data collected from the highway agencies and paving contractors. Appendix D is a user's manual for the computer software tool.

CHAPTER 2. DATA COLLECTION ACTIVITIES

Introduction

A significant data collection effort was required to obtain the information needed to fulfill the overall project objectives. This data collection effort consisted of:

- Literature review. A detailed literature review was conducted to identify the available information on costs and benefits of various PCC pavement design procedures, and to provide a foundation for the pavement design categories and design features to be evaluated through the questionnaire surveys.
- Questionnaire surveys of State Highway Agencies (SHAs). To provide information on the relative performance benefits of various PCC pavement design features, questionnaire surveys were submitted to targeted SHAs. These surveys solicited information on what performance benefits could be expected when adding selected PCC pavement design features to a *Standard* pavement design.
- Questionnaire surveys of PCC paving contractors. Questionnaire surveys were also sent to PCC paving contractors to obtain information on the costs associated with various PCC design features. These surveys solicited information on what the relative increase in cost might be for adding PCC pavement design features to a *Standard* pavement design.

The development, distribution, and collection of the questionnaire surveys as well as the overall processing of the data represented a major work effort. The processed and summarized performance and cost data serve as the "default" database (i.e., default cost and performance data sets) for use in evaluating the relative cost and performance benefits of each PCC pavement design feature.

This chapter describes each of the primary data collection activities mentioned above. The overall approach and methodology used in each activity are described, along with a summary of how the collected data were used in this project.

Literature Review

The detailed literature review for this project aimed to identify pertinent reference documents discussing the costs and performance benefits of different PCC pavement design features. A previous literature search conducted for a National Highway Institute (NHI) training course was the basis for the search conducted for this project.[(5)] However, the previous search by Smith and Hall targeted pavement design and performance information for the three major PCC pavement types (jointed plain concrete pavements (JPCP), jointed reinforced concrete pavements (JRCP), and continuously reinforced concrete pavements (CRCP)), so portions were not necessarily applicable to the current study which focuses primarily on JPCP designs and on the costs and performance benefits of PCC design features. Moreover, additional information was sought on pavement cross sections, PCC strength, PCC materials, and ride specifications, as these saw limited coverage in the original literature review.

To supplement the applicable documents identified in the original literature review, additional targeted searches were completed using the Transportation Research Information Services (TRIS), National Technical Information Services (NTIS), and Engineering Index (EI Compendex) national bibliographic databases. These searches were limited to recent publications (TRIS: 1995 to 2000; NTIS: 1990 to 2000; EI Compendex: 1998 to 2000). The search identified more than 100 additional records that were deemed potentially useful to the project.

After reviewing the records identified in the new literature searches, the researchers prepared a short annotation describing the record's content. A final annotated bibliography was prepared and grouped by the following topics:

- Base/Subbase Design.
- Concrete Mix Considerations.
- Construction.
- Costs.
- Cross Section.
- Design and Performance.
- Drainage.
- Joint Design.
- Maintenance.
- Reinforcement.
- Ride Specifications.
- Shoulder Design and Performance.
- Subgrade.
- Surface Texture and Noise.

The final annotated bibliography (appendix A) is not intended to be a comprehensive coverage of every PCC pavement design and performance study; instead, it is a listing of pertinent recent reports and papers (those published within the past 10 years) that provide guidance on the costs and performance of PCC pavement design features. The key documents were reviewed and used as a foundation for the pavement design categories and design features to be evaluated in the study, and also provided insight into general cost and performance trends.

Questionnaire Survey Development

This section describes the data collection effort for the performance and cost surveys conducted for this project. First, general background information is provided on the PCC pavement design features included in the surveys and on the construction of the surveys themselves. This is followed by a description of the data collection processes utilized in both the SHA (relative performance) surveys and the PCC paving contractor (relative cost) surveys.

Design Categories and Alternative Design Features

Based on the literature review, 10 primary categories of PCC pavement design features were identified, along with a list of possible design feature alternatives within each category. Because pavement designs can vary considerably within each of the design feature categories, one feature was selected to be the "standard" feature within that category. The collection of "standard" features from all design feature categories then represents the *Standard* PCC pavement cross section. The *Standard* PCC pavement cross section is the basis for comparison and allows for the determination of the incremental increase (or decrease) in cost and performance relative to that standard design (instead of, for example, each agency's standard design).

The final design feature categories and alternative design features considered within each category are summarized below. Those design features included as part of the *Standard* pavement design are indicated by "STD." A two-lane roadway (two lanes in one direction and part of a four-lane divided highway) is assumed as the *Standard* construction section.

- Subgrade
  - Prepared subgrade (natural soil, no other treatment) (STD).
  - 300-mm (12-inch) lime-treated subgrade (and elimination of 150-mm (6-inch) dense-graded aggregate base).

- Base/Subbase
  - 150-mm (6-inch) dense-graded aggregate base on prepared subgrade (STD).
  - No base (PCC pavement placed directly on prepared subgrade).
  - 150-mm (6-inch) asphalt-treated base (ATB).
  - 150-mm (6-inch) cement-treated base (CTB).

- Drainage
  - No positive drainage system (STD).
  - Open-graded nonstabilized aggregate layer with longitudinal underdrain system. (Note: A longitudinal drainage system consists of an open-graded trench wrapped with a geotextile material and containing 150-mm (6-inch) flexible longitudinal drainage pipe and rigid pipe outlets at 152-m (500-ft) spacings).
  - Asphalt-treated permeable base (ATPB) with longitudinal underdrain system.
  - ATPB with daylighted drainage into drainage ditch or swale (no longitudinal underdrain system).
  - Cement-treated permeable base (CTPB) with longitudinal underdrain system.
  - CTPB with daylighted drainage into drainage ditch or swale (no longitudinal underdrain system).

- Thickness/Slab Size
  - 250-mm (10-inch) PCC pavement with 4.6-m (15-ft) transverse joint spacing (STD).
  - 200-mm (8-inch) PCC pavement with 3.7-m (12-ft) transverse joint spacing.
  - 300-mm (12-inch) PCC pavement with 5.5-m (18-ft) transverse joint spacing.

- 250-mm (10-inch) JRCP with 150 x 300 mm–MW 80 x MW 35 (6 x 12 inch–W 12 x W 5.5) welded wire fabric mesh placed on steel chairs at midslab depth. Transverse joint spacing is 9.1 m (30 ft) with 32-mm (1.25-inch) diameter epoxy-coated dowel bars that are 450 mm (18 inches) long and spaced at 300 mm (12 inches) across the transverse joints.
- 240-mm (9.5-inch) CRCP with 19-mm (0.75-inch) epoxy-coated deformed bars, 200 mm (8 inches) on-center longitudinally and 900 mm (36 inches) on-center transversely.
- 240-mm (9.5-inch) CRCP with 19-mm (0.75-inch) noncoated deformed bars, 200 mm (8 inches) on-center longitudinally and 900 mm (36 inches) on-center transversely.

- Pavement Cross Section
  - 250-mm (10-inch) thick uniform cross section (STD).
  - Trapezoidal cross section, with the PCC pavement 275 mm (11 inches) thick at the outside edge of the truck lane, tapering to 200 mm (8 inches) thick at the outside edge of the passing lane.
  - Thickened edge design, with the outer edges of both the truck and the passing lane constructed 275 mm (11 inches) thick and tapering to a thickness of 200 mm (8 inches) at the centerline of the pavement.

- Joints/Load Transfer
  - 250-mm (10-inch) JPCP, 4.6-m (15-ft) perpendicular transverse joint spacing, 32-mm (1.25-inch) diameter epoxy-coated dowel bars that are 450 mm (18 inches) long and spaced at 300 mm (12 inches) across the transverse joints (STD).
  - 250-mm (10-inch) JPCP, 4.6-m (15-ft) perpendicular transverse joint spacing, 32-mm (1.25-inch) diameter noncoated dowel bars that are 450 mm (18 inches) long and spaced at 300 mm (12 inches) across the transverse joints (no epoxy on the dowel bars).
  - 250-mm (10-inch) JPCP, 4.6-m (15-ft) skewed transverse joint spacing, 32-mm (1.25-inch) diameter epoxy-coated dowel bars that are 450 mm (18 inches) long and spaced at 300 mm (12 inches) across the transverse joints. The joints are skewed counter-clockwise at 0.6 m (2 ft) per 3.7-m (12-ft) lane.
  - 250-mm (10-inch) JPCP, 4.6-m (15-ft) perpendicular transverse joint spacing, and no dowel bars.
  - 250-mm (10-inch) JPCP, 4.6-m (15-ft) perpendicular transverse joint spacing, with three 32-mm (1.25-inch) diameter epoxy-coated dowel bars that are 450 mm (18 inches) long located in the center of each wheelpath on 300-mm (12-inch) spacings.

- Joint Sealing
  - 4.6-m (15-ft) joint spacing with 3-mm (0.12-inch) control cut, 9.5-mm (0.38-inch) widening cut, backer rod, and hot-poured asphaltic sealant (STD).
  - 4.6-m (15-ft) joint spacing with hot-poured sealant and a single 3-mm (0.12-inch) saw cut for crack control.

- 4.6-m (15-ft) joint spacing with 3-mm (0.12-inch) control cut, 9.5-mm (0.38-inch) widening cut, backer rod, and silicone sealant.
- 4.6-m (15-ft) joint spacing with silicone sealant and a single 3-mm (0.12-inch) saw cut for crack control.
- 4.6-m (15-ft) joint spacing with 3-mm (0.12-inch) control cut, 9.5-mm (0.38-inch) widening cut, and preformed compression sealant.
- 4.6-m (15-ft) joint spacing with no sealant and a single 3-mm (0.12-inch) saw cut for crack control.

- Shoulders
  - 150-mm (6-inch) hot-mix asphalt (HMA) shoulders over 250-mm (10-inch) dense graded aggregate base (STD).
  - 400-mm (16-inch) gravel shoulders.
  - 150-mm (6-inch) partial-depth, tied PCC shoulder over a 250-mm (10-inch) aggregate base (outer lane only, standard HMA shoulder on passing lane).
  - 250-mm (10-inch) full-depth, tied PCC shoulder over a 150-mm (6-inch) aggregate base (outer lane only, standard HMA shoulder on passing lane).
  - *Standard* pavement section with a 0.6-m (2-ft) widened PCC slab and a 2.4-m (8-ft) HMA shoulder.

- Concrete Strength/Materials
  - 4.5-MPa (650 lb/in^2) flexural strength (28-day value tested under third-point loading, American Society for Testing and Materials (ASTM) C78) (STD).
  - 5.2-MPa (750 lb/in^2) flexural strength (28-day value tested under third-point loading (ASTM C78)). Higher strength is obtained by using additional Type I cement.
  - High-early strength, opening to traffic within 48 hours (high-early strength is obtained by using Type III cement).
  - PCC mix using a well-graded aggregate gradation.

- Initial Smoothness (as measured by a California Profilograph with a 5-mm (0.20-inch) blanking band)
  - 110 to 142 mm/km (7 to 9 inches/mi) (STD).
  - 79 to 110 mm/km (5 to 7 inches/mi).
  - 47 to 79 mm/km (3 to 5 inches/mi).
  - 16 to 47 mm/km (1 to 3 inches/mi).
  - Less than 16 mm/km (1 inches/mi).

Clearly, a virtually unlimited number of design feature alternatives could have been selected for this project. However, the number of alternatives was limited to not only facilitate the questionnaire survey process but also to represent more established design practices (although a few unique design features were included to reflect new or innovative practices).

Questionnaire Survey Forms

After the design feature categories were identified and the various design feature alternatives selected, questionnaire surveys were developed: one targeted at highway agencies (to solicit relative performance data) and one targeted at PCC paving contractors (to solicit relative cost data). Although these were separate surveys, the pavement design variables presented in each questionnaire are identical. This allows the results from each data collection effort to be directly paired for analysis. Additional data (including a short summary of the agency's PCC pavement maintenance activities and the ranking of the design categories in terms of their impact on pavement performance) were also a part of the relative performance questionnaire surveys.

Both surveys were structured so that only one design feature (from the *Standard* design) was changed at a time, and the survey participants were then asked to assess what effect that change might have in terms of the relative performance (agency questionnaire) or costs (contractor questionnaire). In this way, the relative effects of the change in that one design feature could be determined. For example, one scenario is to change the base from the 150-mm (6-inch) aggregate base in the *Standard* design to a 150-mm (6-inch) CTB. Considering this single change, the highway agencies were asked to assess what effect this would have on the relative performance of the modified pavement design in comparison to the *Standard* design. Assigning the relative performance of the *Standard* design as 1.0, if the modified pavement (with the CTB) is believed to be capable of carrying 5 percent more 80-kN (18-kip) equivalent single-axle load (ESAL) applications than the *Standard* pavement, the relative performance rating is 1.05. On the other hand, if the design feature change is believed to result in a 5-percent decrease in the number of 80-kN (18-kip) ESAL applications that the pavement can carry, the relative performance rating is 0.95. Again, all performance ratings are made relative to the *Standard* design performance rating of 1.0.

A similar approach is employed for the cost surveys. Using the same example (changing the aggregate base of the *Standard* section to a CTB), PCC paving contractors were asked to estimate what the change in relative cost of the modified pavement design might be, assuming a relative cost of 1.0 for the *Standard* section. If the cost for the modified pavement (containing the CTB) is believed to be 15 percent more than the *Standard* section, the relative cost rating is then 1.15.

Survey respondents provided these relative ratings for the entire group of design feature alternative changes listed above. Respondents were asked to not enter a rating if they had no experience with a particular design feature.

As part of the survey development, several design and construction assumptions were established to provide a common foundation for all respondents and to help to maintain overall consistency in the responses. These assumptions include the following:

- The pavement is assumed to be located on a rural divided highway.
- The average daily traffic (ADT) is assumed to be 20,000 vehicles per day in each direction with 15 percent trucks. This represents approximately 700,000 to 800,000 ESALs per year in the design lane (no growth in annual ESALs is assumed during the life of the pavement).

- The project is located within 80 km (50 mi) of the contractor's mobilization center.
- The project is 8 km (5 mi) long and has four lanes of traffic (two lanes in each direction).
- The project is constructed at grade with no earthwork and all paving is on longitudinal slopes less than 5 percent.
- The pavement is constructed during normal daytime business hours.
- Standard slipform paving equipment will be used for construction.
- A portable PCC batch plant will be utilized.
- The PCC for this project is a typical local mix.
- Local environmental regulations will govern the project.
- Local specifications for construction apply to the highway work.
- Local load restrictions will be used.

Appendix B contains the final questionnaires used in conducting both surveys.

Desired Number of Survey Responses

Prior to sending out the questionnaire surveys, the researchers performed an initial statistical evaluation to determine the desired number of questionnaire responses required to have a reasonable estimate of the relative performance and cost. In determining the desired sample size, it is assumed that the total population has a normal distribution. The purpose of the questionnaires is to predict the average of the population (for example, the average PCC pavement life or the average PCC pavement relative construction cost). Thus the following equation is applicable to estimate the desired sample size:

$$n = \left[\frac{\sigma\, z}{T}\right]^2 \tag{1}$$

where:

n = desired number of survey samples.
σ = estimated standard deviation.
z = number of standard error units (based on the desired confidence level and obtained from a normal probability table).
T = required precision or tolerance.

With this approach, the standard deviation of the to-be-determined average value is yet unknown. Furthermore, the desired confidence levels and the desired precision levels must also be selected. However, by running a range of values with an initially assumed, reasonable average, the effects of these inputs on the resulting number of samples can be determined and reasonable target values can be selected.

For the purposes of estimating the number of samples, the analyses for the performance and cost questionnaires are broken out separately, as described below.

- The relative performance questionnaires essentially ask, "What is the performance life of PCC pavements?" Assuming the average result is approximately 40 years, standard deviations of 4, 6, and 10 years (roughly equivalent to coefficients of variation of 10, 15, and 25 percent) were selected as estimates of the true standard deviation. Next, three levels of desired precision (the desired closeness of the life estimates) were selected for consideration: 2, 5, and 10 years. Based on these assumptions and using equation 1, table 1 was generated.

- The relative cost questionnaires consider the basic question, "What is the cost of constructing a PCC pavement?" Assuming the average result is approximately $35.90/m^2 ($30/yd^2), standard deviations of $3.60, $7.20, and $10.80/m^2 ($3.00, $6.00, and $9.00/yd^2) (roughly equivalent to coefficients of variations of 10, 15, and 20 percent) were selected as estimates of the true standard deviation. Again, three levels of required precision were selected for consideration: $2.40, $4.80, and $7.20/m^2 ($2.00, $4.00, and $6.00/yd^2). Based on these assumptions and again using equation 1, table 2 was produced.

Table 1. Summary of the required number of samples for performance questionnaires.

		Number of Required Samples					
		90% Confidence (z = 1.645)			**95% Confidence (z = 1.960)**		
		Precision of Average Performance Life Estimate (T)			**Precision of Average Performance Life Estimate (T)**		
		Within 2 years	**Within 5 years**	**Within 10 years**	**Within 2 years**	**Within 5 years**	**Within 10 years**
Estimated Standard Deviation of Performance Life Estimates (σ)	**4 years**	10.8	1.7	0.4	15.4	2.5	0.6
	6 years	24.4	3.9	1.0	34.6	5.5	1.4
	10 years	67.7	10.8	2.7	96.0	15.4	3.8

As expected, table 1 shows that for higher levels of precision, higher standard deviation values, and higher confidence levels, greater numbers of relative performance responses are needed. For this project, a 95 percent confidence level and a precision level of 5 years are considered appropriate. Assuming a high variability in the life estimates (10 years), a minimum of 16 surveys (rounded up from the corresponding value of 15.4 in table 1) is desirable for the performance surveys.

Similarly, table 2 shows that for higher levels of precision, higher standard deviation values, and higher confidence levels, more relative cost responses are needed. For this project, a 95 percent confidence level and a precision level of $4.80/m^2 ($4.00/yd^2) are probably appropriate.

Assuming a high variability in the responses ($10.80/m^2 ($9.00/yd^2)), a minimum of 20 surveys (rounded up from the corresponding value of 19.4 in table 2) is desirable for the cost surveys.

Table 2. Summary of the required number of samples for relative cost questionnaires.

		Number of Required Samples					
		90% Confidence (z = 1.645)			95% Confidence (z = 1.96)		
		Precision of Average Cost Estimate (T)			Precision of Average Cost Estimate (T)		
		Within $2.40/m^2 ($2.00/yd^2)	Within $4.80/m^2 ($4.00/yd^2)	Within $7.20/m^2 ($6.00/yd^2)	Within $2.40/m^2 ($2.00/yd^2)	Within $4.80/m^2 ($4.00/yd^2)	Within $7.20/m^2 ($6.00/yd^2)
Estimated Standard Deviation of Cost Estimates (σ)	$3.60/m^2 ($3.00/yd^2)	6.1	1.5	0.7	8.6	2.2	1.0
	$7.20/m^2 ($6.00/yd^2)	24.4	6.1	2.7	34.6	8.6	3.8
	$10.80/m^2 ($9.00/yd^2)	54.8	13.7	6.1	77.8	19.4	8.6

The above analysis is very approximate, but it was used as an initial goal for the number of questionnaires to be obtained for the performance and cost surveys.

SHA Performance Questionnaire Survey

A project summary and request for participation was faxed to 43 SHAs; 25 agencies ultimately agreed to participate. Once appropriate contacts within each agency had been identified, the relative performance questionnaire forms were faxed or mailed to the participants; 45 days were allotted to them to submit completed forms. During this time, typically three deadline reminders were faxed to the volunteers. These reminders were sent during the third and fourth weeks and on the survey completion date.

A total of 12 SHA responses were received, despite several concentrated efforts to increase that number. This falls slightly lower than the original target of 16, but is believed to still be sufficiently high to provide meaningful results. Agencies providing responses to the questionnaire surveys were:

- Florida.
- Illinois.
- Indiana.
- Kansas.
- Minnesota.
- New Jersey.

- Ohio.
- Oklahoma.
- South Dakota.
- Tennessee.
- Washington.
- West Virginia.

A summary of the raw SHA relative performance ratings is provided in appendix C; a concise summary of the responses is shown in table 3. This table shows several different columns of information related to the relative performance ratings. The column titled *Average Survey Results* contains the direct averages of the performance ratings provided by all SHAs. However, because of the considerable variability associated with many of these responses, perceived outlier data points were removed and the resultant performance ratings presented under the column titled *Modified Survey Results*. Outlying data points were identified as those performance values that 1) grossly contradicted the expected performance trends or 2) were greatly different in magnitude from the reasonable performance range.

The column titled *Average Model Results* in table 3 presents the average performance ratings as computed by available performance models; it is used as a basis for checking the reasonableness of the survey results (see *Performance Model Evaluation* section below). Based on the consideration of the survey results and the performance models, the recommended ranges of performance ratings for each design feature category are presented in the column titled *Recommended Values*.

A quick review of the recommended values shown in table 3 indicates that slab thickness has the largest effect on relative pavement performance, as might be expected. Other factors noted to have a major effect on relative pavement performance include the type of base (including the absence of a base course), drainage, dowel bars (the absence of dowel bars considerably decreased performance), and widened slabs.

Performance Model Evaluation

Due to the variability observed in the performance-related survey responses, available PCC pavement performance models were used to develop relative performance ratings for certain design features that may be added to or deleted from the *Standard* pavement section. Unfortunately, not all design features could be evaluated using these models because many features are not direct input variables in the models. Table 4 summarizes the models that were used in this study and the corresponding design features that could be evaluated with each model.

The average performance ratios coming from an investigation of all of the aforementioned models are presented in the *Average Model Results* column of table 3. As previously mentioned, using a combination of the results from the modified performance ratings and the results from the available performance models, recommended ranges for each design feature category were developed and are presented in table 3. The recommended ranges are used to define the default performance changes (i.e., the default performance data set) used in the analytical software tool developed for this project.

Table 3. Summary of performance ratings.

Design Feature Category	Design Feature	Average Survey Results	Modified Survey Results	Average Model Results	Recommended Values	Comments
Subgrade	Untreated prepared subgrade **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	300-mm (12-in) lime treated subgrade (and elimination of aggregate base)	0.86	0.86	0.93	0.85 to 0.95	Base course eliminated.
Base	150-mm (6-in) dense-graded aggregate base on prepared subgrade **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	No base (placed directly on prepared subgrade)	0.76	0.83	0.89	0.70 to 0.90	—
	150-mm (6-in) dense-graded ATB	1.11	1.16	1.25	1.00 to 1.20	—
	150-mm (6-in) dense-graded CTB	0.88	1.09	1.29	1.00 to 1.20	—
Drainage	No drainage layers, no underdrains **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	150-mm (6-in) open-graded, nonstabilized aggregate base (with underdrains)	1.32	1.22	1.19	1.00 to 1.20	High survey performance results not validated with field data. Performance of permeable bases without underdrains not substantiated with field data.
	150-mm (6-in) ATPB (with underdrains)	1.52	1.34	1.33	1.00 to 1.20	
	150-mm (6-in) ATPB (without underdrains)	1.27	1.17	n/a	0.90 to 1.20	
	150-mm (6-in) CTPB (with underdrains)	1.28	1.24	1.33	1.00 to 1.20	
	150-mm (6-in) CTPB (without underdrains)	1.28	1.17	n/a	0.90 to 1.20	
Thickness/ Slab Size	250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing	0.57	0.82	0.44	0.40 to 0.80	—
	300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing	2.05	1.17	13.64	1.20 to 1.80	Models largely consider effects of thickness on fatigue damage.
	250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing (32-mm (1.25-in) epoxy-coated dowels, 150- x 300-mm (6- x 12-in) mesh)	0.91	0.91	0.79	0.80 to 1.00	Long joint spacing detracts from performance in models.
	240-mm (9.5-in) CRCP (19-mm (0.75-in) epoxy-coated deformed bars, 200-mm (8-in) o.c. longitudinal, 914-mm (36-in) o.c. transverse)	1.05	1.05	0.74	0.90 to 1.10	American Association of State Highway and Transportation Officials (AASHTO) models assume same thickness required for all pavement types.
	240-mm (9.5-in) CRCP (19-mm (0.75-in) noncoated deformed bars, 200-mm (8-in) o.c. longitudinal, 914-mm (36-in) o.c. transverse)	1.12	1.12	0.74	0.90 to 1.10	
Cross Section	250-mm (10-in) thick uniform cross section **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	Trapezoidal cross section, 275 to 200 mm (11 to 8 in)	1.00	1.00	n/a	0.95 to 1.05	—
	Thickened edge cross section, 200 mm (8 in) at centerline to 275 mm (11 in)	1.00	1.00	n/a	0.95 to 1.05	—
Joints/Load Transfer	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	250-mm (10-in) JPCP, 32-mm (1.25-in) noncoated dowels, 4.6-m (15-ft) perpendicular joints	0.97	0.97	0.86	0.90 to 1.00	—
	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) skewed joints	0.96	0.96	n/a	0.95 to 1.00	—
	250-mm (10-in) JPCP, no dowels, 4.6-m (15-ft) perpendicular joints	0.59	0.59	0.50	0.50 to 0.60	—
	250-mm (10-in) JPCP, reduced # of 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints	0.85	0.85	n/a	0.80 to 0.95	—

Key: STD = Standard; ATB = asphalt-treated base; CTB = cement-treated base; ATPB = asphalt-treated permeable base; CTPB = cement-treated permeable base; o.c. = on center; n/a = not available.

Table 3. Summary of performance ratings (continued).

Design Feature Category	Design Feature	Average Survey Results	Modified Survey Results	Average Model Results	Recommended Values	Comments
Joint Sealing	Hot-poured rubberized asphalt with widening cut (4.6-m (15-ft) joints) **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	Hot-poured rubberized asphalt without widening cut (4.6-m (15-ft) joints)	0.99	0.99	n/a	0.95 to 1.00	—
	Silicone sealant with widening cut (4.6-m (15-ft) joints)	1.06	1.06	0.64	1.00 to 1.05	Models show reduced performance from silicone sealants.
	Silicone sealant without widening cut (4.6-m (15-ft) joints)	0.98	0.98	n/a	0.95 to 1.05	
	Preformed compression sealant (4.6-m (15-ft) joints)	1.05	1.07	2.97	1.00 to 1.10	Models show extremely high performance from preformed seals.
	No sealant	0.86	0.95	0.56	0.90 to 1.00	—
Shoulders	150-mm (6-in) HMA over 250-mm (10-in) dense graded aggregate base **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	400-mm (16-in) gravel	0.98	0.98	1.00	0.95 to 1.00	—
	150-mm (6-in) partial-depth tied PCC over 250-mm (10-in) aggregate base	1.14	1.14	1.73	1.00 to 1.15	Models show extremely high performance from tied shoulders and widened slabs.
	250-mm (10-in) full-depth tied PCC over 150-mm (6-in) aggregate base	1.30	1.18	1.73	1.00 to 1.30	
	0.6-m (2-ft) widened PCC slab and a 2.4-m (8-ft) HMA shoulder	1.34	1.21	2.74	1.10 to 1.40	
Strength/ Materials	4.5-MPa (650 lb/in^2) flexural strength **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	5.2-MPa (750 lb/in^2) flexural strength	1.17	1.17	4.35	1.00 to 1.20	Models largely consider effects of strength on fatigue damage.
	High-early strength	0.97	0.97	n/a	0.95 to 1.05	—
	Well-graded mix	1.26	1.08	n/a	1.00 to 1.10	—
Initial Smoothness	*Note: These smoothness performance values are assumed to be the same regardless of the base type used. Reported smoothness values assume measurement with a 5-mm (0.20-in) blanking band.*					
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	1.00	1.00	1.00	1.00	*Standard* section.
	79 to 110 mm/km (5 to 7 in/mi)	1.03	1.03	1.04	1.00 to 1.05	—
	47 to 79 mm/km (3 to 5 in/mi)	1.08	1.08	1.07	1.00 to 1.10	—
	16 to 47 mm/km (1 to 3 in/mi)	1.12	1.12	1.10	1.00 to 1.14	—
	< 16 mm/km (1 in/mi)	1.10	1.10	1.13	1.00 to 1.16	—

All ratings are relative to the performance of the *Standard* pavement section.

Key: STD = Standard; n/a = not available.

Table 4. PCC pavement performance models used to evaluate effect of design features.

Model	Performance Indicator	Design Features Evaluated
1993 AASHTO[6]	Serviceability	Base Drainage Slab Thickness Load Transfer Shoulder Type PCC Strength Initial Smoothness
1998 AASHTO[7]	Serviceability	Subgrade Base Slab Thickness/Slab Size Shoulder Type PCC Strength Initial Smoothness
1990 Ripper[8]	Slab Cracking	Slab Thickness/Slab Size PCC Strength Shoulder Type
1997 Ripper[9]	Slab Cracking	Base Type Slab Thickness/Slab Size PCC Strength Shoulder Type
	Joint Faulting	Base Type Drainage Slab Thickness/Slab Size Load Transfer Shoulder Type
	Joint Spalling	Slab Size Joint Sealant

SHA Design Category Ranking Survey

In addition to the relative performance ratings, SHA respondents were asked to rank the relative importance of each design feature category to PCC pavement performance. That is, of the 10 design feature categories (see below) respondents were asked to rank each factor on an integer scale of 1 to 10, with 10 representing the most important and 1 the least important. No two design features were allowed to share the same ranking, so the result was a "forced ranking" of the importance of each design feature category. These rankings were incorporated into the analysis approach as a way of accounting for the effects of multiple design feature changes on pavement performance (see discussion in chapter 3).

The final recommended *category rankings* are presented in table 5 and are based on the collected survey results. It is readily acknowledged that a designer or agency could have different opinions about the ranking factors. Therefore, although the category ranking factor set displayed in table 5 is used as the default ranking factor set in the software, an agency may create their own ranking factor set that reflects their expected relationships between design categories and concrete pavement performance.

Table 5. Details of the recommended category ranking factor set.

Design Category	Ranking Factor	
Joints/Load Transfer	10	**Most Important**
Thickness/Slab Size	9	
Base/Subbase	8	
Drainage	7	
Strength/Materials	6	
Subgrade	5	
Initial Smoothness	4	
Joint Sealing	3	
Cross Section	2	
Shoulders	1	**Least Important**

Contractor Cost Questionnaire Survey

A project summary and request for participation was faxed to 216 contracting companies provided by the American Concrete Pavement Association (ACPA). A total of 53 firms responded to the request; 38 volunteered to participate in the study. The relative cost questionnaire forms were faxed or mailed to each volunteer. Forty-five days were allotted to complete the questionnaire surveys. As with the relative performance questionnaire surveys, typically three deadline reminders were faxed to the volunteers. These reminders were sent during the third and fourth weeks and on the survey completion date.

Sixteen responses were received from contractors. As with the SHA surveys, this value falls slightly lower than the original target of 20, but it is believed to be sufficiently large to provide meaningful results. Paving contractors responding were:

- Baker Concrete Construction, Monroe, OH.
- Berns Construction Company, Indianapolis, IN.
- Cedar Valley Corporation, Waterloo, IA.
- Duit Construction Company, Edmond, OK.
- Flynn Company, Dubuque IA.

- Gale Tschuor Company, Muncie, IN.
- The Harper Company, Hebron, KY.
- Irving F. Jenson Company, Sioux City, IA.
- K-Five Construction, Lemont, IL (estimator A).
- K-Five Construction, Lemont, IL (estimator B).
- K-Five Construction, Lemont, IL (estimator C).
- Koss Construction, Topeka, KS.
- The Sundt Companies, Tucson, AZ.
- Weaver-Bailey Contractors, El Paso, AR.
- Western Plains Construction, Lincoln, NE.
- Wittwer Paving Inc., Wichita, KS.

A summary of the raw relative cost data collection responses is provided in appendix C. The averages of all survey results for each alternative design feature are displayed in table 6. An initial review of the data indicates that comprehensive drainage systems result in a substantial increase in pavement costs. Other design features that contribute significantly to pavement costs are PCC shoulders and high-early strength PCC mixtures. On the other hand, factors such as pavement cross sections (trapezoid or thickened edge), widened slabs, and joint sealing appear to have very little effect on pavement construction costs. Note that the cost associated with achieving different levels of initial smoothness is dependent on the selected base type.

Summary

This chapter summarizes the data collection activities for this project. A description of the literature search is first presented, which provided many useful resource documents that served as the basis for the development of the questionnaire surveys. The approach used in developing the questionnaire surveys is also provided, which produced two separate questionnaire surveys: one targeted to state highway agencies and intended to collect relative performance ratings for changes in design features, and one targeted to PCC paving contractors and intended to collect relative cost ratings for changes in design features. Each questionnaire survey was structured in the same manner so that the performance and cost data could be matched up for each design feature.

The results of the performance and cost surveys are summarized in this chapter, with raw survey results presented in appendix C. The performance ratings that were received from the highway agencies were evaluated in conjunction with available performance models to check their validity and reasonableness. Overall, these data serve as one "data set" for use in the computer program developed under this project (see chapter 3).

Table 6. Summary of contractor initial cost ratings.

Design Feature Category	Design Feature	Average Survey Results
Subgrade	Untreated prepared subgrade (STD)	1.00
	300-mm (12-in) lime treated subgrade (and elimination of aggregate base)	0.98
Base	150-mm (6-in) dense-graded aggregate base on prepared subgrade (STD)	1.00
	No base (placed directly on prepared subgrade)	0.85
	150-mm (6-in) dense-graded ATB	1.16
	150-mm (6-in) dense-graded CTB	1.06
Drainage	No drainage layers, no underdrains (STD)	1.00
	150-mm (6-in) open-graded, nonstabilized aggregate base (with underdrains)	1.22
	150-mm (6-in) ATPB (with underdrains)	1.33
	150-mm (6-in) ATPB (without underdrains)	1.28
	150-mm (6-in) CTPB (with underdrains)	1.28
	150-mm (6-in) CTPB (without underdrains)	1.27
Thickness/ Slab Size	250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing (STD)	1.00
	200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing	0.87
	300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing	1.09
	250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing (32-mm (1.25-in) epoxy-coated dowels, 150- x 300-mm (6- x 12-in) mesh)	1.08
	240-mm (9.5-in) CRCP (19-mm (0.75-in) epoxy-coated deformed bars, 200-mm (8-in) o.c. longitudinal, 914-mm (36-in) o.c. transverse)	1.20
	240-mm (9.5-in) CRCP (19-mm (0.75-in) non-coated deformed bars, 200-mm (8-in) o.c. longitudinal, 914-mm (36-in) o.c. transverse)	1.15
Cross Section	250-mm (10-in) thick uniform cross section (STD)	1.00
	Trapezoidal cross section, 275 to 200 mm (11 to 8 in)	0.99
	Thickened edge cross section, 200 mm (8 in) at centerline to 275 mm (11 in) edges	1.00
Joints/Load Transfer	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints (STD)	1.00
	250-mm (10-in) JPCP, 32-mm (1.25-in) noncoated dowels, 4.6-m (15-ft) perpendicular joints	0.99
	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) skewed joints	1.01
	250-mm (10-in) JPCP, no dowels, 4.6-m (15-ft) perpendicular joints	0.94
	250-mm (10-in) JPCP, reduced # of 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints	0.97
Joint Sealing	Hot-poured rubberized asphalt with widening cut (4.6-m (15-ft) joints) (STD)	1.00
	Hot-poured rubberized asphalt without widening cut (4.6-m (15-ft) joints)	0.99
	Silicone sealant with widening cut (4.6-m (15-ft) joints)	1.03
	Silicone sealant without widening cut (4.6-m (15-ft) joints)	1.02
	Preformed compression sealant (4.6-m (15-ft) joints)	1.07
	No sealant	0.95
Shoulders	150-mm (6-in) HMA over 250-mm (10-in) dense graded aggregate base (STD)	1.00
	400-mm (16-in) gravel	0.90
	150-mm (6-in) partial-depth tied PCC over 250-mm (10-in) aggregate base	1.13
	250-mm (10-in) full-depth tied PCC over 150-mm (6-in) aggregate base	1.17
	0.6-m (2-ft) widened PCC slab and a 2.4-m (8-ft) HMA shoulder	1.07
Strength/ Materials	4.5-MPa (650 lb/in^2) flexural strength (STD)	1.00
	5.2-MPa (750 lb/in^2) flexural strength	1.05
	High-early strength	1.12
	Well-graded mix	1.01

Key: STD = Standard; ATB = asphalt-treated base; CTB = cement-treated base; ATPB = asphalt-treated permeable base; CTPB = cement-treated permeable base; o.c. = on center.

Table 6. Summary of contractor initial cost ratings (continued).

Design Feature Category	Design Feature	Average Survey Results
Note: these smoothness performance values differ based on the base type used. Reported smoothness values assume measurement with a 5-mm (0.20-in) blanking band.		
Initial Smoothness (measured with a 5-mm (0.20-in) blanking band)	150-mm (6-in) dense-graded aggregate base on prepared subgrade **(STD)**	—
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	1.00
	79 to 110 mm/km (5 to 7 in/mi)	1.00
	47 to 79 mm/km (3 to 5 in/mi)	1.00
	16 to 47 mm/km (1 to 3 in/mi)	1.02
	< 16 mm/km (1 in/mi)	1.04
	No base (placed directly on prepared subgrade)	—
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	Base type not included in survey. Assumed to be similar to costs associated with the 150-mm dense-graded aggregate base.
	79 to 110 mm/km (5 to 7 in/mi)	
	47 to 79 mm/km (3 to 5 in/mi)	
	16 to 47 mm/km (1 to 3 in/mi)	
	< 16 mm/km (1 in/mi)	
	150-mm (6-in) ATB	—
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	1.03
	79 to 110 mm/km (5 to 7 in/mi)	1.04
	47 to 79 mm/km (3 to 5 in/mi)	1.04
	16 to 47 mm/km (1 to 3 in/mi)	1.05
	< 16 mm/km (1 in/mi)	1.07
	150-mm (6-in) CTB	—
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	1.01
	79 to 110 mm/km (5 to 7 in/mi)	1.01
	47 to 79 mm/km (3 to 5 in/mi)	1.01
	16 to 47 mm/km (1 to 3 in/mi)	1.03
	< 16 mm/km (1 in/mi)	1.04
	150-mm (6-in) open-graded, nonstabilized aggregate base	—
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	Base type not included in survey. Assumed to have higher costs than any of the other included base types due to the difficulty of constructing on this base type.
	79 to 110 mm/km (5 to 7 in/mi)	
	47 to 79 mm/km (3 to 5 in/mi)	
	16 to 47 mm/km (1 to 3 in/mi)	
	< 16 mm/km (1 in/mi)	
	Open-graded stabilized drainage layers	—
	110 to 142 mm/km (7 to 9 in/mi) **(STD)**	1.03
	79 to 110 mm/km (5 to 7 in/mi)	1.03
	47 to 79 mm/km (3 to 5 in/mi)	1.03
	16 to 47 mm/km (1 to 3 in/mi)	1.05
	< 16 mm/km (1 in/mi)	1.06

All ratings are relative to the cost of the *Standard* pavement section.

Key: STD = Standard; ATB = asphalt-treated base; CTB = cement-treated base.

CHAPTER 3. ANALYSIS METHOD

Introduction

As described in chapter 2, the analysis approach used in this project is a simple method to investigate the cost and performance implications of changing design features in a PCC pavement. The primary purpose of the analysis method is to compare two different defined pavement sections (i.e., two sections with different design features) to determine the relative differences in their expected costs and expected performance (in terms of allowable ESALs). Although the calculations used in the methodology are simple and straightforward, many components are involved in the overall process. This chapter defines the basic components of the methodology, provides illustrative examples demonstrating the computational methods of the approach, and introduces the analytical software that automates the methodology.

Components of the Analysis Approach

Before discussing the detailed performance and cost computations, it is important to first explain the general components making up the approach. Specifically, this section discusses the definition of pavement sections (and in particular the *Standard* pavement section), cost and performance data sets, category ranking factors, and the simplified life-cycle cost analysis (LCCA) included as part of an analysis session.

Definition of a Pavement Section

A pavement section is defined as a unique combination of specific pavement features chosen from the following 10 different design feature categories:

- Subgrade.
- Base/Subbase.
- Drainage.
- Thickness/Slab Size.
- Cross Section.
- Joints/Load Transfer.
- Joint Sealing.
- Shoulders.
- Strength/Materials.
- Initial Smoothness.

For practical purposes, the number of available design feature options had to be limited to those alternative design features (organized by design category) previously outlined in the *Design Categories and Alternative Design Features* section of chapter 2.

The *Standard* Pavement Section

Because the goal of the survey of user agencies and contractors was to estimate the expected changes in cost and performance associated with making one design feature change at a time, a *Standard* pavement design was defined to serve as a baseline. The unique combination of design features for the *Standard* section are listed in table 7. This defined *Standard* pavement section is expected to carry approximately 700,000 to 800,000 ESALs per year over an assumed 20-year design life (design life is defined as the time until first major rehabilitation).

Table 7. Pavement design features defining the *Standard* pavement section.

Design Category	Design Features
Subgrade	Untreated prepared subgrade
Base/Subbase	150-mm (6-in) dense-graded aggregate base
Drainage	No drainage layers, no underdrains
Thickness/Slab Size	250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing
Cross Section	250-mm (10-in) uniform thickness
Joints/Load Transfer	32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints
Joint Sealing	Hot-poured asphalt with widening cut
Shoulders	150-mm (6-in) HMA over 250-mm (10-in) dense graded aggregate base
Strength/Materials	4.5-MPa (650 psi) flexural
Initial Smoothness	110 to 142 mm/km (7 to 9 in/mi) (measured with a 5-mm (0.2-in) blanking band)
Note: ADT is 20,000 vehicles per day in each direction with 15% trucks. This is approximately 700,000 to 800,000 ESALs per year in the design lane. Assume no growth in annual ESALs during the life of the pavement.	

As part of the survey, the respondents were asked to use the expected cost and performance of the *Standard* section as a basis while they systematically replaced one of the *Standard* section's features with one of the other available alternative design features. With every feature replacement, respondents were asked to estimate expected percent changes in terms of cost and performance ratios. That is, the cost and performance ratios for the Standard pavement section were assumed to be 1.0. If a particular design feature change was expected to result in a 5-percent increase in cost and a 2-percent decrease in performance, then the survey respondent would have reported cost and performance ratios of 1.05 and 0.98, respectively.

<u>Cost and Performance Data</u>

The most basic part of the methodology is based on estimating the total change in cost and performance associated with changing one or more design features from the *Standard* pavement section. Within the analysis tool, these relative percent changes in cost and performance are summarized into cost and performance data sets. Each data set is defined as a summary of the relative percent changes in cost or performance associated with all available design feature values in each of the 10 design categories (i.e., all of the available design features outlined in chapter 2). The following sections contain specific discussions of how the summarized cost and performance data are used to estimate an overall section cost and performance within the analysis procedure.

Relative Cost Data

When multiple design features are changed from the *Standard* section, the percent change in costs of each feature can be simply summed to determine the cost of the modified section (i.e., cost changes are cumulative). This procedure is believed to be valid because most of the costs of additional design features are independent construction variables. For example, assume that a pavement section differs from the *Standard* pavement section in that the 150-mm (6-inch) dense-graded aggregate base is replaced with a 150-mm (6-inch) cement-treated base, and the hot-poured asphalt with widening cut joint seal is replaced with a preformed compression sealant. Next, assume that the base type and joint sealant changes are expected to result in 6.0 and 7.0 percent respective cost increases. The construction of these two design features is completely independent, so these costs can be directly added for a total expected cost increase of 13.0 percent for this pavement section.

Relative Performance Data

Unlike cost data, the impact of individual design features on performance is not necessarily cumulative because most, if not all, design variables are interdependent. This means that performance impacts are also different depending on what other design features are present in the pavement section. For example, tied PCC shoulders might have a larger impact on performance if the pavement is constructed with an aggregate base rather than a stabilized base. Similarly, the effect of a permeable base may be greater if the joints are not doweled. To address this challenge, the underlying methodology relies on a simple method to calculate the impact of performance when multiple design features are changed simultaneously. This method uses a ranking system that rates the design features based on impact and importance. These impact rankings, referred to as *Category Ranking Factors*, are discussed in greater detail in the next section.

<u>Category Ranking Factors</u>

One of the most difficult steps in the analysis approach is the assignment of category ranking factors that accurately reflect an agency's assessment of which design feature categories have the largest impact on overall performance. Because the assignment of realistic ranking factors is imperative for computing realistic overall performance values, a detailed discussion illustrating the importance of these ranking factors is helpful to explain the meaning of ranking factors, illustrate their use, and identify possible pitfalls to avoid.

Background

For those cases when multiple design features change at the same time, a method is needed to estimate the overall expected performance change. Historically, many different methods of different complexity levels have been used to make this performance estimate. By far the most complex approach is to conduct extensive research to develop mechanistic performance models as functions of the changing design features. While this most complex approach would likely provide the most accurate results, it is obviously beyond the scope of this project. More simple methods that have been used include estimating overall performance as the product, sum, average, or weighted average of the different individual performance changes associated with each changing design feature.

For this project it was decided to adopt a method in which the relative importance of the 10 different design categories would be used to weight each individual performance change. These relative weighting factors are referred to as category ranking factors. More specifically, ranking factors are used to determine how much of each individual performance value is added to the overall modified performance.

Obtaining more accurate estimates of overall performance would have required including the entire spectrum of design feature combinations in the questionnaire surveys, or the use of reliable performance prediction models that incorporate every design feature. Because these approaches were not possible under this project, the described approach was adopted as a simple and expedient measure of estimating overall performance.

Practical Range for Overall Performance

Before discussing the detailed computations associated with applying category ranking factors, it is helpful to conduct a reasonableness check by estimating the practical range for the overall expected change in performance. For the category ranking factor method adopted in this analysis approach, the following may be used to define practical limits that establish a range of reasonable values:

- If all individual percent changes in performance are positive, the overall modified value for all practical purposes lies between the smallest individual value and the sum of all individual values.
- If all individual percent changes in performance are negative, the overall modified value most likely lies between the greatest individual value (i.e., that closest to zero) and the sum of all individual values.
- If the individual percent changes in performance contain both positive and negative values, the overall modified value most likely lies between the sum of the negative performance ratings on the lower end, and the sum of the positive performance ratings on the upper end.

To demonstrate these practical limits with an example, assume that relative performance values of +8.0 percent, –1.0 percent, and +15.0 percent are associated with changes in *Subgrade*, *Base/Subbase*, and *Drainage*, respectively. Because this example contains both positive and negative performance changes, the third practical limit situation applies. Therefore, the modified performance value should be expected to be between –1.0 and +23.0 percent. The application of category ranking factors will always result in a modified performance value that lies within these established practical limits.

One might argue that the largest individual performance change would define the minimum expected overall performance for every analysis case. For example, if individual performance values of +6.0 percent, +2.0 percent, and +12.0 percent are associated with changes in *Subgrade*, *Base/Subbase*, and *Drainage*, one might conclude that the overall performance would be between +12.0 percent (the largest individual value) and +20.0 percent (the sum of all values). While this approach may hold true for some cases, without understanding all of the interdependencies among individual design features, it is more conservative to describe the practical limits as defined in the bulleted list above.

Category Ranking Factors Sets

A category ranking factor set is a prioritized list of the 10 different design categories; it reflects their relative importance on the overall performance of a PCC pavement. A ranking factor set is defined by first sorting all 10 design feature categories in order of decreasing importance. Next, integer values from 10 to 1 are assigned to the sorted category list (10 is assigned to the category deemed most important and 1 to the least important category). For the analysis procedure to work as intended, no two design categories are allowed to be assigned the same integer ranking value. To form a basis for this process, SHA survey respondents were asked to submit their assumed category ranking set along with their complete performance data set. The survey results were then used to prepare the default category ranking factor set presented in table 5. It is emphasized that the resulting overall performance change predicted using category ranking factors is merely an estimate of the true performance change that would be realized.

Application of Category Ranking Factors

Category ranking factors are defined as relative because for a given investigation, normalized ranking multipliers are computed by dividing each individual ranking factor by the largest of the included ranking factors (i.e., for only those design features changing from the *Standard* pavement section). This method can be demonstrated by using the example displayed in table 8, where relative performance values of +1.0 percent, +7.0 percent, and +10.0 percent are associated with changes in *Subgrade*, *Base/Subbase*, and *Drainage*. Next, assume individual ranking factors of 5, 8, and 7 are associated with the three respective design feature categories. The largest impact factor of the three included feature categories is the "8" associated with *Base/Subbase*. Therefore, all three of the included impact factors are divided by "8" to compute normalized ranking multipliers. These normalized ranking multipliers are then multiplied by the associated expected relative performance values to give a modified performance value for each design category. The overall section performance is then determined as the sum of all modified performance values. For this example, the expected increase in performance is estimated to be 16.4 percent.

Table 8. Example of use of category ranking factors to determine an overall modified performance rating.

Design Feature Category	**Expected Relative Performance (%)**	**Category Ranking Factor**	**Normalized Ranking Multiplier**	**Modified Performance (%)**
Subgrade	+1.0	5	(5/8) = 0.625	+0.6
Base/Subbase	+7.0	8	(8/8) = 1.00	+7.0
Drainage	+10.0	7	(7/8) = 0.875	+8.8
TOTAL				**+16.4**

In this example, because *Base/Subbase* is assigned the largest ranking factor, by definition its whole associated individual performance is added to the overall modified performance value (i.e., its individual performance change is deemed the most important of the three associated individual performance values). That is, the +7.0 percent expected relative performance associated with *Base/Subbase* is the starting point for the overall performance computations. Next, the individual performance changes associated with the other included design feature categories (*Subgrade* and *Drainage*) are, therefore, diminished by dividing each associated ranking factor by the largest included ranking factor.

It is important to note that within this methodology, the relative differences between ranking factors (rather than the actual ranking factor values) are important when determining overall modified performance. For example, one might think that the design feature assigned an impact factor of 10 is always going to be important when determining the overall pavement section performance. The previous example shows that this is not the case, as none of the three changing design feature categories had a category ranking factor of 10. Normalizing all individual ranking factors to the largest of the included factors ensures that the performance of the most important included design feature becomes the starting point of the modified performance computation. In the example, it is noted that if *Base/Subbase* were the only design feature category that was changing, then the total modified performance would be +7.0 percent. Therefore, the other design features deemed less important are, in a sense, used to adjust the +7.0-percent value associated with *Base/Subbase*. The normalized ranking multipliers give an indication of the relative impact of the adjustments.

Selection of Appropriate Ranking Factors

As mentioned previously, the assignment of realistic category ranking factors is one of the most important and challenging steps of the analysis approach. As with any methodology that has built-in functionality, it is possible to misuse this method and achieve results that are counter-intuitive. This section is intended to provide guidance on selecting ranking factors so as to avoid such pitfalls.

The first recommendation is that it is very important to assign category ranking factors that do not contradict the performance values observed within different design categories. That is, those design categories where the largest percent increases or decreases in performance are observed should most likely be the design categories with the largest category ranking factors. For example, assume that investigated *Thickness/Slab Size* choices result in individual performance changes from –40.0 to +50.0 percent, while different *Joint Sealing* choices result in a range of individual performance between –5.0 and +5.0 percent. For this case, the category ranking factor assigned to *Thickness/Slab Size* should be significantly larger than that assigned to the *Joint Sealing*. To illustrate this point, the two examples (although extreme) shown in tables 9 and 10 illustrate how important it is to assign category ranking factors that reflect the largest impacts on overall performance.

Table 9. Example showing the matching of category ranking factors and performance.

Design Feature Category	Expected Relative Performance (%)	Category Ranking Factor	Normalized Ranking Multiplier	Modified Performance (%)
Thickness/ Slab Size	+50.0	10	(10/10) = 1.0	+50.0
Joint Sealing	–5.0	1	(1/10) = 0.1	–5.0
TOTAL				**+45.0**

Table 10. Example showing a contradiction in the matching of category ranking factors and performance.

Design Feature Category	Expected Relative Performance (%)	Category Ranking Factor	Normalized Ranking Multiplier	Modified Performance (%)
Thickness/ Slab Size	+50.0	1	(1/10) = 0.1	+5.0
Joint Sealing	–5.0	10	(10/10) = 1.0	–5.0
TOTAL				**+0.0**

Note that in table 9 the category ranking factors are intuitively assigned to reflect the relative importance of the two design feature categories; the total modified performance value appears reasonable at +45 percent. Table 10 illustrates the same example but with the category ranking factors reversed. For this case, the overall modified performance is computed to be 0.0 percent, which appears to be a counterintuitive value as one would expect the larger percent performance change associated with the *Thickness/Slab Size* to control. While the analysis method and associated software will allow the user to set these category ranking factors to any number, it is very important to understand how they will be used in the computations.

It is equally important to remember that the modified performance values resulting from the application of ranking factors are additive. The individual performance associated with the largest ranking factor is used as the starting point (e.g., 7.0 percent was used as the starting point of the overall performance computation in table 8). This is because the category with the largest associated ranking factor is assumed to have the largest influence on overall performance. That is, in many cases, one should expect the overall performance to be close to the one individual performance value associated with the largest ranking factor as it, by definition, is the governing performance value. As mentioned previously, the ranking factors associated with other included design features are used to diminish those associated individual performance changes before adding them to the overall performance calculation. That is, all design feature categories that are not deemed to be the most important category (i.e., their ranking factors are less than the largest included ranking factor) are simply used to adjust the individual performance change associated with the largest ranking factor. It is the defined ranking factors that are used to determine the ranking factor ratios (normalized ranking multipliers) that determine the how much of each individual performance change is added to the overall performance value.

Another way to assess the relative meaning of these impact factors is to compare the included values directly. For instance, in the example presented in table 8, based on the entered *Subgrade*, *Base/Subbase*, and *Drainage* ranking factors of 5, 8, and 7, respectively, one can deduce that the user assumed the following:

- The individual performance value associated with *Subgrade* is 5/8 as important as the individual performance associated with *Base/Subbase*.
- The individual performance value associated with *Drainage* is 7/8 as important as the individual performance associated with *Base/Subbase*.
- The individual performance value associated with *Subgrade* is 5/7 as important as the individual performance associated with *Drainage*.

An earlier section of this chapter (Demonstration of the Analysis Method) illustrates how ranking factors are used in the computations. This should help emphasize the importance of selecting appropriate impact factors that correspond with the assumed impact on performance associated with each design category. The selection of user-defined category ranking factor sets should be carefully considered so that appropriate values reflective of observed performance are developed.

Simplistic Life-Cycle Cost Analysis

Because design feature changes alter the expected performance (estimated service life) of a given pavement section, the associated life-cycle cost (LCC) stream is also affected. To investigate the magnitude of the impact of design feature changes on LCCs, the analytical tool does provide a means of conducting a simplistic LCCA as part of the analysis. However, because of its simplistic nature, the user of the software tool is warned that the results of the LCCA results should be viewed with caution. While the cost trends may be realistic, the actual computed dollar values may or may not be accurate. If more accurate LCCA results are desired, it is recommended that a more rigorous LCCA be conducted using established methods.[10]

The LCCA conducted within this software is described as simplistic in that the cost stream values (annual maintenance, rehabilitation, and salvage value costs) can all be determined using simplified methods. Specifically, the following important components are included in the simplistic LCC approach:

- *Standard* Pavement Section Design Life—The expected design life (in years) of the defined *Standard* pavement section. Based on the design features used to define the *Standard* pavement section, this design life is assumed to be 20 years (note: this 20-year design life is hard coded in the software). Other pavement section design lives are computed by multiplying this 20-year design life by the expected performance ratio computed by using the analysis methodology. For example, if a custom section was found to have an overall modified expected performance of +7.0 percent, the expected service life of the custom section is computed as 1.07 * 20 years = 21.4 years.

- Initial Construction Cost for the *Standard* Pavement Section—Fence-line-to-fence-line pavement-related cost expressed in dollars per lane-mile.
- Analysis Period—Period of time over which future maintenance and rehabilitation costs are to be considered in the LCCA. Based on FHWA recommendations, the analysis period must be a minimum of 35 years.
- Discount Rate—The discount rate, entered as a percentage, is the estimated difference between the interest and inflation rates over a long time period. It is used to translate actual LCCs into equivalent present worth costs. Historically, the discount rate has been in the range of 3 to 5 percent.
- Salvage Value—The user has the choice whether or not to include a salvage value in the LCCA. If it is included, a linear deprecation of the rehabilitation cost is used to determine the salvage value at the end of the analysis period. For example, if a rehabilitation with an expected life of 10 years and a cost of $50,000 is applied 3 years before the end of the analysis period, the salvage value would be computed as $50,000 * (10 – 3)/10 = $35,000. The salvage value is essentially a negative cost applied at the end of the analysis period.
- Time Until First Rehabilitation—The time until first rehabilitation may either be defined as a percentage of the expected design life or as an actual pavement age (in years). This input is used to determine the time at which the rehabilitation cost is to be included in the LCCA. For example, if the design life is 20 years, and the time until first rehabilitation is entered as 80 percent, then the rehabilitation cost included in the LCC stream would be applied at a time of 20 years * 0.8 = 16 years.
- Assumed Rehabilitation Life—The assumed life (in years) of the chosen rehabilitation activity. As subsequent rehabilitation activities are assumed to be applied until the end of the analysis period, this LCC input determines the years in which those subsequent rehabilitation activities will be applied.
- Cost of Annual Maintenance—The cost of annual maintenance may either be entered as a percentage of the provided initial construction cost or as an actual cost. This input is used to determine the maintenance cost applied annually within the LCCA. For example, if the initial construction cost is $300,000 per lane-mile, and the cost of annual maintenance input is set to 5 percent, then the annual maintenance costs included in the LCC stream are computed as 0.05 * $300,000 = $15,000/lane-mile per year.
- Rehabilitation Cost—The cost of rehabilitation may either be entered as a percentage of the provided initial construction cost or as an actual cost. This is used to determine the cost of the rehabilitation activity that is to be applied at regular intervals (after first application) until the end of the analysis period.

It should be noted that all annual maintenance and rehabilitation-related LCCA inputs are section-specific within the analysis approach. That is, all of these LCC inputs may be customized for each unique pavement section that is defined. The primary purpose of linking these cost inputs to a section is to accommodate the many cases where the inclusion of a design feature directly influences the future maintenance and rehabilitation costs associated with that section (e.g., including edge drains will result in the additional cost of cleaning the edge drains).

The LCCA for a given pavement section is dependent on the expected cost and performance ratios computed for that section. Specifically, as a first step, these ratios are used to compute the expected initial cost (in actual dollars) and expected design life (in actual years) used to compile an associated cost stream. As an example, assume that cost and performance ratios of 1.07 and 1.05, respectively, are computed for a given pavement section. In addition, assume a design life of 20 years and an initial construction cost of $500,000 for the *Standard* pavement section. Therefore, the custom pavement section is assumed to have an expected service life of 20 years * 1.05 = 21 years, and an initial construction cost of $500,000 * 1.07 = $535,000. These values are used as the basis of the LCCA associated with the custom pavement section. For a given pavement section, the primary outputs of the simplified LCCA are the individual computed costs (annual maintenance, rehabilitation, and salvage value), their total present worth values, the summarized equivalent uniform annual cost (EUAC), and ratios of the EUACs between the different sections being compared.

Demonstration of the Analysis Method

As previously discussed, the underlying analysis method, in its most basic form, is based on making cost and performance comparisons between a user-defined pavement section and the *Standard* pavement section. This underlying methodology can be expanded so that two user-defined sections can be compared to each other after each has been individually compared to the *Standard* pavement section. The computation methodology used to make such comparisons is described below.

<u>Comparing a Custom Pavement Section to the *Standard* Pavement Section</u>

The following example illustrates the computations involved when comparing a user-defined (modified) pavement section to the *Standard* section. In this example, the modified pavement section is defined by changing three different design features from the *Standard* section. The specific feature changes are summarized in table 11. The expected cost and performance changes and applicable category rankings for the example are defined in table 12.

Table 11. Design features that differ in the current example.

Design Category	***Standard* Pavement Section Design Features**	***Modified* Pavement Section Design Features**
Base/Subbase	150-mm (6-in) dense-graded aggregate base	150-mm (6-in) dense-graded asphalt-treated base
Joints/Load Transfer	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints	250-mm (10-in) JPCP, 32-mm (1.25-in) uncoated dowels, 4.6-m (15-ft) perpendicular joints
Joint Sealing	Hot-poured asphalt with widening cut	Preformed compression sealant

As mentioned previously, the costs associated with different design feature changes are cumulative. Therefore, for this example (table 12), the change in cost is computed to be +16.0 – 1.0 + 7.0 = +22.0 percent (i.e., the modified section is expected to cost 22.0 percent more than the *Standard* pavement section).

Table 12. Expected percent changes in cost and performance associated with the changed design features.

Design Category	*Modified* Pavement Section Design Features	Expected Change in Cost (%)	Expected Change in Performance (%)	Category Ranking Factor
Base/Subbase	150-mm (6-in) dense-graded asphalt-treated base	+16.0	+5.0	8
Joints/Load Transfer	250-mm (10-in) JPCP, 32-mm (1.25-in) uncoated dowels, 4.6-m (15-ft) perpendicular joints	–1.0	–5.0	10
Joint Sealing	Preformed compression sealant	+7.0	+5.0	3

Because the performance changes include both positive and negative values, the practical range of performance is estimated to be between the sum of the negative values on the lower end and the sum of the positive values on the upper end. Therefore, for this example the overall performance change is expected to be between –5.0 percent (the one negative value) and +10.0 percent (the sum of the two positive values). The category ranking factors are used to estimate the weighted contributions of each individual performance rating. The details of this computation process are illustrated in table 13.

Table 13. Cost and performance computation example.

Design Feature Category	*Standard* Pavement Section Design Features	*Modified* Pavement Section Design Features	Expected Relative Cost (%)	Expected Relative Performance (%)	Category Ranking Factor	Normalized Ranking Multiplier	Modified Performance (%)
Base/ Subbase	150-mm (6-in) dense-graded aggregate base	150-mm (6-in) dense-graded asphalt-treated base	+16.0	+5.0	8	(8/10) = 0.80	+4.0
Joints/ Load Transfer	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints	250-mm (10-in) JPCP, 32-mm (1.25-in) uncoated dowels, 4.6-m (15-ft) perpendicular joints	–1.0	–5.0	10	(10/10) = 1.00	–5.0
Joint Sealing	Hot-poured asphalt with widening cut	Preformed compression sealant	+7.0	+5.0	3	(3/10) = 0.30	+1.5
TOTAL			**+22.0**				**+0.5**

Normalized ranking factor multipliers are determined for each changing design feature category by dividing each individual category ranking factor by the largest of the included category ranking factor value. For this example, the largest included category ranking factor is the 10 associated with the *Joints/Load Transfer* design feature category. Modified performance values are computed by multiplying the individual expected relative performance values by the computed normalized ranking multipliers. Summing the individual modified performance ratings for this example gives a total estimated increase in performance of 0.5 percent.

Comparison of Two Custom Pavement Sections (Section A vs. Section B)

Although the previous example compared a defined custom pavement section to the *Standard* pavement section, the methodology also allows the user to compare one custom section to another (e.g., *Section A* to *Section B*). The first step of this process is to make the following two independent comparisons: 1) compare *Section A* to the *Standard* pavement section and obtain expected percent cost and performance changes, and 2) compare *Section B* to the *Standard* pavement section and obtain expected percent cost and performance changes. The final step of the process is to compare the individual results so that the expected cost and performance of *Section B* can be reported in terms of the expected *Section A* cost and performance. The following example demonstrates this process.

Step 1: Compare Section A to the Standard Pavement Section

Assume that the goal of this example is to compare the two pavement sections summarized in table 14 (note that the shaded cells in table 14 indicate that those design features do not differ from the *Standard* pavement section). As indicated, the first step is to compare each individual pavement section to the *Standard* pavement section. Noting that *Section A* is the pavement section used in the previous example, its analysis details were those summarized in table 13. Therefore, comparing *Section A* to the *Standard* pavement section found that the expected cost and performance change expected with *Section A* are the following:

- Expected relative cost change: +22.0 percent.
- Expected relative performance change: +0.5 percent.

Table 14. Summary of two custom sections being compared (*Section A* and *Section B*).

Design Category	***Standard* Pavement Section Design Features**	***Section A* Design Features**	***Section B* Design Features**
Subgrade	Untreated prepared subgrade	Same as *Standard* section	Same as *Standard* section
Base/Subbase	150-mm (6-in) dense-graded aggregate base	150-mm (6-in) dense-graded asphalt-treated base	150-mm (6-in) dense-graded cement-treated base
Drainage	No drainage layers, no underdrains	Same as *Standard* section	Same as *Standard* section
Thickness/Slab Size	250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing	Same as *Standard* section	Same as *Standard* section
Cross Section	250-mm (10-in) uniform thickness	Same as *Standard* section	Same as *Standard* section
Joints/Load Transfer	250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints	250-mm (10-in) JPCP, 32-mm (1.25-in) dowels w/o epoxy, 4.6-m (15-ft) perpendicular joints	Same as *Standard* section
Joint Sealing	Hot-poured asphalt w/widening cut (4.6-m (15-ft) joint spacing)	Preformed compression sealant	Same as *Standard* section
Shoulders	150-mm (6-in) HMA over 250-mm (10-in) dense graded aggregate base	Same as *Standard* section	Same as *Standard* section
Strength/Materials	4.5-MPa (650 psi) flexural	Same as *Standard* section	5.2-MPa (750 psi) flexural
Initial Smoothness	110 to 142 mm/km (7 to 9 in/mi) (measured with a 5-mm (0.2-in) blanking band)	Same as *Standard* section	Same as *Standard* section

Step 2: Compare Section B to the Standard Pavement Section

A closer look at *Section B* finds that only *Base/Subbase* type and *Strength/Materials* differ from the *Standard* pavement section. The analysis details for *Section B* are summarized in table 15.

Table 15. Expected percent changes in cost and performance associated with the changed design features of *Section B*.

Design Category	***Section B*** **Design Features**	Expected Change in Cost (%)	Expected Change in Performance (%)	Category Ranking
Base/Subbase	150-mm (6-in) cement-treated base (CTB)	+6.0	+5.0	8
Strength/Materials	5.2-MPa (750 psi) flexural	+5.0	+10.0	6

Since the costs associated with different design feature changes are cumulative for this example, the change in cost +6.0 + 5.0 = +11.0 percent (i.e., *Section B* is expected to cost 11.0 percent more than the *Standard* pavement section).

The true measure of performance for *Section B* is assumed to be somewhere between the lowest individual performance value and the sum of all of the values. Therefore, for this example the performance rating is between +5.0 percent (the lowest individual value) and +15.0 percent (the sum of all of the values). The category ranking factors are used to estimate the weighted contributions of each individual performance rating. This computation process is illustrated in table 16.

Table 16. Computation details associated with the changed design features of *Section B*.

Design Feature Category	***Standard*** **Pavement Section Design Features**	***Section B*** **Design Features**	Expected Relative Cost Change (%)	Expected Relative Performance Change (%)	Category Ranking Factor	Normalized Ranking Multiplier	Modified Performance (%)
Base/ Subbase	15-mm (6-in) dense-graded aggregate base	150-mm (6-in) cement-treated base	+6.0	+5.0	8	(8/8) = 1.00	+5.0
Strength/ Materials	4.5-MPa (650 psi) flexural	5.2-MPa (750 psi) flexural	+5.0	+10.0	6	(6/8) = 0.75	+7.5
TOTAL			**+11.0**				**+12.5**

Step 3: Comparison of Sections A and B

Finally, a simple comparison of the expected relative cost and performance changes for both Sections A and B is made by reporting the *Section B* cost and performance as ratios of the expected *Section A* values. Table 17 summarizes the expected relative cost and performance changes from the individual comparisons with the *Standard* pavement section.

Table 17. Summary of comparisons of Sections A and B to the *Standard* pavement section.

Pavement Section	Total Relative Cost Change (%)	Total Expected Modified Performance (%)	Benefit/Cost Ratio
Section A	+22.0	+0.5	(1.005/1.22) = 0.82
Section B	+11.0	+12.5	(1.125/1.11) = 1.01
B/A Ratio	**1.11/1.22 = 0.91**	**1.125/1.005 = 1.12**	

Therefore, a ratio of expected *Section B* costs to expected *Section A* costs is computed as (1.11/1.22) = 0.91 (i.e., the cost of *Section B* is expected to cost 0.91 times the cost of *Section A*). Using the same approach, a ratio of expected *Section B* performance to expected *Section A* performance is computed as (1.125/1.005) = 1.12 (i.e., *Section B* is expected to carry 1.12 times as many ESALs as *Section A*).

Step 4: Interpreting Benefit/Cost Ratios

Other interesting information can be gleaned by looking at the benefit/cost (B/C) ratios computed for each pavement section. These B/C ratios are informative in that they allow an assessment of the cost-effectiveness of a defined combination of different design features. In a comparison of two pavement sections, the section with the largest B/C ratio is the most cost effective section to construct. Another way to interpret this B/C ratio is that the larger the B/C ratio, the more performance achieved per dollar spent.

In the example above, the *Section A* numbers indicate that the chosen set of design features will cost 22 percent more, but will only result in a 0.5 percent increase in performance. These cost and performance numbers translate to a B/C ratio of 0.82. In contrast, the *Section B* numbers indicate that the chosen set of design features will cost 11 percent more, but will result in a 12.5 percent increase in performance. These *Section B* numbers translate to a B/C ratio of 1.01. Therefore, a comparison of B/C ratios indicates that *Section B* is more cost effective to construct than *Section A* (1.01 is greater than 0.82).

Introduction to the Analytical Software Tool

A major part of this project is the development of an analytical software tool that automates the analysis method described in the subsequent sections of this chapter. The software is provided as a tool for pavement engineers or contractors who are interested in investigating estimated cost versus performance trade-offs associated with the selection of different design features during the concrete pavement design process; it is absolutely not intended as a "design" tool. Instead, it provides a "reasonableness" check regarding the "justification" or "questioning" of the addition of different design features. Also, because the default cost and performance changes in the software are estimated based on collected survey data from all over the United States, it is strongly suggested that the user define cost sets, performance sets, and category ranking factor sets that reflect local experiences and conditions. That is, the results from this tool are only as good as the data on which they are based. Finally, it is again emphasized that the output results from this tool are solely "estimates" of cost and performance associated with changing design features and, therefore, should be used with caution.

The software utilizes a "modular" data storage approach in that all unique user-defined pavement sections, cost sets, performance sets, and category ranking factors sets are defined, named, and saved to respective "master lists" within the software. Therefore, the process of defining an analysis session is simplified in that the user simply builds an analysis session by selecting these named and saved data groups from their respective master lists. This approach was adopted because of the large number of data elements that need to be defined when conducting an analysis. When getting started with the software, users are encouraged to define all of the modular data groups anticipated for use in the analysis sessions. Much more detailed descriptions of the software's components are contained in the *Software User's Guide* included as appendix D of this report.

Once all unique pavement sections, cost data sets, performance data sets, and category ranking factor sets of interest are defined and saved within the software, the user may then focus on building specific analysis sessions of interest. Specifically, two general types of analysis sessions may be conducted using the software:

1) *Direct Comparison*—The *Direct Comparison* analysis is used to compare two defined pavement sections to assess expected differences in cost and performance. A by-product of this type of analysis is the B/C ratio associated with each section. In a comparison of two pavement sections, the section with the largest B/C ratio is the most cost effective section to construct (the larger the B/C ratio, the more performance achieved per dollar spent).

2) *Sensitivity Analysis*—The *Sensitivity Analysis* is provided as a method of defining more complex relationships among pavement designs. Specifically, the following two general types of sensitivity analyses may be defined in the software:

    - Comparison of multiple pavement sections—The first sensitivity analysis type that may be defined is the comparison of one pavement section to a number of other pavement sections with different design features. This series of direct section comparisons is conducted while holding all other analysis inputs constant.

    - Sensitivity of a chosen pavement section to other process inputs—The second sensitivity analysis type allows the user to investigate the sensitivity of one defined pavement section to changes in the other inputs of the process. Specifically, these sensitivity analyses are used to compute cost and performance changes associated with subjecting a chosen pavement section to a series of different cost data sets, performance data sets, category ranking sets, or a combination of both cost and performance sets. Each of these different data sets is discussed in detail in the *Software User's Guide* included as appendix D.

The results of any defined analysis session are summarized in a customizable output report that may be previewed in an on-screen window or printed. The detailed output report contains all details of the analysis session including a list of input values, tables of intermediate cost and performance computations, and a series of summary tables for cost, performance, and LCCA results.

Summary

This chapter outlines the simple analysis approach used in this project to estimate overall cost and performance changes associated with changing pavement design features. The basis components of the analysis procedure are first discussed in detail to explain the underlying theory used to estimate overall changes in cost and performance. Next, the analysis method is demonstrated through examples to further clarify the details of the analysis method. Finally, the associated analytical software tool is introduced with specific discussion of the intended usage and limitations of the software. A detailed explanation of how to define and conduct analysis sessions using the analytical software tool is contained in the *Software User's Guide* (included as appendix D).

CHAPTER 4. SUMMARY

Various design features (such as dowel bars, tied shoulders, or drainable bases) may be added to a PCC pavement design to improve its overall performance by maintaining a higher level of serviceability or by extending its service life. However, the addition of these features also increases the initial cost of the pavement design, in some cases quite significantly. This then raises the question of whether the improved performance benefits gained by adding the design features are worth the increase in cost. Furthermore, the effects of adding more and more design features to a PCC pavement design may produce smaller and smaller performance gains, while significantly increasing the overall costs of the pavement structure. Unfortunately, current design practices do not always consider this trade-off between performance benefits and costs when design features are added to a PCC pavement design.

To address this need, this project was initiated to develop a methodology and an evaluation tool that can be used to assess the costs and benefits of incorporating design features in PCC pavements. To establish a foundation for quantifying the relative performance benefits and costs of the various PCC pavement design features, two detailed surveys were conducted. The first survey targeted SHAs and collected relative performance ratings for changes in design features; that is, the expected percent change in performance if a selected design feature is added to the *Standard* PCC pavement design. The performance ratings received from the SHAs were evaluated in conjunction with available PCC pavement performance prediction models to check their validity and reasonableness. The second survey targeted PCC paving contractors and collected relative cost ratings for changes in design features; that is, the expected percent change in cost if a selected design feature is added to the *Standard* PCC pavement design.

Each questionnaire was structured in a similar manner so that the relative performance and cost data could be matched up for each design feature. The results from these surveys form the basis of the default data sets used in the evaluation tool. Copies of the questionnaires sent to the participating agencies and contractors are presented in appendix B, with the raw data and summarized results presented in appendix C.

The primary product developed under this project is an analytical software tool that can be used to evaluate the relative performance benefits and costs associated with the addition of different design features to a PCC pavement design. The software tool can be used by pavement design engineers who are interested in investigating the cost versus performance trade-offs associated with the selection of different design features during the PCC pavement design process. By allowing pavement design engineers to compare the impact of different design features on a pavement's expected performance and its construction and life-cycle costs, insight can be gained on the most cost-effective combinations of design features for a particular PCC pavement design. A detailed explanation on the use and application of the analytical software tool is provided in appendix D.

It is important to recognize that the software developed under this project is merely a computational tool that allows for the comparison of costs and performance of PCC pavements with different design features. The tool can use the default cost and performance data sets that were developed in this study or it can use custom-defined data sets that are based on an agency's local conditions and experience. The tool is not intended to provide absolute answers on the effect of different design features, but rather to provide insight into general performance and cost trends associated with the use of different design features. In this way, the tool can be used in developing more cost-effective PCC pavement designs, and reflects the strong need to consider both the performance benefits and the costs associated with the inclusion of PCC pavement design features.

The anticipated users of the software tool include State highway engineers, design engineers, paving contractors, and industry representatives. It is recommended that the tool be distributed by FHWA to these users, either through direct mailings or through establishment of a Web page containing the software in downloadable format. In addition, regional workshops could be hosted by FHWA (either in conjunction with the FHWA's resource centers or perhaps with local ACPA chapters) as a further aid in implementing the software.

REFERENCES

1. Darter, M. I. 1977. Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume I—Development of Design Procedure. FHWA-RD-77-111. Federal Highway Administration, Washington, DC.

2. Smith, K. D., D. G. Peshkin, M. I. Darter, A. L. Mueller, and S. H. Carpenter. 1990. *Performance of Jointed Concrete Pavements, Volume I—Evaluation of Concrete Pavement Performance and Design Features*. FHWA-RD-89-136. Federal Highway Administration, McLean, VA.

3. Smith, K. D., M. J. Wade, D. G. Peshkin, L. Khazanovich, H. T. Yu, and M. I. Darter. 1998. *Performance of Concrete Pavements, Volume II—Evaluation of Inservice Concrete Pavements*. FHWA-RD-95-110. Federal Highway Administration, McLean, VA.

4. Khazanovich, L., M. I. Darter, R. Bartlett, and T. McPeak. 1998. *Common Characteristics of Good and Poorly Performing PCC Pavements*. FHWA-RD-97-131. Federal Highway Administration, McLean, VA.

5. Smith, K. D. and K. T. Hall. 2001. *Concrete Pavement Design Details and Construction Practices*. Reference Manual for NHI Course 131060. National Highway Institute, Arlington, VA.

6. American Association of State Highway and Transportation Officials (AASHTO). 1993. *AASHTO Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials, Washington, DC.

7. American Association of State Highway and Transportation Officials (AASHTO). 1998. *Supplement to the AASHTO Guide for Design of Pavement Structures, Part II—Rigid Pavement Design and Rigid Pavement Joint Design*. American Association of State Highway and Transportation Officials, Washington, DC.

8. Smith, K. D., A. L. Mueller, M. I. Darter, and D. G. Peshkin. 1990. *Performance of Jointed Concrete Pavements, Volume II—Evaluation and Modification of Concrete Pavement Design and Analysis Models*. FHWA-RD-89-137. Federal Highway Administration, McLean, VA.

9. Yu, H. T., K. D. Smith, M. I. Darter, J. Jiang, and L. Khazanovich. 1998. *Performance of Concrete Pavements, Volume III—Improving Concrete Pavement Performance*. FHWA-RD-95-111. Federal Highway Administration, McLean, VA.

10. Walls, J. and M. R. Smith. 1998. *Life-Cycle Cost Analysis in Pavement Design—Interim Technical Bulletin*. FHWA-SA-98-079. Federal Highway Administration, Washington, DC.

APPENDIX A. ANNOTATED BIBLIOGRAPHY

The focus of the literature review was to locate references discussing cost/benefits of concrete pavement design features. The basis for the current project reference list was a previously conducted literature review completed for the FHWA/NHI training course *Concrete Pavement Design Details and Construction Practices*. The results of that literature search focused on references that discussed pavement design features and their effects on pavement performance.

As a means of upgrading this existing reference list, additional targeted searches were completed using the TRIS, NTIS, and EI Compendex national bibliographic databases. These searches were limited to recent publications (TRIS: 1995 to 2000; NTIS: 1990 to 2000; EI Compendex: 1998 to 2000). The search identified slightly more than 100 additional records that were deemed potentially useful to the project.

After reviewing the records identified in the new literature searches, a short summary (or annotation) describing the contents of each relevant document was prepared. A final annotated bibliography was created by combining the initial results of the FHWA/NHI training course bibliography with the pertinent records found in the more recent searches.

Base/Subbase Design

1. American Concrete Pavement Association (ACPA). 1995. Subgrades and Subbases for Concrete Pavements. Technical Bulletin TB011.02P. ACPA, Skokie, IL.

   This bulletin provides guidance on preparing subgrades and subbases for concrete pavements. Emphasis is on the establishment of uniform support for the pavement that will prevail throughout its life. Guidance is included on the preparation of subgrade, identification and treatment of expansive soils, protection against frost action, subbase drainage requirements, and the design and construction of cement-treated, lean concrete, and permeable subbases.

2. Crovetti, J. A., and B. J. Dempsey. 1993. "Hydraulic Requirements of Permeable Bases." *Transportation Research Record 1425*. Transportation Research Board, Washington, DC.

   The design requirements of open-graded permeable materials (OGPM) to handle surface water infiltration are reported. Infiltration rates and required permeabilities of the OGPM are calculated for a range of conditions typical for pavement design. The effects of pavement geometry on required permeabilities, including cross slope, longitudinal gradient, and drainage layer thickness and width, are discussed. Analysis of selected materials, typical for use in Illinois, is completed to determine appropriate permeabilities.

3. Hajek, J. J., T. J. Kazmierowski, H. Sturm, R. J. Bathurst, and G. P. Raymond. 1992. "Field Performance of Open-Graded Drainage Layers." *Transportation Research Record 1354*. Transportation Research Board, Washington, DC.

   The results of a field study to investigate the performance of pavements incorporating open-graded drainage layers (OGDL) are presented. Five paving projects built since 1975 and incorporating OGDLs are described and evaluated. The projects encompass flexible, composite, and rigid pavements, and include both asphalt-treated and untreated OGDL materials. The results show that the existence of OGDLs alone does not guarantee better pavement performance. This can be achieved only if the OGDL is part of a properly designed internal drainage system.

4. Hall, M. 1994. "Cement-Stabilized Open-Graded Base Strength Testing and Field Performance Versus Cement Content." *Transportation Research Record 1440*. Transportation Research Board, Washington, DC.

   The use of cement-stabilized open-graded base (CSOGB) to provide a drainage system and construction platform for concrete pavements is explored. Extensive laboratory testing was combined with a field evaluation project to assess the capabilities of CSOGB. Low-, medium-, and high-cement content material was placed on grade and used as a haul road during paving. The condition of the CSOGB was monitored during construction; and trucks loaded with concrete and empty were counted. Performance under load was found to depend on cement content, truck traffic, sublayer stability, segregation, and surface irregularities. A cement content of 119 kg/m^3 (200 lb/yd^3) was suitable for general use; 89 kg/m^3 (150 lb/yd^3) was adequate for low trucking volumes, and 148 kg/m^3 (250 lb/yd^3) was appropriate for high trucking volumes or poor support conditions.

5. Heckel, L. 1997. "Open-Graded Drainage Layers: Performance Problems Under Continuously Reinforced Concrete Pavements in Illinois." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance.* Purdue University, West Lafayette, IN.

Illinois has recently constructed 14 pavement sections that contain OGDLs. The pavement types include CRCPs, jointed concrete pavements, and full-depth asphalt pavements. Three of the CRCPs have begun to show a higher level of distress than would be otherwise expected, so an in-depth investigation has been conducted consisting of visual distress surveys, core analysis, analysis of Shelby tube samples, analysis of OGDL gradations, separation layer gradations, subgrade gradations, deflection tests, and inspection of slabs removed in patching operations. The distresses evident on these pavements may be related to the characteristics of the OGDL, separation layer, and subgrade. The bonding of the CRCP to the OGDL, as well as crack spacing, may have contributed to the poor performance of these pavements.

6. Heckel, L. 1997. "Performance Problems of Open-Graded Drainage Layers Under Continuously Reinforced Concrete Pavements In Illinois." *Transportation Research Record 1596*. Transportation Research Board, Washington, DC.

Illinois has constructed 14 pavements that contain OGDLs. The pavement types include CRCPs, jointed pavements, and full-depth asphalt concrete pavements. Three of the CRCPs have begun to show a higher level of distress than would be expected from pavements of that cross section and age. An in-depth investigation into the cause of the distress has begun and is expected to be completed early in 1997. The investigation includes visual distress surveys, core analysis, analysis of Shelby tube samples, deflection tests, and inspection of slabs removed in patching operations. The State has placed a moratorium on the construction of new OGDLs pending the outcome of the investigation. The investigation indicates that possible causes of the problems are the incompatibility of the OGDL and CRCP, the lack of a separation layer on two of the three sections, the cement content in the OGDL, the percentage of reinforcing steel and epoxy-coating of the steel in the slab, and the design of the CRCP and shoulders. The current maintenance plan includes patching and undersealing of problem areas. No recommendations for the future use of OGDLs can be made until the investigation narrows the possible causes.

7. Ioannides, A. M., L. Khazanovich, and J. L. Becque. 1992. "Structural Evaluation of Base Layers in Concrete Pavement Systems." *Transportation Research Record 1370*. Transportation Research Board, Washington, DC.

A theoretically sound and practical approach is described for determining maximum responses in concrete pavement systems that incorporate a base layer. Equations presented may be used with either an elastic solid or a dense liquid foundation under any of the three fundamental loading conditions. These formulas are extensions of available closed-form solutions and account for the compressions in the two placed layers that are ignored by plate theory. The proposed methodology may be easily implemented in a personal computer spreadsheet or on a programmable calculator. Research activities for its full verification and refinement are continuing. It is anticipated that such theoretically based investigations will encourage the elimination of theoretically questionable empirical concepts, such as that of deriving a composite "top-of-the-base" subgrade modulus.

8. Kazmierowski, T. J., A. Bradbury, and J. Hajek. 1994. "Field Evaluation of Various Types of Open-Graded Drainage Layers." *Transportation Research Record 1434*. Transportation Research Board, Washington, DC.

   In 1990, the Ministry of Transportation of Ontario initiated a demonstration project to evaluate alternative methods of increasing the constructibility of open-graded drainage layers beneath concrete pavements. Three types of OGDLs were placed in this study: cement-treated OGDL with varying cement contents, untreated OGDL, and an asphalt-treated OGDL. All three met the requirements for permeability and stability, including the ability to carry construction traffic without any significant damage. The cement- and asphalt-treated bases required some minor adjustments to conventional construction practices. The untreated base exhibited some sloughing at the edges of the pavement and exhibited somewhat lower strength characteristics than the treated materials.

9. Mathis, D. M. 1989. "Permeable Base Design and Construction." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation*. Purdue University, West Lafayette, IN.

   This paper summarizes the state of the practice in use of permeable bases for drainage of asphalt and concrete pavements. Open-graded or "permeable" bases are used to allow infiltrated water to rapidly drain through the base and out from beneath the pavement structure. The practices of several States with respect to permeable base gradations, dimensions, collector systems, filter layers, and construction are described, and observations presented on performance, ride quality, and cost of pavements with permeable bases.

10. Minnesota Department of Transportation (Mn/DOT). 1994. *Permeable Aggregate Base Drainage Systems—Design Guidelines*. Final Report. Minnesota Department of Transportation, Maplewood, MN.

    These drainage guidelines are intended to aid the design engineer in determining when, where, and how to use permeable aggregate bases. Permeable bases, used in conjunction with edge drains and discharge pipes, are currently seen as the most positive method of removing excess moisture from beneath a pavement structure. This type of drainage system is expected to improve the overall performance and service life of the pavement structure and reduce maintenance costs. However, the indiscriminate use of permeable bases beneath all pavement structures is not recommended. Traffic, subgrade soil type, pavement type, and functional class are just a few variables to be considered in determining the appropriateness of the system for a given project.

11. Nelson, T. B. and S. Owusu-Ababio. 1999. *Effects of Thick Subbases on Pavement Design, Construction, Performance and Life Cycle Costs*. Final Report. Report No. WI/SPR-14-99, WisDOT Study # 94-09. Wisconsin Department of Transportation (WisDOT), Madison, WI.

In an effort to improve the performance of pavements in Wisconsin, WisDOT was interested in reviewing the practice and performance of thick subbases within pavement structures. While some pavements have been constructed in Wisconsin using subbases of thicknesses between 300 mm and 600 mm (12 inches and 24 inches), the design inputs, the pavement performance, and the life cycle costs associated with these pavements have not been documented. This study evaluated 20 pairs of adjacent asphalt and 8 pairs of jointed reinforced concrete pavement (JRCP) sections that were constructed with thin and thick subbases. These adjacent pairs were selected to minimize differences in performance due to traffic, soils, or environment. An evaluation of time to first overlay indicated that thick subbases provided an additional 1.6 to 13.7 years for asphalt pavements, and an additional 4.8 to 13.7 years for concrete pavements. A life cycle cost analysis showed costs of thick and thin subbases to be comparable for both flexible and rigid pavements. While this trend toward longer pavement life due to the use of thick subbases is important, the lack of detailed design-construction data and maintenance records, and the small number of sections available for study precluded detailed analysis to definitively correlate increased service life to increased subbase thickness for the JRCP. For the asphalt pavements, service life was found to increase with increasing subbase thickness.

12. Reed, C. M. 1995. "Impact of Open-Graded Drainage Layers on the Construction of Concrete Pavements in Illinois." *Transportation Research Record 1478*. Transportation Research Board, Washington, DC.

The first portland cement concrete pavement test section containing an OGDL was constructed by the Illinois Department of Transportation in 1989. Since then more than 20 centerline miles of concrete pavements have been built using an OGDL. Although the benefits of using OGDLs to improve concrete pavement performance are widely accepted, the practical impact of the design details on the constructibility of concrete pavements requires additional attention. Examples of problems encountered during the construction of concrete pavements with OGDLs include the placement of the layer itself, anchoring dowel baskets to the OGDL, obtaining a pavement with adequate ride characteristics, and using the OGDL to support paving operations. From the experience acquired while constructing several pavements with OGDLs in Illinois, it is clear that concrete pavements with OGDLs can be constructed to meet today's high construction standards.

13. Signore, J. M. and B. J. Dempsey. 1998. *Accelerated Testing of Separation Layers for Open-Graded Drainage Layers*. Final Report, Project C960014. Illinois Department of Transportation, Springfield, IL.

An accelerated testing procedure was developed to evaluate the performance of separation layers used between open-graded bases and lime-stabilized subgrades. Of significant concern was assessing the performance of the separation layer with regard to the degree of pumping into the open-graded base and the magnitude of the deformations occurring during the testing. A low plasticity clay and a silty clay till soil were tested in the research along with two separator materials: nonwoven geotextile and dense-graded base aggregate blend. Geotextiles consistently provided separation between the soils and open-graded aggregates, although pumping of fines occurred. The dense-graded separation layer showed a marked drop-off in performance when the material was wet. Although the dense-graded layer often prevented the soil layer from intruding into the open-graded layer, the dense-graded layer itself intermixed into the open-graded layer. A distinct breakpoint in performance occurred at a soil strength of CBR 4 under accelerated testing. Soils below a CBR of 4 showed considerable pumping and deformation while soils above that value yielded minimal pumping and deformation. Based upon performance comparisons with nonseparated test cases, the use of a separation layer between lime stabilized subgrades and open-graded aggregate bases is imperative.

14. Smith, K. D., H. T. Yu, and M. I. Darter. 1993. "A Performance Evaluation of PCC Pavements Constructed on Permeable Bases." *Western States Drainable PCC Pavement Workshop—Summary Report*. FHWA-SA-94-045. Federal Highway Administration, San Francisco, CA.

    In recent years, many highway agencies have used permeable base courses to provide positive drainage to their pavement structures. However, little information is available regarding how these permeable bases actually affect the performance of the concrete pavement. This paper presents a summary of the performance of 30 concrete pavement sections that incorporate permeable bases in their design. Direct comparisons between permeable and nonpermeable base sections were possible for many of the projects. The permeable base sections had less faulting than the nonpermeable base sections. This was more apparent for nondoweled joints than for doweled joints. No discernible difference between the performance of permeable aggregate, permeable asphalt-treated, and permeable cement-treated bases was observed. However, the results of this investigation are considered preliminary, as the pavement sections evaluated are not very old and have not been subjected to significant traffic levels.

15. Tarr, S. M., P. A. Okamoto, M. J. Sheehan, and R. G. Packard. 1999. *Bond Interaction Between Concrete Pavement and Lean Concrete Base*. Preprint No. 991273. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

    In the past, concrete pavement design procedures have been based on stresses computed with the assumption of either a frictionless or fully bonded interface between pavement layers, but neither of these assumptions is realistic for layers not intentionally bonded. The purpose of the research described in this paper is to provide information on the degree of interaction between pavement layers. Sponsored by the Portland Cement Association, full-scale load-induced strain and falling weight deflectometer (FWD) testing was conducted to measure the degree of bonding between PCC pavement and a lean concrete base (LCB) layer prepared using five commonly specified interface treatments. For each interface treatment, the effect of extending the LCB layer beyond the slab edge was also evaluated. Strain gages installed at different pavement depths measured the load-induced edge and wheelpath stress profiles, which allowed the degree of layer interaction to be evaluated. By analysis of load-induced stress and deflection, it was verified that a double layer of polyethylene promotes an unbonded interface with very little frictional interaction. Other interface treatments create varying degrees of friction or partial bond between the layers,

which reduces the load-induced stress. Slab bottom tensile stress reductions were computed in comparison to measured slab top compressive stress, measured polyethylene interface tensile stress, and theoretically calculated unbonded stress. Overall, the most effective stress-reducing interface treatments included in this study were asphalt emulsion and plain (no treatment) conditions where tensile stress reductions of more than 40 percent were measured.

16. Winter, M. G., J. R. Cross, and J. Oliphant. May 1997. "Use and Cost-Effectiveness of Permeable Geosynthetics on Scottish Road Projects." *Proceedings of the Institution of Civil Engineers-Transport*. Volume 123, Issue 2.

The use of permeable geosynthetics on Scottish trunk roads was studied between 1980 and 1994. Questionnaires were sent to 55 organizations and details of a total of 103 applications were received. Calculations show that permeable geosynthetics are generally more cost effective than conventional solutions; this is supported by a review of the available literature. Relatively few data were obtained on alternative solutions. If the increasing use of permeable geosynthetics is to be justified economically, then alternative solutions need to be considered and costed. However, their use is expected to increase regardless, as justification is frequently on technical grounds. Problems identified in relation to the use of permeable geosynthetics in soil environments emphasize the importance of correct design and specification, careful installation and adequate site drainage. In addition, problems were encountered with reinforcement of pavements. The use of geogrids in bound pavement structures is not currently allowed on Scottish trunk roads and motorways. There is no evidence of contractual claims arising from the use of permeable geosynthetics.

Concrete Mix Considerations

1. Chini, S. A., J. P. Duxbury, S-S. Kuo, W. J. Mbwambo, and F. Monteiro. 1998. *Guidelines and Specifications for the Use of Reclaimed Aggregates in Pavement*. Final Report. Report No. WPI 0510797. Federal Highway Administration, Washington, DC.

The objective of this research study was to evaluate the performance of nine recycled concrete aggregate (RCA) design sections through both theoretical and experimental analyses. Five of these sections were 254 mm (10 in.) thick slabs of concrete pavement with varying percentages of recycled and virgin aggregates. The remaining four sections were composed of an 88.9 mm (3.5 in.) layer of hot mix asphalt (HMA) with a 12.7 mm (0.5 in.) friction wear course, over different thicknesses of RCA base material. Both types of pavements had control sections to which the test data could be compared and contrasted. For the analytical approach, the KENSLABS and KENLAYER computer programs were used to determine the stresses in the concrete slabs along with the tensile and compressive strains in the HMA sections. These parameters were then used to estimate the theoretical number of allowable repetitions for failure to occur in the individual sections. Part 1 of this report presents the results of the study of the use of RCA in rigid pavement. It includes a literature review, a review of the use of RCA in the United States, and laboratory test results. Part 2 presents the results of the study of the use of RCA as a base course in flexible pavement. It includes a literature review, a review of the state of concrete recycling in Florida, a look at aggregate properties important for base courses, and a review of international standards and U.S. Department of Transportation standards. Part 3 gives the results of the performance tests, and Part 4 the results of the theoretical analysis.

2. Cramer, S. M., M. Hall, and J. Parry. "Effect of Optimized Total Aggregate Gradation on Portland Cement Concrete for Wisconsin Pavements." *Transportation Research Record 1478*. Transportation Research Board, Washington, DC.

   Most State paving specifications for PCC pavement allow a broad range of total aggregate gradation for concrete mixes. It has long been debated whether special efforts to control total aggregate gradation provide concrete improvements that justify potential increased costs. The results of an investigation examining the effect of optimizing total aggregate gradation on the properties of concrete used for paving in Wisconsin are reported. The investigation optimized gradations consisting of carefully selected proportions of locally available aggregate. Unit weight, shrinkage, change in the water-to-cement (w-c) ratio at constant slump, change in slump at a constant w-c ratio, compressive strength, and possible segregation under vibration were measured in field test sections and laboratory mixes. This use of optimized total aggregate gradations instead of near-gap-graded gradations in pavement resulted in an increase in compressive strength of 10 to 20 percent, reduced water demand by up to 15 percent to achieve comparable slump, air contents achieved with 20 to 30 percent reductions in air entraining agent, potentially higher spacing factors in the air void system of hardened concrete, and reduced segregation following extended vibration (1 to 3 minutes). Not all efforts at gradation optimization in this study yielded measurable improvements in performance and the availability of local aggregates may still limit, to varying degrees, the ability to optimize. However, a reasonable effort to optimize gradation can lead to significant mix benefits.

3. Cuttell, G. D., M. B. Snyder, J. M. Vandenbossche, and M. J. Wade. "Performance of Rigid Pavements Containing Recycled Concrete Aggregates." *Transportation Research Record 1574*. Transportation Research Board, Washington, DC.

   State highway agencies in Connecticut, Kansas, Minnesota, Wisconsin, and Wyoming have successfully designed and constructed rigid pavements containing recycled concrete aggregate (RCA). Success has been attributed in part to the minimization of old mortar content in the RCA during recycling processes, thereby controlling the total mortar content of the new PCC mixture, or to the achievement of higher-than-expected compressive strengths through adjustments in mix proportions, or both. There was no clear correlation between mortar content and cracking distresses in field investigations, although one project did exhibit significantly more slab cracking in the recycled pavement than in the corresponding control pavement. The increased cracking may have been due to the large differences in total mortar content between the recycled and control sections. In general, the recycled PCC pavements considered in this study have performed comparably with their conventional PCC pavement counterparts, including the recycled pavements that incorporated RCA derived from concrete affected by D-cracking and alkali-silica reactivity (ASR). There is, however, evidence of small amounts of localized recurrent ASR in the recycled Wyoming pavement. Whether this reactivity will eventually develop into widespread distress remains to be seen.

4. Lane, D. S. 1998. *Evaluation of Concrete Characteristics for Rigid Pavements*. Report No. VTRC-98-R24. Virginia Transportation Research Council. Virginia Department of Transportation (VDOT), Richmond, VA.

The researcher developed correlations among flexural, split tensile, and compressive strengths and ultrasonic pulse velocity from laboratory testing using materials and mix designs proposed for use in a pavement project. These relationships were used to review the current VDOT specifications and quality control procedures for concrete used in the construction of rigid pavements. Correlations between compressive and flexural strength for project-specific materials and mix designs permit the use of compressive strength cylinders rather than beams for compliance testing. The relationships also provide a means for evaluating the quality of the concrete as placed in the pavement.

5. Ozyildirim, G. Jan 1993. "High-Performance Concrete for Transportation Structures." *Concrete International: Design & Construction*. Volume 15, Issue 1.

When bridges and pavements are repaired or rebuilt, lane closure time should be minimized to increase safety and reduce costs and inconvenience for the traveling public. To provide a short lane closure time, special concretes are needed that attain an appropriate strength more rapidly than is possible with conventional concretes. The research reviewed in this paper demonstrates the usefulness of special concretes to provide performance and placement advantages not attainable with conventional concretes. The special concretes discussed are those containing slag, those with pozzolanic additions (silica fume or fly ash), latex-modified concrete, concretes made with Pyrament-blended cement, and portland cement with a low water-cement ratio for fast-track construction. Field applications and supporting laboratory investigations document the characteristics and superior performance levels of these special concretes.

6. Snyder, M. B. 1995. *Use of Crushed Concrete Products in Minnesota Pavement Foundations*. Final Report. Report No. MN-RC-96-12. Minnesota Department of Transportation, St Paul, MN.

This report reviews 11 field and laboratory studies performed to address concerns about the use of recycled concrete aggregate in pavement foundations. Performance concerns have centered on the possible impairment of drainage systems by deposits of calcium carbonate precipitate and other fines derived from the recycled concrete base materials. Environmental concerns have focused on the relatively high pH of the effluent produced by drainage systems that remove water from untreated recycled concrete aggregate foundation layers. The studies considered in this report demonstrate that all recycled concrete aggregates are capable of producing various amounts of precipitate, with the precipitate potential being directly related to the amount of freshly exposed cement mortar surface. It appears that selective grading and blending with virgin aggregates are techniques that should significantly reduce precipitate potential. One study suggests that washing recycled concrete products will reduce accumulations of crusher dust and other fines in and around the pavement drains. Others indicate that the use of filter fabrics with sufficiently high initial permittivity will allow the accumulation of precipitate and other fines without significantly impairing drainage function. This report discusses study results related to environmental concerns and provides recommendations for revisions to current specifications.

7. Sukley, R. 1995. *Accelerated Rigid Paving. Final Report*. Demonstration Project 201. Report No. FHWA-PA-95-006. Federal Highway Administration, Washington, DC.

   The use of accelerated cement concrete mixes in reconstruction or rehabilitation of existing highways significantly reduces lane closure times, and costs for maintenance, protection of traffic, and user delay. This study evaluated the development of two accelerated PCC mixes, which attained a compressive strength of 20.7 MPa (3000 psi) in 24 hours. These design mixes were then placed in high truck traffic pavement areas for field evaluation. The fast track concrete construction was monitored and found to be more labor intensive, but otherwise similar to the placement of normal concrete. The use of the nondestructive testing concrete maturity meter, to predict compressive strength as a function of time and performance in these projects, was a valuable quality control tool. This evaluation lasted 5 years, and the performance has been satisfactory except for initial and minor cracking. The initial cracking may have formed due to slab widths in excess of 4.27 m (14 ft) in the first project, and hairline cracking in the second site may have been caused by excessive heat of hydration. Accelerated rigid paving is recommended as a standard special provision as an alternative to class AA concrete for pavements in situations where time of closure is an important factor.

8. Tikalsky, P. J. and A. Scanlon. 2000. *High-Performance Concrete*. Transportation Operations and Systems Research and Development Partnership. Report No. PTI 2K16. Pennsylvania Transportation Institute, University Park, PA.

   The primary goal of this research project was to evaluate the Pennsylvania Department of Transportation's (PennDOT) current concrete mixture designs for performance characteristics and provide specific recommendations on the effective use of concrete with high-performance characteristics. Highway concrete mixtures in Pennsylvania are largely designed for strengths between 23 and 31 MPa (3,300 and 4,500 psi), and for resistance to freezing and thawing. While strength and freeze-thaw resistance are important in Pennsylvania, other parameters impact the long-term performance of concrete in highway applications. Concrete can be developed to address economic considerations, as well as multiple combinations of strength, permeability, modulus, cracking tendency, abrasion resistance, freeze-thaw resistance, alkali-aggregate reaction, internal and external sulfate attack, workability, construction scheduling, traffic openings, or other criteria. The report defines high performance concrete (HPC) in the context of the PennDOT; describes the characteristics and benefits derived from the use of HPC; evaluates the current state of the practice in Pennsylvania; and identifies the performance criteria that benefit PennDOT bridges, structures, and concrete pavements. It also provides a series of recommendations for consideration for the Commonwealth of Pennsylvania.

9. Wade, M. J., G. D. Cuttell, J. M. Vandenbossche, H. T. Yu, K. D. Smith, and M. B. Snyder. 1997. *Performance of Concrete Pavements Containing Recycled Concrete Aggregate*. FHWA-RD-96-164. Federal Highway Administration, Washington, DC.

   This report documents the field performance of nine concrete pavement projects that incorporate recycled concrete aggregate (RCA) in the construction of the pavement. Multiple sections were evaluated on many of the nine projects due to perceived differences in performance levels or variations in pavement design (such as the use of virgin aggregate or the inclusion of dowel bars). All told, a total of 17 sections (of which 12 contain RCA) were subjected to an extensive field testing program consisting of pavement condition surveys, drainage surveys, FWD testing, coring, and serviceability assessments. Each of the 17 sections included in the investigation is described in detail. Performance observations and results from the FWD and laboratory testing are presented, with emphasis on evaluating the effect of RCA on pavement performance. An overall summary is provided that synthesizes the findings and conclusions of the field-testing program.

10. Wang, Z. and S. Yu. 1997. "A Study on Optional Design of Early-Strength Cement Concrete for Pavements." *XIIIth World Meeting of the International Road Federation,* Toronto, Canada. June 16-20, 1997. International Road Federation, Washington, DC.

   This paper focuses on optimal concrete mix design, including the determination of w/c ratio (water over cement in quantity), amount of admixtures, and grading of course aggregate for the cement concrete. Through this research, a special methodology was developed for a concrete mix design that is capable of adapting to changeable temperature, environment and climate, surface characters of the course aggregate, and concrete curing time for pavements. In addition, this paper presents new ideas in the design of concrete containing admixtures and in the study of surface characteristics of the coarse aggregate. The method is also applicable to the design of ordinary PCC.

Construction

1. American Concrete Institute (ACI). 1991. "Guide for Construction of Concrete Pavements and Concrete Bases." *ACI Manual of Concrete Practice*. ACI 325.9R-91. American Concrete Institute, Detroit, MI.

   This report covers the construction of concrete pavements and concrete bases without attempting to include inflexible specifications for procedures, materials, or equipment. References are made to specifications, but only as a guide to enable a selection of requirements suitable for a particular location or class of work. Sections are devoted to specifying, sampling, and testing materials, and to the possible influence of materials on skid resistance, economy, and durability. Subgrades and subbases are treated only as a final preparation for paving. Arrangement of joints is described, and references are given for guidance in using reinforcement. Sections on mixing, placing, finishing, and curing concrete refer to other ACI reports where pertinent, but make recommendations for the special handling requirements for pavement construction.

2. American Concrete Pavement Association (ACPA). 1994. *Fast-Track Concrete Pavements*. TB004.02P. American Concrete Pavement Association, Skokie, IL.

This technical bulletin describes the design and construction of fast-track concrete pavements. It includes a summary of appropriate applications of fast-track paving technology and also describes the planning efforts required. Considerable discussion is spent on obtaining suitable concrete material for fast-track paving projects, including the cement type and content, supplementary cementing materials, use of admixtures, and aggregate gradation and properties. Details on critical construction aspects of fast-track paving are also provided, including a description of recommended curing and temperature management procedures.

3. American Concrete Pavement Association (ACPA). 1996. *Construction of Portland Cement Concrete Pavements—Participant's Manual*. FHWA-HI-96-027. Federal Highway Administration, Washington, DC.

This manual accompanies a 2 ½ day training course on concrete pavement construction. The course material is aimed at field personnel, both contractor and agency, and is intended to provide them with a general working knowledge of field operations. Field operations presented here include central mix plant operations, ready mix plant operations, slipform paving operations, fixed-form paving operations, joint sawing and sealing operations, and concrete pavement restoration activities.

4. Ayton, G. P. and E. W. Haber. 1997. "Curing and Interlayer Debonding." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance.* Purdue University, West Lafayette, IN.

Current Australian practices in the areas of concrete curing and interlayer debonding reflect a blending of experience from leading overseas authorities together with that obtained from substantial local experience since the 1970s. Australian practices were summarized (for the wider range of design and construction issues) in a paper presented at the Fifth Purdue Conference in 1993, but significant developments have occurred since. In terms of curing practices, the major developments have been in the types of curing compounds being used (for both pavements and bases), and in the method of specifying and monitoring their quality in contracts. In the area of interlayer debonding, changes have been made to the specified materials and methods. With few exceptions, concrete pavements constructed within the past 20 years have been over a lean-mix concrete base. Ongoing experience confirms that the level of interlayer friction is critical to the early thermal movement and to joint induction. It is also critical to the avoidance of unplanned reflection cracking in the pavement and thereafter to its long-term performance. Particular attention has been focused on refining debonding treatments under continuously reinforced pavement following experience on several projects, which exhibited undesirable cracking patterns that were thought to have been largely a consequence of too high a level of interlayer friction. This paper discusses the Australian specifications and practices in these areas. Comparison is also made to the international experience as reflected in the published literature, and a brief account is provided of current ongoing development work in this field. A brief discussion of factors (additional to interlayer friction) that appear to influence cracking patterns in continuously reinforced concrete pavements is provided.

5. Cole, L. W. and G. F. Voigt. 1995. "Roadway Rehabilitation with Fast-Track Concrete Paving." *Transportation Congress, Proceedings*. Volume 1. American Society of Civil Engineers, New York, NY.

To meet the need of transportation authorities, accelerated methods for constructing concrete pavements have been developed. These techniques, called fast-track concrete paving, have been developed through several years with experience gained on many projects. Fast-track concrete paving requires changes to traditional construction specifications and processes. In addition to material modifications, changes in worker responsibilities, construction staging, pavement joint construction, and opening-to-traffic criteria can be made to accelerate a concrete paving project.

6. Cole, L. W. and G. F. Voigt. 1996. "Fast-Track Concrete Paving—Overview of Key Components." *Materials for the New Millennium Proceedings of the Materials Engineering Conference*. Volume 1. American Society of Civil Engineers, New York, NY, USA.

To meet the need of transportation authorities, accelerated methods for constructing concrete pavements have been developed. These techniques, called fast-track concrete paving have been developed through several years with experience gained on many projects. Fast-track often uses conventional concrete paving materials and processes, but key changes can significantly accelerate construction. These key components include material modifications, and changes in worker responsibilities, construction staging, pavement joint construction, and opening-to-traffic criteria that can be made to accelerate a concrete paving project.

7. Eisenmann, J. and G. Leykauf. 1990. "Effect of Paving Temperatures on Pavement Performance." *Proceedings, Second International Workshop on the Theoretical Design of Concrete Pavements.* Sigüenza, Spain.

Field tests at have proved that the temperature development in concrete immediately after paving is of great importance on slab curling. Placing the concrete on warm, sunny days causes an upward curling due to a zero-stress temperature differential through the slab. That is, the no-curl condition (flat slab) is not reached at a zero temperature differential, but rather at a high positive temperature gradient. This construction-induced curling can have a significant effect on pavement performance. Wet curling lowers the zero-stress temperature at the surface and counteracts this upward curling.

8. Gress, D. 1997. *Early Distress in Concrete Pavements.* FHWA-SA-97-045. Federal Highway Administration, Washington, DC.

Concerns over concrete roadways exhibiting distress at ages earlier than expected led to the initiation of a study to examine several concrete pavements located throughout the Midwest. The goals were to reach a consensus as to the mechanisms of the distress and to recommend corrective actions to prevent future occurrences until additional research could be conducted. Primary materials-related distress mechanisms identified include freeze–thaw deterioration, sulfate attack, ASR, and alkali–carbonate reactivity. Primary construction related distresses include poor consolidation and material segregation.

Specific recommendations include advice on returning to the basic methods of concrete construction and avoiding the use of materials known to contribute to early distress.

9. Grove, J. D. and K. B. Jones. 1996. *Fast Track Basics*. Materials for the New Millennium Proceedings of the Materials Engineering Conference. Volume 1. American Society of Civil Engineers, New York, NY.

   Fast track PCC paving is a concept, not merely a concrete mixture. There are four major aspects that in combination, achieve rapid strength gain and early opening time: cement type, cement content, curing method, and opening strength criteria each must be considered. The actual time of opening can now be designed to match the requirements of a project or portion of a project. Because early strength comes with a cost, to use only those elements that are needed to meet the project requirements will allow the most economical solution and still achieve the desired goals. Fast track concrete can be designed to achieve opening strength in less than 6 hours, 2 or more days, or almost anything in between. This paper discusses the various aspects of fast track concrete paving and offers some examples of opening times based on various combinations of the elements.

10. Hossain, M. and J. B. Wojakowski. 1994. "Construction and Performance of a Fast-Track Concrete Pavement in Kansas." *Transportation Research Record 1465*. Transportation Research Board, Washington, DC.

    The fast-track or high early-strength concrete offers the opportunity of taking advantage of higher early strength gain in a smaller time for construction or rehabilitation of high-volume roads and city streets serving businesses. A section of fast-track concrete pavement built in an urban setting in Manhattan, KS, had mixture design developed using a special Type-III cement and three different types of locally available aggregates. A recent visual survey indicates that the longitudinal surface texture of the pavement is showing wear. This might have been due to the grinding action of the sand particles on the pavement surface applied during the winter months under the traffic load. Overall, the performance of this pavement is excellent.

11. Hossain, M. and J. B. Wojakowski. 1996. "Effect of Concrete Mix Consolidation on Joint Faulting and Load Transfer Efficiency." *Transportation Research Record 1544*. Transportation Research Board, Washington, DC.

    Six jointed reinforced concrete pavement and one jointed plain concrete pavement test sections on US 69 in Miami County, KS, and constructed in 1979 have been surveyed annually for faulting for the past 9 years. FWD tests were conducted in 1995 to assess the load transfer efficiency of the joints. The results show that, in general, as the original concrete density increases due to improved consolidation, the rate of increase of the joint fault depth decreases at doweled joints at a given pavement age. The occurrence of joint faulting is much more severe when load transfer devices are not present; this was observed even for the pavement section built on a nonerodible subbase. Improved consolidation sometimes appears to help improve load transfer, resulting in a lower rate of faulting. Thus, the mandatory density requirement of 98 percent rodded unit weight, which has been in effect since 1980, has undoubtedly led to better joint performance for concrete pavements in Kansas.

12. McCullough, B. F. and T. Dossey. 1999. *Considerations for High Performance Concrete Paving: Recommendations from 20 years of Field Experience in Texas*. Preprint No. 991462. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

This paper presents recommendations for high performance concrete paving practice drawn from 20 years of designing and monitoring the performance of continuously reinforced concrete (CRC) pavements in Texas. Performance indicators used include crack spacing distribution, crack width, crack randomness, delamination spalling, and vertical distribution of tensile strength. Variables studied include aggregate type (limestone or siliceous gravel), aggregate blending, placement season, placement time of day, placement above 32 °C (90 °F), use of crack initiators, use of skewed transverse steel, evaporation rate, percent steel reinforcement, and steel bar diameter. The variables studied are ranked in the order they affected performance to identify which are significant and can be controlled in the design and construction phases.

13. McCullough, B. F. and R. O. Rasmussen. 1999. Fast-Track Paving: Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays, Volume 1. Final Report. Report No. FHWA-RD-98-167. Federal Highway Administration, McLean, VA.

This research focuses on modeling early-age behavior of both concrete pavements and bonded concrete overlays (BCOs) subjected to stresses from moisture and thermal changes. It includes the development of a two-part, versatile, comprehensive set of guidelines that provide direction in the proper selection of design and construction variables to minimize early-age damage to the PCC pavement and BCO. The first part of these guidelines is qualitative in nature and is based upon the results of this effort, past experience, and engineering judgment. The guidelines are intended to identify design and construction inputs that are most likely to lead to good behavior during the early-age period.

14. McCullough, B. F. and R. O. Rasmussen. 1999. Fast-Track Paving: Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays, Volume 2. HIPERPAV® User's Manual. Report No. FHWA-RD-98-168. Federal Highway Administration, McLean, VA.

This research focuses on modeling early-age behavior of both concrete pavements and BCOs subjected to stresses from moisture and thermal changes. It includes the development of a two-part, versatile, comprehensive set of guidelines that provide direction in the proper selection of design and construction variables to minimize early-age damage to the PCC pavement and BCO. The first part of these guidelines is qualitative in nature and is based upon the results of this effort, past experience, and engineering judgment, and intended to identify design and construction inputs that are most likely to lead to good behavior during the early-age period. The end product from this research is a comprehensive software package termed HIgh PERformance PAVing (HIPERPAV). This package, which incorporates the complex models developed, can be used as a stand-alone product to verify the overall effect of specific combinations of design, construction, and environmental inputs on early-age behavior of a PCC pavement and BCO.

15. Nam, C. H. and C. B. Tatum. 1992. "Government-Industry Cooperation: Fast-Track Concrete Innovation." *Journal of Construction Engineering & Management-ASCE*. Volume 118, Issue 3, September.

Technology for fast-cure concrete, used in other construction areas in the past, recently found application in highway pavement. During July 1986, a 10-cm (4-inch) concrete overlay was applied to 11.2 km (7 mi) of U.S. Highway 71 north of Storm Lake, IA. The project team developed a new type of concrete mix that cures fast to allow traffic onto the road in only 24 hours. This innovation provides an example of cooperative government-industry effort and successful procurement policies implemented by a government agency. Initiated as a response to competition from other materials, the development of fast-track concrete illustrates the process and involvement of many organizations in product innovation. This paper describes the development of paving technologies in Iowa, the formation of an industry association, and the innovation process to bring about technical improvements and cost competitiveness. The implications section describes elements of government policy to foster an increased rate of innovation in U.S. public construction (including supporting increased technical capability) using demonstration projects and encouraging competing technologies.

16. Okamoto, P. A., P. J. Nussbaum, K. D. Smith, M. I. Darter, T. P. Wilson, C. L. Wu, and S. D. Tayabji. 1994. *Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements, Volume I—Final Report*. FHWA-RD-91-079. Federal Highway Administration, Washington, DC.

A study with the objectives of providing guidelines for (1) timing of contraction joint sawcutting to avert uncontrolled pavement cracking and (2) early loading of pavements by construction traffic has been conducted. This volume presents the results of the research. A laboratory study of early age (4 to 24 hours) and early pavement loading (1 to 28 days) concrete strength properties for a range of highway concrete mixes was made. Sawcutting tests were made to determine earliest contraction joint sawcutting. Earliest sawcut timing was correlated on the basis of sawcut ratings to concrete strength properties and nondestructive test results that can be used for determining earliest sawcutting time. The latest sawcutting time was targeted on the basis of buildup of restraint stresses attributable to slab cooling. Concrete pavement placement and joint sawcutting were observed at three highway construction sites to verify test results. Guidelines for sawcut timing are presented to facilitate construction site decision-making based on nondestructive test methods. Early loading by construction traffic was analyzed using the ILLI-SLAB finite element program, and guidelines developed for assessing damage done by early loading.

17. Okamoto, P. A., P. J. Nussbaum, K. D. Smith, M. I. Darter, T. P. Wilson, C. L. Wu, and S. D. Tayabji. 1994. *Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements, Volume II—Appendix*. FHWA-RD-91-080. Federal Highway Administration, Washington, DC.

This volume presents supporting documentation for the research study described in item 16, including a summary of early age (4 to 24 hours) laboratory test data, a summary of early age (1 to 28 days) laboratory test data, a summary of the laboratory sawing strip data, a compilation of the field joint sawcutting data, a summary of the field load testing data, and the state-of-the-art review of concrete sawcutting activities.

18. Papaleontiou, C. G., M. D. Loeffler, A. H. Meyer, and D. W. Fowler. 1986. *The Effectiveness of Texas Membrane Curing Compound Quality and Application Requirements*. FHWA/TX-87/05+427-1F. Texas State Department of Highways and Public Transportation, Austin, TX.

    This report discusses the relative merits of Texas specifications (Tex-219-F) and ASTM specifications (ASTM C 156-80) for the testing of moisture retention by liquid membrane-forming curing compounds. Preliminary work toward the development of a new moisture retention test to replace Tex-219-F and/or ASTM C 156-80 is also outlined, in addition to suggestions for continuing research in this direction. Recommendations are presented with regard to the 6-month curing compound shelf life in effect at the time of the study and the possibility of extending this shelf life. Research is also reported dealing with the effects of altering application rates and patterns on moisture retention. Finally, the use of optical reflectance as a measure of application rate is examined.

19. Rasmussen, R. O., B. F. McCullough, J. M. Ruiz, and P. J. Kim. 1999. Fast Track Paving: Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays, Volume III: Addendum to the HiperPav User's Manual. Report No. FHWA-RD-99-200. Federal Highway Administration, McLean, VA.

    This is an addendum to the *User's Manual* of the comprehensive software package termed HIPERPAV. This package, which incorporated the complex models developed, can be used as a stand-alone product to verify the overall effect of specific combinations of design, construction, and environmental inputs on early-age behavior of a PCC pavement and BCO. This report provides color illustrations and an update of information in the User's Manual.

20. Senadheera, S. P. and D. G. Zollinger. 1996. *Influence of Coarse Aggregate in Portland Cement Concrete on Spalling of Concrete Pavements*. FHWA/TX-97/1244-11. Texas Department of Transportation, Austin, TX.

    Spalling is a form of distress in concrete pavements. However, the current state of knowledge and the available prediction models on spalling indicate a lack of understanding of the spalling mechanism. This research attempts to fill the need to develop a framework to incorporate spalling in the design of concrete pavements based on a mechanistic approach. The report presents results from a comprehensive field survey on spalling in concrete pavements in Texas. Based on the results from this field survey, a mechanism for spalling is proposed. According to this mechanism, spalling is the culmination of damage initiated as delaminations early in the life of pavements. The development of delaminations is related to the concrete mix design and conditions at the time of paving, including ambient conditions and the method of curing. The delaminations are extended into spalls as a result of fatigue damage induced by traffic and temperature fluctuations in the pavement. Early-age analysis of concrete pavements was performed using a finite element program developed to predict stresses in the pavement caused by shrinkage. Results from the analysis indicated that a high level of stress sufficient to create delaminations might be generated at a very early age. The coarse aggregate type in concrete was noted to have a significant effect on the level of spalling. Based on results from laboratory studies, a mechanism on how the aggregate type influences spalling is proposed. This proposed spalling mechanism is included in a framework to incorporate spalling in the design of concrete pavements.

21. Wojakowski, J. B. 1998. *High Performance Concrete Pavement*. Report No. FHWA-KS98-2. Kansas Department of Transportation (Topeka), Bureau of Materials and Research. Kansas City, KS.

    PCC pavement of especially high quality became an area of interest in the early 1990s and precipitated a tour by representatives of industry and government to observe European construction practices. Following the tour, the FHWA developed a research program to encourage and aid States in constructing high-performance concrete pavement.

Costs

1. Cole, L. W. and M. J. Hall. 1996. "Relative Costs of Various Concrete Pavement Features." *Transportation Research Record 1574*. Transportation Research Board, Washington, DC.

   The design and construction of PCC pavement involves the selection, specification, and construction of a number of concrete pavement features that can significantly affect pavement construction costs. In this study, the relative effect on pavement construction cost of several concrete pavement features was investigated, including concrete pavement thickness, foundation, shoulders, cross section thickness variation (trapezoidal section), joint spacing, transverse joint load transfer, and transverse joint sealant. Careful consideration and study should be given the cost effects of various features when designing and specifying concrete pavement. The ideal pavement design is one that selects the least costly pavement section that will perform to the expected level over the life of the facility. The least costly pavement section is that with the least life-cycle costs.

2. Federal Highway Administration (FHWA). 1998. *Life-Cycle Cost Analysis In Pavement Design Participant's Notebook. Demonstration Project No. 115*. Report No. FHWA-SA-98-040. Federal Highway Administration, Washington, DC.

   This participant's notebook was developed by FHWA staff to compliment a 2-day workshop on life-cycle cost analysis (LCCA) in pavement design. This workshop will be of interest to State highway agency personnel responsible for conducting and/or reviewing pavement design LCCAs. The FHWA Office of Engineering, Pavement Division, in cooperation with the Office of Technology Applications, offers LCCA technical support through Demonstration Project No. 115, "Probabilistic LCCA in Pavement Design" (DP-115). DP-115 is a free 2-day workshop that demonstrates best practices in performing LCCAs for pavement design. This workshop is available, upon request, to State highway agencies. The participant's notebook is presented in 13 modules and a set of class exercises (with solutions).

3. Goldbaum, J. 2000. *Life Cycle Cost Analysis State-of-the-Practice*. Final Report. Report No. CDOT-R1-R-00-3. Colorado Department of Transportation (CODOT), Aurora, CO.

   This report provides an outline for the engineer seeking to conduct a LCCA in pavement design and selection. The guidance, recommendations, and default values provided here were collected from 10 years of paving projects. Most of these projects were constructed or rehabilitated in the mid-1980s in order to evaluate the current design and construction practices in Colorado. At this time, the Colorado DOT uses a deterministic approach to the LCCA and is researching the move toward a probabilistic LCCA.

4. Harrison, R. *Influence of Road-Surface Roughness on Vehicle Operating Costs. Reviewing the Evidence From Developing Countries*. American Society for Testing and Materials Special Technical Publication, STP 1031. Published by ASTM, Customer Service Department, Philadelphia, PA.

   The paper describes some features of the vehicle operating cost/road-roughness relationships reported in major international research studies from 1972 to 1986. This research is characterized by the use of road-surface roughness devices, fleets of experimental vehicles to measure fuel consumption, large-scale surveys of vehicle operators, improvements to modeling speed and fuel, and the development of user-friendly economic evaluation models. All studies report significant effects on operating costs following changes in surface roughness. The issues of calibration, new vehicle technologies, and extrapolation of study results are then discussed. The paper concludes by characterizing the main features of the research studies and shows the rise in operating costs attendant on allowing surface conditions from deteriorating to high levels of roughness.

5. Packard, R. G. 1994. "Pavement Costs and Quality." *Concrete International*. Volume 16, Issue 8, August.

   The true value of any pavement is determined by factors such as quality, initial cost and cost of upkeep, service life, and quality of service life. The choice between pavement types of equivalent design is based not solely on initial cost, but is almost always dependent on subsequent costs and length of service life. Thus, recent information and references on these topics were summarized and are presented here.

6. PIARC Technical Committee on Concrete Roads. 2000. *Whole Life Costing of Roads: Concrete Pavements*. Permanent International Association of Road Congresses, La Defense, Cedex, France.

   Whole-life costs (WLC), or life-cycle costs, of a road pavement are understood to represent the costs incurred during the lifetime of a road and may include costs to the road administration from planning the road to its full replacement. In practice, however, WLC are limited to the costs incurred by construction and maintenance of the road over a specified period. The contents of this report are as follows: (1) Background; (2) Introduction; (3) Costs; (4) Functions; (5) Material Characteristics; (6) Road Deterioration Models; (7) Whole-Life Costing; (8) Value of Tied Up Capital/Interest Rate; (9) Probabilistic Approaches; (10) The Use of WLC Models, Examples; (11) Future Development; and (12) Conclusions.

7. Walls J. III and M. R. Smith. 1998. *Life-Cycle Cost Analysis In Pavement Design—Interim Technical Bulletin*. Report No. FHWA-SA-98-079. Federal Highway Administration, Washington, DC.

This bulletin recommends procedures for conducting LCCA of pavements, provides detailed procedures to determine work zone user costs, and introduces a probabilistic approach to account for the uncertainty associated with LCCA inputs. The bulletin begins with a discussion of the broad fundamental principles involved in an LCCA. It discusses input parameters and presents simple examples of traditional LCCA in a pavement design setting. It discusses the variability and inherent uncertainty associated with input parameters, and provides recommendation on acceptable ranges for the value of time as well as discount rates. It explores the use of sensitivity analysis in traditional LCCA approaches. User costs are a combination of delay, vehicle operating costs, and crash costs. Each of these cost components is explored and procedures are presented to determine their value. Given the power and sophistication of today's computers and software, simulation techniques (such as Monte Carlo) are recommended for incorporating variability associated with LCCA inputs into final results.

Cross Section

1. Crovetti, J. A. 1999. *Cost Effective Concrete Pavement Cross Sections*. Report No. WI/SPR12-19. WisDOT, Madison, WI.

The current pavement selection policy of WisDOT limits the design alternatives for PCC pavements and inhibits the designer's ability to select cross sections deviating from uniform slab thickness with doweled transverse joints. Currently, uniform slab thicknesses and conventional joint load transfer devices are incorporated into the design based on the heavy truck traffic in the outer lane. While this strategy provides for adequate pavement structure in this truck lane to limit faulting and slab cracking to tolerable levels, there is a potential for over-design in other traffic lanes, which receive significantly lower ESAL applications over the service life of the pavement.

2. Rhodes, A. H. 1997. "Highway Pavements: Use of Tapered Sections to Extend Design Life." *Proceedings of the Institution of Civil Engineers-Transport.* Volume 123, Issue 1, February.

This paper begins by examining the lateral and longitudinal distribution of traffic loading applied to highway pavements. The effects of crossfall, superelevation, lane distribution, direction, longitudinal profile, acceleration, and deceleration are considered. A tapered cross section is proposed as the most appropriate design to achieve the best value for money. Following a desk-top study, the author, in collaboration with Lancashire County Council, designed and constructed a full-scale trial on the A584 road bypassing Freckleton village. This was to be the first tapered section pavement in the UK. There were three conventional control sections and three corresponding tapered sections. Following deflectograph and impulse radar surveys, it was shown that it was possible to double the design life for a pavement by redistributing the base material to form a tapered section at no extra cost.

Design and Performance

1. American Association of State Highway and Transportation Officials (AASHTO). 1993. *AASHTO Guide for Design of Pavement Structures*. AASHTO, Washington, DC.

   This document presents the current AASHTO procedures for the design of flexible and rigid pavement structures. It contains the major revisions to the AASHTO new pavement design procedure that were adopted in 1986 and which incorporate such new elements as design reliability, subgrade resilient modulus, environmental considerations, drainage provisions, life-cycle costing, low volume road design, pavement management concepts, and rehabilitation guidelines. New pavement overlay design procedures adopted in 1993 are also contained in the guide.

2. American Association of State Highway and Transportation Officials (AASHTO). 1998. *Supplement to the AASHTO Guide for Design of Pavement Structures, Part II—Rigid Pavement Design and Rigid Pavement Joint Design*. American Association of State Highway and Transportation Officials, Washington, DC.

   This supplement to the 1993 *AASHTO Design Guide* includes alternative design procedures that can be used in place of or in conjunction with Part II, Section 3.2 "Rigid Joint Design" and Section 3.3 "Rigid Pavement Joint Design." The development of these alternative design procedures was initiated under National Cooperative Highway Research Program (NCHRP) Project 1-30 and continued under an FHWA-sponsored study using LTPP performance data. Improved guidance on the selection of appropriate k-value for use in design, a rederived performance model representative of critical loading conditions and incorporating joint spacing and associated curling, and improved guidelines on joint design details are found in the new procedure. A detailed example is provided, along with recommended design faulting check for doweled and nondoweled pavements.

3. Ardani, A., N. Suthahar, and D. A. Morian. 2000. *Early Evaluation of LTPP Specific Pavement Studies-2, Colorado*. Report No. CDOT-DTD-R-2000-2. Colorado Department of Transportation, Denver, CO.

   This report presents the early results of the Specific Pavement Study (SPS)-2 experiment, "Strategic Study of Structural Factors for Rigid Pavements" documenting construction details of 13 different test sections with varying structural characteristics. The SPS-2 experiment was developed as a coordinated national experiment to address the effects of various strategic environmental and structural factors on the performance of rigid pavements. The factors studied under this experiment included concrete thickness, concrete strength, base type, lane width, drainage and environmental factors such as temperature, moisture and soil type. This paper discusses the performance of these test sections after being in service for 4 years. The results are based on monitoring data collected by the LTPP.

4. Armaghani, J. M. 1993. "Factors Affecting Performance of Concrete Pavements." *Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

   This paper discusses several aspects of concrete pavement design and construction and their relationship to performance, with emphasis on the effects of temperature displacements on performance of pavement slabs. Experience with concrete pavements in Florida indicates that doweled joints perform better than undoweled joints, skewed transverse joints did not demonstrate advantages over perpendicular joints, slab lengths between 4.6 and 5.5 m (15 and 18 ft) experienced less cracking, joint sealing is beneficial but is not a substitute for good drainage, preformed compression sealants last considerably longer than silicone or other joint sealing materials, slab widening reduces cracking, the optimal base stiffness is one that provides a k value between about 5.4 and 10.9 MPa/m (200 and 400 pci) (lower k values resulting in higher load-related slab stresses, and higher k values resulting in higher curling-related slab stresses), water curing is preferable to curing compound and blanket curing in terms of moisture retention and temperature control, and joints should be sawed within 1.5 hours of slab finishing (sooner in hot weather and/or in accelerated paving).

5. Ayton, G. P. 1993. "Concrete Highway Pavements in Australia." *Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

   Since 1975, the Roads and Traffic Authority of New South Wales has promoted the increased use of concrete pavements throughout the State. Approximately 1300 lane-km of concrete pavement had been constructed as of mid-1993. The Australian industry expanded rapidly in an environment that is geographically remote from overseas areas of long-standing experience such as the United States and western Europe. The adoption of this relatively new technology in the Australian context attracted close attention from both the media and the traveling public. It has therefore been a major priority to implement and refine good design and construction practices as rapidly as possible. Current Australian practice reflects a blending of local experience with that from leading overseas authorities. This paper summarizes local developments in Australian concrete pavement technology since the mid-1970s. Recent indications point to a likely growth in concrete pavement construction throughout the southeast Asian region, and recent Australian experience would appear to be relevant.

6. Bendaña, L. J., D. McAuliffe, and W. S. Yang. 1994. "Mechanistic-Empirical Rigid Pavement Design for New York State." *Transportation Research Record 1449.* Transportation Research Board, Washington, DC.

   In 1993, New York published a new *Thickness Design Manual for New and Reconstructed Pavements* based on the 1986 *AASHTO Design Guide*. The AASHTO equation for rigid pavement performance was calibrated with performance data for 225-mm rigid pavements in New York, and the calibrated equation was then used to design rigid pavements. Because New York does not have experience with thicknesses greater than 225 mm, the modified AASHTO equation could not be verified for thicker pavements. The development of a mechanistic-empirical (M-E) design procedure for verifying the designs presented in the new thickness manual is described in this paper. First, a nondimensional fatigue model was established on the basis of New York's past pavement performance, environmental conditions, and traffic loadings. The study was then extended to develop design curves for thicknesses of 225, 250, 275, 300, and 325 mm (9, 10, 11, 12, and 13 inches) (5-m (15-ft) slab lengths for 225- to 275-mm (9- to 11-inch) thicknesses and 5.5-m (18-ft) slab lengths for 300- to 325-mm (12- to 13-inch) thicknesses). Finally, the M-E design curve was compared with the modified AASHTO equation. The results indicate that for thicknesses greater than 275 mm (11 inches), AASHTO predicts up to 40 percent more equivalent single-axle loads than the M-E approach.

7. Burnham, T. R. and W. M. Pirkl. 1997. *Application of Empirical and Mechanistic-Empirical Pavement Design Procedures to Mn/Road Concrete Pavement Test Sections*. Final Report. Report No. MN-RC-97-14. Minnesota Department of Transportation, Maplewood, MN.

   Current pavement design procedures are based principally on empirical approaches. The current trend toward developing more mechanistic-empirical pavement design methods led Minnesota to develop the Minnesota Road Research Project (Mn/ROAD), a long-term pavement testing facility. The project consists of 40 heavily instrumented test sections, 14 of which are jointed plain concrete (JPC) designs. Mn/ROAD researchers determine the predicted lives of the concrete test sections by applying design and as-built data to three currently accepted concrete pavement design methods: Minnesota Department of Transportation's rigid pavement design guidelines; *AASHTO Guide for Design of Pavement Structures* 1993; and the Portland Cement Association (PCA) *Thickness Design for Concrete Highway and Street Pavements* (1984). The analysis began with determining the applicable as-built parameter values for each respective design method. Applying the as-built parameters to the three methods resulted in widely varied predictions of pavement life. For the 1993 AASHTO design method, reliability levels of 50 percent and 95 percent were applied for comparison. An experimental procedure for converting PCA method fatigue and erosion results to AASHTO type CESALs (concrete pavement equivalent single axle loads) demonstrated unsuitability. Validation of the predictions presented will occur as the test cells reach their terminal serviceability.

8. California Department of Transportation (Caltrans). 1995. *Design Criteria for PCCP*. Design Information Bulletin No. 80 (Draft). California Department of Transportation, Sacramento, CA.

   This document implements changes to the current design guidance contained in Chapter 600 of the California Highway Design Manual for PCCP on the state highway system. Major changes contained in the bulletin include the use of lean concrete bases for heavy traffic volumes, the use of perpendicular joints while maintaining variable joint spacing of 3.6, 4.6, 4.0, and 4.3 m (12, 15, 13, and 14 ft), the use of tied and sealed longitudinal joints, the use of epoxy-coated dowel bars at transverse joints, and tied concrete shoulders of the same structural thickness as the mainline pavement.

9. Canada, M., C. Jofre, and I. Picazo. 1997. "Influence of the Length of Slabs on Performance of All Concrete Pavements." *XIIIth World Meeting of the International Road Federation*, Toronto, Canada. June 16-20, 1997. International Road Federation, Washington, DC.

The paper compares the performance of concrete pavements of two stretches of autoroute, the Albacete and La Roda bypasses, located in central Spain. Both pavements can be considered carbon copies of each other because they are close to each other; they are almost the same age; they support almost the same amount of traffic; and they share the same project features. The most significant difference is the length of the concrete slabs: between 3.7 and 5 meters (12 and 16 feet) for the slabs on the Albacete and 3.4 and 4.5 meters (11 and 14.7 feet) for La Roda. This major separation of joints may explain the very different performance of each stretch of road; whereas the Albacete segment has over 25-percent cracked slabs, the figure for La Roda is 6 percent.

10. Chen, H. J., L. J. Bendaña, D. E. McAuliffe, and R. L. Gemme. 1996. "Updating Pavement Design Procedures for New York State." *Transportation Research Record 1539*. Transportation Research Board, Washington, DC.

New York's effort in adapting concepts from AASHTO's pavement design guide as a basis for a revised state design procedure for thickness of new and reconstructed pavements is summarized. The rationale for this revised procedure was to design more durable pavements and reduce life-cycle costs. New York's past pavement design practice and the background for the revisions are briefly described. A sensitivity analysis was conducted to identify how AASHTO design variables affect pavement thickness. Past performance of selected New York pavements was also studied. The rationale is discussed for determination of appropriate design variables, based on the sensitivity analysis, performance studies, and reviews of past and current practice. Also described is the justification of other design features, such as 50-year design life, granular subgrade, permeable base, edge drains, shorter slabs, maximum and minimum pavement thicknesses, and new dowel and tiebar designs. Development and implementation of New York's new AASHTO-based thickness design procedure are major steps toward accomplishing the goals of building longer lasting pavements and reducing life-cycle costs.

11. Cho, Y. H., T. Dossey, and B. F. McCullough. 1997. "Early Age Performance of Continuously Reinforced Concrete Pavement with Different Types of Aggregate." *Transportation Research Record 1568*. Transportation Research Board, Washington, DC.

The effect of coarse aggregate on pavement performance has been attributed to the volume of aggregate used in pavement construction. The different patterns of crack development for limestone (LS) and siliceous river gravel (SRG) are a typical example of aggregate-induced variable performance in continuously reinforced concrete pavement (CRCP). Laboratory and field testing was conducted to evaluate these aggregate types for suitability in CRCP construction. CRCP sections constructed with LS aggregates displayed better performance than those containing SRG. Surprisingly, CRCP sections containing an LS–SRG blend showed worse performance than did those containing SRG only.

12. Croney, D. and P. Croney. 1998. *Design and Performance of Road Pavements*. Third Edition. McGraw-Hill. New York, NY.

This book presents a comprehensive look at the design, construction, and performance of pavements. It examines the principles of pavement design and describes in detail the critical data needed for a reliable pavement design: materials characterization, traffic estimation, climatic factors, and geological and subgrade parameters. Drawing upon experimental roads constructed in both the United States and in Great Britain, it presents valuable information on the performance of different pavement designs under different traffic and environmental loading. Finally, current design procedures for both flexible and rigid pavements are presented, along with descriptions of the movement toward more mechanistic-based procedures.

13. Crovetti, J. A. and S. Owusu-Ababio. 1999. *Investigation of Feasible Pavement Design Alternatives for WisDOT*. Final Report. Report No. WI/SPR-15-99, WisDOT Study # 94-13. Wisconsin Department of Transportation, Madison, WI.

The current pavement design and selection process of the Wisconsin Department of Transportation (WisDOT) for all new pavements or reconstructions of existing pavement structures provides for the design of one asphaltic concrete (AC) and one portland cement concrete (PCC) pavement alternative. Life-cycle cost analyses are then used to determine the preferred alternative for construction. Previous restrictions in the WisDOT pavement selection process have essentially excluded the construction of thick AC (AC thickness > 150 mm (6 inch)) and thin PCC (PCC thickness < 225 mm (9 inch)) pavements and thus the validity of current life-cycle cost inputs for these pavement types is questionable. This report presents a performance analysis of existing thick AC and thin PCC pavements constructed in and around Wisconsin. The performance trends developed indicate that current design assumptions utilized by WisDOT and related to the expected service life to first rehabilitation of AC and PCC pavements may also be used for thick AC and thin PCC pavements.

14. Darter, M. I. 1977. Design of a Zero-Maintenance Plain Jointed Concrete Pavement, Volume I—Development of Design Procedures. Report No. FHWA-RD-77-111. Federal Highway Administration, Washington, DC.

Comprehensive procedures for the structural design of "zero-maintenance" JPCP for heavily trafficked roadways are presented. The term "zero-maintenance" refers to the structural adequacy of the pavement lanes and shoulder. The design procedures are based upon results from long-term field studies, comprehensive mechanistic analyses, and laboratory studies, the basis for which is provided in this report. Both a serviceability-performance analysis and a concrete fatigue analysis are used in the structural design, and additional procedures are included for the design of the subbase, shoulders, joints, and subsurface drainage. Example designs are included with sensitivity and incremental cost analyses.

15. Darter, M. I. and E. J. Barenberg. 1977. *Design of a Zero-Maintenance Plain Jointed Concrete Pavement, Volume II—Design Manual*. Report No. FHWA-RD-77-112. Federal Highway Administration, Washington, DC.

    This is an engineering guide for the design of heavily trafficked JPCP to provide "zero-maintenance" performance over the selected design period. Procedures are included for designing the concrete slab, subbase, shoulders, joints, and subsurface drainage. A computer program (JCP-1) is used to provide serviceability-performance and fatigue data for the structural design of the pavement. The manual includes specific recommendations for obtaining all necessary inputs and for performing the structural design. A detailed design example for a heavily trafficked freeway pavement is provided, including a sensitivity analysis of major design factors.

16. Darter, M. I. and E. J. Barenberg. 1977. "Zero-Maintenance Design for Plain Jointed Concrete Pavements." *Proceedings, International Conference on Concrete Pavement Design*. Purdue University, West Lafayette, IN.

    The development of design procedures for heavily trafficked plain jointed concrete pavements to provide "zero-maintenance" performance is described. "Zero-maintenance" refers only to structural maintenance, such as patching, crack repair, slab replacement, grinding of faults, and overlay. Procedures are included for designing the concrete slab, subbase, shoulders, joints, and subsurface drainage. A computer program (JCP-1) was used to provide serviceability/performance and fatigue data for the structural design of the pavement. These procedures were developed based on nationwide field studies, long-term pavement performance data from in-service pavements, mechanistic analyses, and laboratory studies. A detailed design example for a heavily trafficked freeway pavement is presented, along with a sensitivity analysis of the major design factors.

17. Darter, M. I., J. M. Becker, M. B. Snyder, and R. E. Smith. 1985. *Portland Cement Concrete Pavement Evaluation System—COPES*. NCHRP Report 277. Transportation Research Board, Washington, DC.

    This report describes the development of COPES, which provides a framework and procedures for the systematic collection of historical and field data on the characteristics and performance of in-service PCCP. The first part of the report provides a summary of the development of COPES and demonstrates the potential uses of the data collected under COPES. Data collected from six States were analyzed to illustrate the impact of design features and construction practices on concrete pavement performance. The analyses took the form of regression equations, which, although intended for demonstration purposes, provide insight into the performance of concrete pavements. Detailed appendices present a comprehensive COPES user's manual and the results of the case studies conducted in the six States.

18. Darter, M. I., E. Owusu-Antwi, and R. Ahmad. 1996. "Evaluation of AASHTO Rigid Pavement Design Model Using Long-Term Pavement Performance Data Base." *Transportation Research Record 1525*. Transportation Research Board, Washington, DC.

The AASHTO design guide's rigid pavement equation that is used for thickness design was originally developed in 1960 at the conclusion of the AASHO Road Test. This equation predicts the number of axle loads for a given slab thickness and loss in serviceability. During the past 30 years, the original equation has been extended to include several additional design factors and has been used by many highway agencies for rigid pavement design. Due to the limited inference space of the original road test equation and the subjective nature of the subsequent extensions, there is considerable interest in determining the adequacy of the equation. The availability of the nationwide LTPP data has finally made an overall evaluation possible. The evaluation included determining the adequacy of predicting the number of heavy axle loads required to cause a given loss in serviceability. The results indicate that the original 1960 equation generally overpredicts the number of 18-kip equivalent single-axle load applications for a given loss of serviceability. However, extensions to the original model improve predictions considerably. These results were determined at the 50th percentile. At a higher level of reliability (such as 95 percent), the 1986 AASHTO model provides a conservative design for a majority of the pavement sections. However, several deficiencies remain in the model.

19. Darter, M. I., K. D. Smith, and D. G. Peshkin. 1991. "Field-Calibrated Mechanistic Empirical Models for Jointed Concrete Pavements." *Transportation Research Record 1307*. Transportation Research Board, Washington, DC.

Field-calibrated, mechanistic-empirical models have been developed for key performance indicators of jointed concrete pavements. Performance data from nearly 500 inservice pavements were used along with mechanistic and empirical variables to develop improved prediction models for joint faulting, slab cracking, joint spalling, and current serviceability rating. The models should prove valuable in checking the performance capabilities of various pavement designs determined by other means and in determining the relative impact of different design variables on concrete pavement performance.

20. Darter, M. I., H. Von Quintus, Y. J. Jiang, E. B. Owusu-Antwi, and B. M. Killingsworth. 1997. *Catalog of Recommended Pavement Design Features*. Final Report, NCHRP Project 1-32. Transportation Research Board, Washington, DC.

Following the example of design catalog development by several European countries, this study's major product is the catalog of "good practice" recommendations for design features of highway pavements. Design cells are defined by three main "site conditions": traffic loadings, subgrade support, and climate. Within each cell, recommendations are given for the pavement type, layer thicknesses, materials, joint design, reinforcement design, drainage design, and other features related to performance. The document is not intended for use as a pavement design manual or for project-level pavement design.

21. Dossey, T., S. Easley, and B. F. McCullough. 1996. "Methodology for Estimating Remaining Life of Continuously Reinforced Concrete Pavements." *Transportation Research Record 1525*. Transportation Research Board, Washington, DC.

A methodology was developed for estimating the remaining life of a nonoverlaid continuously reinforced concrete pavement. The models presented use several key predictors: the early-age crack distribution pattern in the concrete caused by volumetric changes, the coarse aggregate type used, and the presence or absence of a swelling subgrade that will accelerate the rate of failure development in later life due to dynamic loadings. Considering these factors, the models can accurately estimate the additional 18-kip ESALs that will take the pavement from its current condition in terms of failures per mile to a user-defined "failure threshold. Using traffic models developed in another study, the number of additional ESALs to failure can be translated into time to failure for planning purposes.

22. Eisenmann, J. and G. Leykauf. 1990. "Simplified Calculation Method of Slab Curling Caused by Surface Shrinkage." *Second International Workshop on the Theoretical Design of Concrete Pavements*. Sigüenza, Spain.

In the simplified calculation method (slab structure replaced by beam on a rigid support) presented in this paper, the moment due to surface shrinkage is compared with the fictitious equivalent moment caused by a linear negative temperature gradient. This analogy allows the prediction of the critical slab length for which the point support at the slab center changes to continuous support with rising slab length, caused by increasing dead weight of the slab. Then the upward shrinkage curling can be calculated using the well-known equations for curling due to a linear temperature gradient, in which the fictitious gradient is replaced by terms derived from the mentioned analogy. Results calculated for different slab thicknesses agree reasonably well with in situ curling measurements on young concrete slabs. Upward curling due to shrinkage can be reduced gradually by increasing slab thickness.

23. ERES Consultants, Inc. 1992. *Concrete Pavement Design Manual*. FHWA-HI-92-015. Federal Highway Administration, Washington, DC.

This manual presents design considerations for concrete pavements. It describes initial considerations in the pavement design process, including subgrade characterization, paving materials characterization, traffic loading considerations, drainage design elements, and design reliability. It presents the AASHTO rigid pavement design procedure and follows up with an introduction to mechanistic-based design concepts. It also presents recommendations on rigid pavement design features. The manual concludes with a summary of overlay rehabilitation methods for rigid pavements.

24. Federal Highway Administration (FHWA). 1990. *Continuously Reinforced Concrete Pavement*. FHWA Technical Advisory T 5080.14. Federal Highway Administration, Washington, DC.

This Technical Advisory outlines recommended practices for the design, construction, and repair of continuously reinforced concrete pavements. It gives recommendations on longitudinal steel reinforcing requirements (including bar sizes, location, placement), base and subbase requirements, joint design guidelines (construction joints, longitudinal joints, terminal joints), and on construction and placement activities.

25. Federal Highway Administration (FHWA). 1992. *Report on the 1992 U.S. Tour of European Concrete Highways.* FHWA-SA-93-012. Federal Highway Administration, Washington, D.C.

    The U.S. TECH Study Tour traveled in France, Austria, Germany, the Netherlands, and Belgium, and heard presentations from Spain, Portugal, Switzerland, and Italy. Concrete pavements are built for heavier loads and longer lives (30 to 40 years) than in the United States. The slab thicknesses generally do not exceed those constructed in the United States. Other design features such as widened slabs, trapezoidal cross sections, thick granular subbases, and highly durable concrete mixes are used to achieve longer performance lives. High population densities make tire/road noise an important environmental issue in Europe; considerable research in development of noise-reducing finishes and textures has been done in several countries. Other European concrete pavement technology advancements recommended by the study tour group for consideration in the United States are warranties for pavement construction, greater cooperation between government and industry in research and development, innovative toll-road financing, and the use of pavement design catalogs.

26. Federal Highway Administration. (FHWA). 1993. *AASHTO Design Procedures for New Pavements, Participant's Manual*. FHWA-HI-94-023. Federal Highway Administration, Washington, DC.

    This course notebook presents the design procedures found in AASHTO's *1993 Guide for Design of Pavement Structures*. It describes the basis for both the flexible and rigid pavement design procedures, including the assumptions and limitations inherent in each approach. Considerable guidance is provided on developing appropriate inputs for use in the development of pavement designs. Additional design elements are also described for each pavement type, such as layer thickness determination for flexible pavements, and steel and joint design for rigid pavements.

27. Federal Highway Administration (FHWA). 1995. *Pavement Analysis and Design Checks, Participant's Manual*. FHWA-HI-95-021. Federal Highway Administration, Washington, DC.

    This course notebook presents a broad overview on pavement design and analysis procedures. Commonly used design procedures are presented for both flexible and rigid pavements, including AASHTO, Asphalt Institute, and PCA. Basic pavement responses (stresses, strains, and deflections) are also described, and various pavement analysis models and performance prediction models are introduced for both flexible and rigid pavements. The emphasis of the information is on checking the reasonableness of resulting pavement design, with the goal of obtaining reliable, long-lasting pavement designs.

28. Federal Highway Administration (FHWA). 1997. *LTPP Data Analysis: Frequently Asked Questions About Joint Faulting with Answers from LTPP*. LTPP TechBrief. FHWA Report No. FHWA-RD-97-101. Federal Highway Administration, McLean, VA.

This LTPP data analysis was intended to examine, in a practical way, the LTPP database and to identify the site conditions and design features that significantly affect transverse joint faulting. Key products developed as part of this research were (1) answers to frequently asked questions regarding design features and site conditions that lead to "good" (better than expected) and "poor" (worse than expected) performance of jointed concrete pavements relative to joint faulting and (2) guidelines to assist highway agencies on what works and what does not work in the design of transverse joints to control joint faulting. This TechBrief presents key findings of this research.

29. Federal Highway Administration (FHWA). 2000. *Key Findings from LTPP Analysis 1990-1999*. Report No. FHWA-RD-00-085. Federal Highway Administration, Washington, DC.

The LTPP analysis program has addressed a broad array of topics—from field validation of pavement design procedures, to the study of variability in traffic and materials data, to investigating pothole repair techniques. The purpose of this document is to highlight some of the key findings from LTPP analysis studies between 1990 and 1999. These findings have been organized into the following areas: site conditions, structural features, materials, initial roughness, pavement maintenance, pavement rehabilitation, AASHTO design validation, and performance modeling.

30. Forsyth, R. A. 1993. *Pavement Structural Design Practices*. NCHRP Synthesis of Highway Practice 189. Transportation Research Board, Washington, DC.

The structural design of flexible and rigid pavements has evolved from the application of engineering judgment to include a variety of processes. This report describes the various methods for structural pavement design in the United States and in several Canadian provinces. It focuses on the elements intended to provide strength and stiffness to the pavement. It includes a summary of current practice and trends in the design of new pavements and overlays for several elements, including thickness design procedures, layer compositions, drainage treatments, characteristics of materials, mitigation of swelling and frost heave, and assessment of pavement residual strength and condition for overlay design.

31. Frabizzio, M. A. and N. J. Buch. 1999. "Performance of Transverse Cracking in Jointed Concrete Pavements." *Journal of Performance of Constructed Facilities*, Volume 13, Issue 4.

Environmental effects and repetitive traffic applications can produce transverse cracks in jointed concrete pavements. Maintaining adequate aggregate interlock load transfer across these cracks is essential to preserving the functional and structural integrity of these pavements. The objectives of this study were to determine the design parameters that significantly affect transverse cracking and to demonstrate methods available for evaluating cracked pavements. Field data collected from in-service jointed concrete pavements located throughout southern Michigan were used to accomplish these objectives. Joint spacing, coarse aggregate type, shoulder type, and pavement temperature were found to

have significant effects on transverse crack development and/or performance. The surface texture of crack faces was assessed using a promising new test method called volumetric surface texture testing. Volumetric surface texture results provided an indication of the aggregate interlock potential of pavements containing various aggregate types. Three performance parameters capable of mechanistically characterizing crack performance were discussed. A relatively simple procedure was described for determining these parameters and evaluating crack conditions. Field data were also used to demonstrate and validate a voids' analysis procedure. This procedure estimates the potential for loss of support near cracks and joints, thus allowing for proper rehabilitation actions before the manifestation of additional distresses.

32. George, K. P., A. Alsherri, and N. S. Shah. May 1988. "Reliability Analysis of Premium Pavement Design Features." *Journal of Transportation Engineering*. Volume 114, Issue 3.

This study evaluates the special features of premium design guidelines―features not considered in the AASHTO flexible and rigid pavement design procedures. The significance of these features (15 in all for 4 pavement types) was investigated by evaluating pavement performance and design reliability. The researchers used the VESYS III program to evaluate features of the flexible pavements and an algorithm developed in a companion paper for the other three types (composite, jointed plain concrete, and continuously reinforced concrete). This computer program, Reliability Analysis and Performance of Pavements I (RAPP-I), employs Monte Carlo simulation techniques to treat all the design variables probabilistically. The effectiveness of each feature is evaluated by comparing the performance or expected life and reliability of typical pavement sections with and without a premium feature.

33. Gharaibeh, N. G., M. I. Darter, and L. B. Heckel. 1999. *Field Performance of CRCP in Illinois*. Preprint No. 990731. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

This paper reviews the design and performance of continuously reinforced concrete pavement (CRCP) in Illinois, which has built more than 4,267 two-lane km (2,650 miles) of CRCP on the Interstate system since the mid-1950s. CRCP has been constructed on nearly all urban freeways in the Chicago area and has shown excellent performance under severe weather and heavy traffic conditions. The effect of key design and construction parameters on long-term CRCP performance is investigated using a database that was compiled based on field surveys conducted from 1977 to 1994 by the Illinois Department of Transportation (IDOT). Analysis of the data shows the following variables have significant effects on performance: longitudinal reinforcement content (greatest effect of all variables), slab thickness (also very significant), traffic load applications, depth of reinforcement, base type, and D-cracking of concrete. CRCP built with tubes or chairs exhibited overall about the same performance. Experimental field studies in Illinois showed that depth of reinforcement has a large effect on crack width and, eventually, on punchouts. Specifically, the investigation indicated that CRCP sections with a slab 178 mm (7 inches) thick and steel content less than 0.6 percent developed the most structural failures. CRCP sections with a slab 254 mm (10 inches) thick and steel content from 0.7 to 0.8 percent developed the fewest failures. However, all the CRCP sections in this study, regardless of thickness

designs and steel content, have typically carried more traffic than they were designed for and have lasted longer than their design traffic life.

34. Gharaibeh, N. G., M. I. Darter, and L. B. Heckel. 1999b. *Field Performance of Continuously Reinforced Concrete Pavement in Illinois*. Report No. UILU-ENG-99-2005, Transportation Engineering SER-101; FHWA-IL-UI268. Illinois State Department of Transportation, Springfield, IL.

This report reviews the design and performance of CRCP in Illinois, which has built more than 4,267 two-lane km (2,650 miles) of CRCP on the Interstate system since the mid-1950s. CRCP has been constructed on nearly all urban freeways in the Chicago area and has shown excellent performance under severe weather and heavy traffic conditions. The effect of key design and construction parameters on long-term CRCP performance is investigated using a database that was compiled based on field surveys conducted from 1977 to 1994 by the Illinois Department of Transportation (IDOT). Analysis of the data shows the following variables have significant effects on performance: longitudinal reinforcement content (greatest effect of all variables), slab thickness (also very significant), traffic load applications, depth of reinforcement, base type, and D-cracking of concrete. CRCP built with tubes or chairs exhibited overall about the same performance. Experimental field studies in Illinois showed that depth of reinforcement has a large effect on crack width and, eventually, on punchouts.

35. Hadi, M. 1998. "Cost Optimum Design of Rigid Road Pavements." *Transport Proceedings—Conference of the Australian Road Research Board (AARB)*. ARRB Transport Research Ltd, Vermont, Australia.

The design of rigid pavements according to AUSTROADS is a lengthy method. The designer assumes a pavement structure then uses a number of tables and figures to calculate the two governing design criteria, the flexural fatigue of the concrete base and the erosion of the subgrade/subbase. Each of these two criteria needs to be less than 100 percent. Ideally, they need to be at their maximum possible value that is less than 100 percent. Designers would repeat the design if either of the criteria is more than 100 percent, in other words, an unsafe design. However if the criteria are much less than 100 percent, i.e., over-design, most designers would stop after one or two iterations due to the lengthy process involved in calculating the criteria, and due to time limitations. This leads to designs that are safe, but not necessarily economical. This paper presents a formulation for the problem of optimum design of rigid road pavements by defining the objective function, which is the total cost of pavement materials, and all the constraints that influence the design. All these are given in terms of design variables and design parameters. The formulation, including the optimizer, was implemented in a spreadsheet. The optimization problem is used in a parametric study where 1,680 pavement structures are optimally designed.

36. Hall, K. T., M. I. Darter, and C. M. Kuo. 1995. "Improved Methods for Selection of k Value for Concrete Pavement Design." *Transportation Research Record 1505*. Transportation Research Board, Washington, DC.

The results of research conducted to improve guidelines for k-value selection for concrete pavement design are summarized. The research included a review of the evolution of the k-value concepts and methods, a review of k-value results from several field studies, an examination of the AASHTO Guide's k-value methods, and proposed new guidelines for selection of design k values by a variety of methods. The k-value was originally considered a useful and simple parameter for characterizing slab support provided by natural soils of fairly low shear strength. Recognizing that real soils are not true dense liquids, early researchers developed standardized test methods, which provided k values in good agreement with full-size slab deflections. Later, substantially higher k values were attributed to granular and stabilized base layers, based on plate tests on top of bases, although slab tests had shown that such bases did not increase k values. Based on the historical review, review of results from several field studies, and a thorough examination of the k-value methods introduced in the 1986 AASHTO guide, it is recommended that k values be selected for natural soil materials, and that base layers be considered in concrete pavement design in terms of their effect on slab response, rather than their supposed effect on k value. Improved guidelines were developed for determining k value from a variety of methods, including correlations with soil type, soil properties, and other tests; backcalculation methods; and plate-bearing methods. Guidelines also were developed for seasonal adjustment to k and adjustments for embankments and shallow rigid layers.

37. Heinrichs, K. W., M. J. Liu, M. I. Darter, S. H. Carpenter, and A. M. Ioannides. 1989. *Rigid Pavement Analysis and Design*. Report No. FHWA-RD-88-068. Federal Highway Administration, Washington, DC.

    This study was conducted to characterize and compare currently available rigid pavement analysis models and design methods and to develop new rigid pavement designs to be evaluated in full-scale experimental projects. Analysis models investigated included ILLI-SLAB, JSLAB, WESLIQID, WESLAYER, JCS-1, H51, CRCP-2, and RISC. Design methods evaluated included AASHTO, Zero-Maintenance, JCP-1, California DOT, PCA, RPS-3 (Texas DOT), ARBP–CRSI, and Illinois DOT. Based upon the evaluation results, several models and methods are recommended for use in the development of new rigid pavement designs. A set of rigid pavement designs, featuring trapezoidal cross sections, widened PCC slabs, permeable drainage layers, longitudinal edge drains, shorter joint spacing, and tied PCC shoulders, was developed. Guidelines were also developed for joint load transfer design and joint spacing.

38. Huang, Y.H. 1993. *Pavement Analysis and Design*. Prentice Hall, Englewood Cliffs, NJ.

    This textbook presents the theory of pavement design, describing pavement behavior and responses under various loading conditions, and including software available for the computation of stresses, strains, and deflections in both flexible and rigid pavement structures. Basic design elements (traffic loading, material characterization, drainage, and reliability) are presented, followed by a review of current highway pavement design procedures for both flexible and rigid pavements. Pavement overlay design procedures also are described in detail.

39. Illinois Department of Transportation (IDOT). 1995. *Pavement Design Procedures*. Report 95-11. Illinois Department of Transportation, Springfield, IL.

Three new design procedures have been developed for use on local agency projects. The design procedures are based on University of Illinois research documents and prepared under the guidance of a project advisory committee consisting of Federal, State, and local representatives. Information on the use of rigid pavements and on structural thickness determination, load transfer requirements, and reinforcement requirements are provided. A condensation of the IDOT subgrade stability manual is also included to provide guidance on subgrade stability requirements.

40. Ioannides, A. M., C. M. Davis, and C. M. Weber. 1999. *Westergaard Curling Solution Reconsidered*. Preprint No. 990693. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

An in-depth. systematic examination is presented of the effect of temperature gradients on slab-on-grade pavements, whose main objective has been the development of practical design tools for use in a typical engineering office. This has been achieved by a critical reconsideration of the literature, a synthesis of currently available analytical resources, and the implementation of recent technological achievements promulgated in related areas of engineering. Prominent among these are the application of the principles of dimensional analysis, the finite element method, advanced statistical regression analysis, and artificial neural networks (ANN). A number of ANNs have been trained for the curling problem, and in several instances they are found to be more efficient predictive tools than corresponding statistical regression equations. It is found that the most important shortcomings of the Westergaard curling solution are his assumption of continuous contact between slab and subgrade (infinite slab self-weight), and his explicit treatment only of daytime conditions. Although Westergaard's curling-only predictions are significantly inferior to those from ANN and statistics, his load-plus-curling predictions exhibit approximately the same scatter as those from these two more modern and nominally more sophisticated tools. The case of Westergaard's curling solution can serve as an example pointing to the usefulness and desirability of theoretical solutions, even when these are only achievable on the basis of considerable abstraction and simplification.

41. Jennings, H., D. L. Johnson, G. M. Moss, and A. W. Saak. 1997. *Pooled Fund Study of Premature Concrete Pavement Deterioration*. Final Report. Iowa DOT Project HR-1063. Iowa Department of Transportation, Ames, IA.

Recently, a number of roads have begun to exhibit the onset of deterioration at relatively early ages. Since this deterioration appears to be the result of materials issues, data concerning raw materials, design, and paving conditions have been collected and analyzed for correlation between independent variables and deterioration. This analysis shows that there is a positive and statistically significant correlation between deterioration and the following variables: alkali and sulfate content of the cementitious materials, impermeable base course, paving temperature, and the presence of fly ash. This study also finds a significant need for improvement in data collection and maintenance by many organizations responsible for the production of concrete.

42. Jiang, Y., M. I. Darter, and E. Owusu-Antwi. 1996. "Analysis of Current State Rigid Pavement Design Practices in the United States." *Transportation Research Record 1525*. Transportation Research Board, Washington, DC.

Current PCC pavement design practices and the key concrete pavement design features used by State highway agencies in the United States are summarized. This information was obtained from a comprehensive survey conducted in 1994 and 1995 under an NCHRP research project. Pavement types, design methodologies, and reliability levels are included, along with many design inputs. Parameters that the States use to characterize pavement site conditions, including climate, subgrade, and traffic, are given. The designed concrete slab thicknesses for different site condition combinations are compared. An analysis of variance compared the mean slab thicknesses designed in different climatic regions. This examination and summary of the details of current pavement design practices and design features for concrete pavements in the United States will be of interest to both pavement researchers and practitioners.

43. Jiang, J. Y. and S. D. Tayabji. 1998. "Mechanistic Evaluation of Test Data From Long-Term Pavement Performance Jointed Plain Concrete Pavement Test Sections." *Transportation Research Record 1629*. Transportation Research Board, Washington, DC.

Over the years, pavement engineers have attempted to develop rational mechanistic-empirical (M-E) methods for predicting pavement performance. In fact, the next version of AASHTO's guide for pavement design will be mechanistically based. Many M-E procedures have been developed on the basis of a combination of laboratory test data, theory, and limited field verification. Therefore, it is important to validate and calibrate these procedures using additional data from in-service pavements. The LTPP program data provide the means to evaluate and improve these models. A study was conducted to assess the performance of some of the existing concrete pavement M-E-based distress-prediction procedures when used in conjunction with the data being collected as part of the LTPP program. Fatigue cracking damage was estimated using the NCHRP 1-26 approach and compared with observed fatigue damage at 52 GPS-3 test sections. Use of LTPP data was shown to successfully develop better insight into pavement behavior and improve pavement performance.

44. Jiang, Y. J., S. D. Tayabji, and C. L. Wu. 1998. *Mechanistic Evaluation of Test Data from LTPP Jointed Concrete Pavement Test Sections*. Report No. FHWA-RD-98-094. Federal Highway Administration, Washington, DC.

This study aimed to assess how well some of the existing concrete pavement M-E based distress-prediction procedures performed when used in conjunction with data being collected as part of the national LTPP program. As part of the study, appropriate data were obtained from the National Information Management System for the GPS-3 and GPS-4 experiments. Structural analysis was performed for up to 140 axle-load configurations for the selected test sections. Then, the ILLI-CONC software (developed under NCHRP 1-26) and PCA's procedures were used to predict fatigue cracking and joint faulting damage, respectively. The computed results were compared with observed values. This study showed that, even given the many current limitations in the LTPP database, LTPP data can be used successfully to develop better insight into pavement behavior and to improve pavement performance.

45. Kazmierowski, T. J. and G. A. Wrong. 1989. "Six Years Experience with Experimental Concrete Pavement Sections in Ontario." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

    In 1982, four experimental sections of rigid pavement were constructed on Highway 3 southeast of Windsor to assess the comparative performance and overall serviceability of various pavement, drainage, and shoulder designs plus two types of surface textures. Recent innovative developments in concrete pavement design methodology, material specification, construction techniques, and pavement drainage systems prompted Ontario's Ministry of Transportation to construct these test pavements. Summaries of the design and construction details, plus the results of an on-going performance-monitoring program, are documented in this paper. The performance of the pavement section is described in terms of load transfer and pavement edge deflections based on FWD testing, pavement condition ratings, roughness, skid resistance, joint movement, and a crack survey. Observations of noise levels, traffic volumes, and surface textures are discussed. Conclusions based on 6 years of performance indicate the superior performance of the free-draining base materials. In addition, some anomalous behavior based on pavement cracking and roughness suggests additional areas of process control are warranted. (Note: An update on the performance of these experimental sections is presented by Kazmierowski and Bradbury, "Ten Years Experience with Experimental Concrete Pavement Sections in Ontario," *Fifth International Conference on Concrete Pavement Design and Rehabilitation,* Purdue University, West Lafayette, IN, 1993.)

46. Khazanovich, L., M. I. Darter, R. Bartlett, and T. McPeak. 1998. *Common Characteristics of Good and Poorly Performing PCC Pavements.* Report No. FHWA-RD-97-131. Federal Highway Administration, Washington, DC.

    This report documents the analysis and findings of a study to identify the site conditions and design/construction features of concrete pavements (JPCP, JRCP, CRCP) that lead to good performance and those that lead to poor performance. Data from the LTPP test sections were used along with findings from previous and ongoing analyses of LTPP data. As there were no known criteria for identifying performance expectations over time as good, normal, or poor, a group of experts was convened to establish criteria. Separate criteria were developed for performance in roughness, joint faulting, transverse cracking, and localized failures (CRCP).

    Many significant site conditions and design/construction features were identified that lead to good and poor performance. The site conditions (traffic, climate, and subgrade) cannot be controlled by the designer, but steps can be taken to mitigate their effects. Several design and construction features can be controlled or specified by the highway agency; these should be given careful consideration. Knowledge of the design features identified as being critical to concrete pavement performance will contribute to improved guidelines for the design and construction of long-lasting PCC pavements.

47. Kim, S-M, M. Won, and B. F. McCullough. "Numerical Modeling of Continuously Reinforced Concrete Pavement Subjected to Environmental Loads." *Transportation Research Record 1629*. Transportation Research Board, Washington, DC.

Continuously reinforced concrete pavement (CRCP) performance depends on, among other factors, the characteristics of early developing cracks caused by environmental loads. The primary objective is to evaluate effects of design, materials, and construction variables on the characteristics of cracks in CRCP when subjected to environmental loads. A mechanistic model is developed using finite element formulations. Concrete and longitudinal steel are discretized using the plane strain and the frame elements, respectively. Various bond stress and slip models between concrete and longitudinal steel and between concrete and the underlying layers are developed using the spring elements. The creep effect is also included using the effective modulus method. CRCP responses from the model vary depending on the concrete and steel bond-slip models. An accurate bond-slip model needs to be investigated further by experiments to increase the accuracy of the mechanistic model. Concrete creep has beneficial effects on CRCP responses. The thermal coefficient of concrete has significant effects on CRCP responses. Using concrete with a low thermal coefficient will improve CRCP performance. Longitudinal steel variables—the amount of steel, bar diameter, and steel location—are important design variables that influence CRCP behavior. For given environmental conditions, an optimum steel design can be developed using the model developed.

48. Ksaibati, K. and R. Staigle. 1995. "Faulting Performance Modeling for Undoweled Plain Concrete Pavements." *Transportation Research Record 1482*. Transportation Research Board, Washington, DC.

Data on factors causing faulting in undoweled plain concrete pavements were collected. A large number of concrete pavement test sections located in southern Wyoming were included in the experiment. Extensive field data were collected on all test sections. These data included faulting over a 4-year period, traffic applications, construction information, annual precipitation, and drainage conditions. A statistical model was developed to predict joint faulting. The most important factors contributing to joint faulting were identified as traffic loadings, slab thickness, and edge drains.

49. Kunt, M. M. and B. F. McCullough. 1992. *Improved Design and Construction Procedures for Concrete Pavements Based on Mechanistic Modeling Techniques*. Report No. FHWA/TX-92+1169-5F. Texas Department of Transportation, Austin, TX.

This report describes an improved set of design and construction procedures for jointed reinforced concrete pavements. A systems approach was used to develop, analyze, evaluate, and implement recommended procedures for designing concrete pavement reinforcement and for determining sawing time and depth. An evaluation of the subgrade drag theory for reinforcement design indicated that it incorrectly predicts the required amount of reinforcement. Revised reinforcement equations are developed; these are believed to be more representative of actual conditions. Monte Carlo simulation was conducted to determine required sawing depths and times for a selected probability level.

50. LaCoursiere, S. A., M. I. Darter, and S. A. Smiley. 1978. *Performance of Continuously Reinforced Concrete Pavement in Illinois*. Report No. FHWA-IL-UI-172. Illinois Department of Transportation, Springfield, IL.

A study of the performance of CRCP constructed on the Illinois interstate system has been conducted. Approximately 1,980 km (1,230 mi) of interstate pavement were surveyed, consisting of 175- to 250-mm (7- to 10-inch) slabs over granular and stabilized subbases. CRCP slab thickness, foundation support, and the presence of susceptible D-cracking aggregate were found to have a substantial effect on the performance of the pavements. Recommendations on the design and construction practices are made.

51. Larson, R. M. and S. D. Tayabji. 1994. "Performance of Continuously Reinforced Concrete Pavements." *Third International Workshop on the Design and Evaluation of Concrete Pavements,* Krumbach, Austria.

Currently, the United States has more than 30,000 lane miles of continuously reinforced concrete (CRC) pavements. Many are more than 20 years old and have provided excellent performance over the years. Much of the CRC pavement technology has developed through experience. This fact and the recent use of new design features (such as tied concrete shoulder, permeable cement-treated base, and epoxy-coated steel) pointed to a need to evaluate performance of existing CRC pavement sections. This paper summarizes the findings of a national pooled fund study (administered by FHWA) aimed at updating the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRC pavements. As part of the study, a comprehensive field investigation of 23 in-service CRC pavements was conducted to study the effects of various design and construction features on performance of CRC pavements. The investigation included crack mapping/distress survey, profile/roughness measurement, FWD testing, and materials sampling and testing. In addition, the data collected to date from the 85 CRC pavement sections in the LTPP GPS-5 experiment were also analyzed. Key findings of the field investigation program as they relate to CRC pavement design and construction are presented.

52. Larson, R. M., S. Vanikar, and S. Foster. 1993. U.S. Tour of European Concrete/U.S. Tour of European Concrete Highways (U.S. TECH), Follow-Up Tour of Germany and Austria—Summary Report. Report No. FHWA-SA-93-080. Federal Highway Administration, Washington, D.C.

This report describes the findings and recommendations of the follow-up U. S. Tour of European Concrete Highways, conducted October 10 to 22, 1992. The goal was to obtain sufficient information to construct experimental highway sections in Michigan and other States using the German design and to review an active construction project in Austria to obtain information on the exposed aggregate surface treatment technique to reduce tire/pavement noise. A major feature of the German cross section is the use of a 15 cm (6 cm) lean concrete or cement-bound pre-notched base, to which the concrete slab is bonded. Other major features are the provision of a thick granular blanket layer under the stabilized base and the provision of longitudinal edge drains, usually outletted to the storm drain system. Plate bearing quality assurance tests are run on the subgrade and the granular blanket surfaces to assure strong support for the stabilized pavement structure.

53. Lee, Y. H., J. H. Bair, C. T. Lee, S. T. Yen, and Y. L. Lee. 1993. "Development of New Stress Analysis and Thickness Design Procedures for Jointed Concrete Pavements." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance.* Purdue University, West Lafayette, IN.

This study focused on the development of an alternative stress-estimation procedure to instantly calculate the critical stresses in jointed concrete pavements. The primary components of stress analysis, including gear configurations, total wheel load, tire pressure, a widened outer lane, a tied concrete shoulder, and thermal curling due to a linear temperature differential, need to be considered. The ILLI-SLAB finite element program was used for this analysis. In validation of the program, very favorable results were obtained in comparison with data from Taiwan's second northern highway, the AASHO road test, and the Arlington road test. Dimensional analysis and experimental design were employed to plan a factorial of finite element runs over wide ranges of pavement design parameters. Prediction equations for stress adjustments were then developed using a modern regression technique (projection pursuit regression). Subsequently, a simplified stress analysis procedure was implemented in a user-friendly computer program (TKUPAV) to facilitate instant stress estimation. Together with PCA's cumulative fatigue damage equation, a modified PCA stress analysis and thickness design procedure was also incorporated in the TKUPAV program. This program may be used not only for stress calculation but also structural analysis and design of jointed concrete pavement.

54. Lee, Y. H. and M. I. Darter. 1994. "Loading and Curling Stress Models for Concrete Pavement Design." *Transportation Research Record 1449*. Transportation Research Board, Washington, DC.

Determining the edge loading tensile bending stress in a concrete slab due to individual and combination effects of wheel loading and thermal curling is important to a mechanistic-based design procedure. The paper describes the identification of two additional dimensionless mechanistic variables, such that the problems encountered in previous investigations that have used dimensional analysis for thermal-related curling problems are resolved. A new regression technique (projection pursuit regression) together with traditional linear and nonlinear regressions is used to develop prediction models, which provide an accurate representation of the finite element model. They are simple, easy to comprehend, dimensionally correct, may be extrapolated to wider ranges of other input parameters, and are ready for implementation in a spreadsheet or computer program. Examples of practical applications using the new models are also provided.

55. Lee, Y. H. and M. I. Darter. 1995. "Development of Performance Prediction Models for Illinois Continuously Reinforced Concrete Pavements." *Transportation Research Record 1505*. Transportation Research Board, Washington, DC.

A new predictive modeling approach is presented and the approach for localized failures in Illinois CRCP is demonstrated. Some data retrieval guidelines from the Illinois Pavement Feedback System database are first presented. A preliminary data analysis was conducted to assist in data cleaning and assessing the variability of the data before the analysis was performed. Several modern regression techniques ("robust" and "nonparametric" regressions) were introduced in a new predictive modeling approach. The proposed modeling approach was used to develop an improved model for localized failures in CRCP. The resulting model includes several variables such as cumulative ESALs, slab thickness, steel reinforcing content, steel reinforcing placement method, and base type for the prediction of CRCP failures. A sensitivity analysis was also performed to illustrate the effect of these variables on failures. Slab thickness and steel content are by far the most significant variables affecting performance. Crack spacing was found to have no effect.

56. Lu, J., B. F. McCullough, and C. L. Saraf. 1989. "Maximum Entropy Spectral Analysis of Transverse Crack Spacing in Continuously Reinforced Concrete Pavements." *Transportation Research Record 1227*. Transportation Research Board, Washington, DC.

    The characteristics of cracks of CRCP are generally evaluated by analyzing the distribution of transverse crack spacing in the pavement. Statistical analysis of data produces the mean and the standard deviation of crack spacing. However, these parameters are not always sufficient for characterizing the crack spacing of CRCP. Therefore, this paper proposes an alternate method for analyzing the transverse crack spacing data. This method, maximum entropy spectral analysis (MESA), analyzes the data in this frequency domain rather than in the space domain. By using MESA, the uniformity and variability of crack spacing can be observed in the frequency domain. The results of analysis using MESA indicate that this method can intuitively distinguish the characteristics of transverse crack spacing distribution in CRCP containing different types of coarse aggregates.

57. McCullough, B. F. 1993. *Design of Continuously Reinforced Concrete Pavements for Highways*. Concrete Reinforcing Steel Institute, Schaumburg, IL.

    This document presents design information for CRCP in highway applications. The focus of the document is on steel reinforcement design, with references to other procedures for structural thickness design. Key steel reinforcement factors included in the design procedure are PCC material properties (tensile strength, shrinkage, thermal coefficient of expansion, and modulus of elasticity), steel reinforcement properties (yield strength, bar size, thermal coefficient of expansion, and modulus of elasticity), base, subbase, and subgrade properties (elastic modulus values, layer thicknesses), wheel loading, climatic data (daily temperature differential and annual temperature difference), and pavement structure geometry (slab width, paving width, and distance to longitudinal construction joint). These factors, in conjunction with user-selected design limits on minimum crack spacing, maximum crack spacing, maximum crack width, and allowable steel stress, are used in a series of design equations to determine the required steel content. An accompanying computer program, CRC–HIGHWAY PAVE, is available to expedite the calculations.

58. McCullough, B. F. and T. Dossey. 1999. Controlling Early Age Cracking in Continuously Reinforced Concrete Pavement: Observations from 12 Years of Monitoring Experimental Test Sections in Houston, Texas. Preprint No. 991460. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

    This paper presents findings drawn from 12 years of monitoring and analyzing thermal cracking in experimental CRCP placed in and around Houston, TX. The purpose of the study was to evaluate new design elements and construction considerations intended to control early-age thermal cracking and cracking-related distress caused by coarse aggregate with a high thermal coefficient of expansion. Experimental factors considered include coarse aggregate type, percentage of steel reinforcement, bar size, double or single mat steel, paving time, and paving season. Findings from the study show significant differences in performance between low and high thermal coefficient aggregates, effective methods to minimize them, as well as some attempts that were not as successful. Aggregate type and placement season were found to be the most significant factors affecting PCC performance, whereas day or night placement, steel percentage, bar size, and skewed placement proved less significant.

59. Moody, E. D. 1998. "Transverse Cracking Distress in Long-Term Pavement Performance Jointed Concrete Pavement Sections." *Transportation Research Record 1629*. Transportation Research Board, Washington, DC.

    The primary design variables and distress mechanisms that can cause varying degrees of transverse cracking in jointed concrete pavements are discussed. Included in the discussion is the Strategic Highway Research Program's LTPP program, which has collected a significant amount of condition survey data on more than 110 jointed plain concrete pavements and 65 jointed reinforced concrete pavements throughout North America over the past 7 years. A complete analysis of the transverse cracking that has occurred in the LTPP test sections, along with their respective relationships with the primary prediction variables found in the primary distress mechanisms, is provided.

60. Mueller, A. L., D. G. Peshkin, K. D. Smith, and M. I. Darter. 1990. Performance of Jointed Concrete Pavements, Volume VI—Appendix C: Synthesis of Concrete Pavement Design Methods and Analysis Models, and Appendix D: Summary of Analysis Data for the Evaluation of Predictive Models. Report No. FHWA-RD-89-141. Federal Highway Administration, Washington, DC.

    A major national field and analytical study has been conducted into the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the country. This volume provides a general evaluation of several concrete pavement design and analysis models. This includes RISC, ILLI-SLAB, JSLAB, H51, WESLIQID, WESLAYER, JSC-1, JCP-1, AASHTO, RPS-3, PCA, PMARP, PEARDARP, PREDICT, BERM, CMS, Liu-Lytton, JRCP-4, and California DOT procedures. Sensitivity analyses are included for each pavement sections. Also included in this volume is supporting documentation for the analyses presented in volume II.

61. Mueller, P. E. and L. A. Scofield. 1989. "An Expanded Evaluation of Arizona's Ten Mile Concrete Test Roadway." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation*. Purdue University, West Lafayette, IN.

    This paper describes the performance of the Superstition Freeway (State Route 360) located near Phoenix, AZ. The construction of this freeway marked a new era in the concrete pavement design philosophies of the Arizona DOT. It was the start of what became a 16-km (10-mi) corridor of experimental test sections. The experimental sections consist of full-depth JPCP, JPCP over cement-treated base, JPCP over lean concrete base, and prestressed concrete pavement over lean concrete base. Performance evaluations based primarily on roughness and skid characteristics are presented, along with faulting, deflection, and maintenance data. A statistical analysis of Arizona DOT's pavement management inventory data was performed to develop linear models for both roughness and skid properties. The roughness models represent the expected roughness over the 20-year design life. The models representing skid properties could only be developed for the years in which the field data were collected.

62. Neal, B. F. 1987. *Evaluation of Design Changes and Experimental PCC Construction Features*. Report No. FHWA/CA/TL-85/07. California Department of Transportation, Sacramento, CA.

This report covers the performance of experimental design and construction features implemented at four study sites. The first study site consisted of an experimental CRCP compared with JPCP with different design features. The second site includes a field trial of four different types of joint sealant materials. The third site included an evaluation of PCC pavements with experimental shoulder treatments, including PCC shoulders. The fourth site included an evaluation of other experimental bridge approach slabs and permeable asphalt-treated and cement-treated bases. Results from these studies are being reviewed and further experimentation, and in some cases implementation, of the design features is being conducted.

63. New York State Department of Transportation (NYSDOT). 1993. *The New York State Thickness Design Manual for New and Reconstructed Pavements*. New York State Department of Transportation, Albany, NY.

The purpose of this manual is to provide information on the background, rationale, and detailed behind the NYSDOT policy for thickness design of new and reconstructed pavements. It has been determined by the NYSDOT pavement design task force that positive pavement drainage and increased pavement life are important aspects in designing quality pavements. This manual is based on the 1986 AASHTO design guide for determining pavement thickness, as well as using a treated open-graded permeable base layer with continuous edge drains in the pavement structure for positive drainage. Rigid pavements will be potentially thicker slabs (up to 325-mm (13-inch) thick) and designed with full-depth tied concrete shoulders with widened PCC slabs to reduce critical edge and corner stresses. Dowel bars are recommended for all Interstate pavements, and additional select granular subbase may be required for certain conditions.

64. Nussbaum, P. J. and E. C. Lokken. 1977. "Design and Construction of Concrete Pavements." *Proceedings, International Conference on Concrete Pavement Design.* Purdue University, West Lafayette, IN.

In 1975, the PCA conducted a survey of concrete pavement design and construction practices in the United States. This paper summarizes the design and construction practices by concrete pavement type, as well as typical construction costs. A field survey of 85 concrete pavement projects in 7 States also was conducted. Major problem areas noted in the survey for each pavement design type are summarized and recommended design and construction modifications are presented.

65. Owusu-Antwi, E., L. Titus-Glover, and M. I. Darter. 1998. Design and Construction of PCC Pavements, Volume I: Summary of Design Features and Construction Practices that Influence Performance of Pavements. Report No. FHWA-RD-98-052. Federal Highway Administration, Washington, DC.

    This study was conducted to evaluate and analyze PCC pavements to develop recommendations for the design and construction of long-lasting concrete pavements. It involved a detailed evaluation and analysis of the PCC pavement data in the LTPP database to determine the beneficial effects of specific design features and construction practices on long-term performance. This volume provides a concise summary of the results obtained from the study. It includes an overview of the engineering and statistical analyses that were conducted, and presents results that can be used by SHAs to obtain high-performance PCC pavements. The focus of the design and construction recommendations is on the prevention of key PCC pavement distresses.

66. Permanent International Association of Road Congresses (PIARC). 1994. *Continuously Reinforced Concrete Pavement*. Report No. 07.06.B. Permanent International Association of Road Congresses, Paris, France.

    The increasing use of CRCP around the world has led PIARC to prepare this synthesis on the state of the art of CRCP. The engineering and economic advantages offered by CRCP are described, followed by detailed discussions on the design (including steel design and terminal end movements), construction, performance, and maintenance and repair of CRCP. New developments in CRCP technology are also presented, along with main recommendations regarding its design and use.

67. Peshkin, D. G., K. D. Smith, G. S. James, L. D. Evans, and M. I. Darter. 1991. *Evaluation of Concrete Pavements in the Phoenix Urban Corridor, Volume II—Appendices*. Report No. FHWA-AZ91-264-II. Arizona Department of Transportation, Phoenix, AZ.

    A comprehensive evaluation of the various concrete pavement designs in the Phoenix, AZ urban corridor was conducted. This volume provides supporting project documentation to the main report. The appendices of this volume summarize the performance data for the sections; provide strip maps taken from the distress surveys; describe the creation and use of the project database; summarize the results from the weigh-in-motion studies; furnish rehabilitation selection guidelines; and provide an overview of recommended rehabilitation methods.

68. Portland Cement Association (PCA). 1984. *Thickness Design for Concrete Highway and Street Pavements*. Report No. EB109.01P. Portland Cement Association, Skokie, IL.

    This bulletin provides methods for determining slab thicknesses adequate to carry traffic loads on concrete streets, roads, and highways. The thickness design procedure is based on mechanistic concepts calibrated to documented concrete pavement performance. Both a fatigue analysis (for the evaluation of the suitability of the slab thickness to withstand fatigue cracking from the projected traffic) and an erosion analysis (for the evaluation of loss of support at slab corners due to pumping under the projected traffic) are conducted. A simplified design procedure is also presented for use when detailed axle-load data are not available. The thickness design criteria are based on general pavement performance experience, and can be modified as particular climate, soil, or drainage conditions dictate.

69. Saraf, C., C. Chou, and B. F. McCullough. 1987. "Effect of Rainfall on the Performance of Continuously Reinforced Concrete Pavements in Texas." *Transportation Research Record 1227*. Transportation Research Board, Washington, DC.

    The effect of rainfall on the performance of CRCP in Texas was studied by analyzing the condition survey data on CRC pavements throughout the State. Rigid pavement condition survey data were collected for 10 years, beginning in 1974. For the purpose of this study, the data were grouped into districts, and the average performance of pavements in each district was estimated. The average performance of CRC pavements was determined by adding the number of patches and punchouts in the pavement and estimating the number of failures per mile. The results of the study indicate that the average annual rainfall on the performance of CRC is significant. Although the initial performance of the pavements located in different rainfall areas is practically the same, when the pavement starts developing failures, the rate of failure development is affected by the average rainfall in the area. Pavements located in 250-mm (10-inch) rainfall areas generally showed an almost zero rate of failure development, whereas pavements located in 1,300-mm (52-inch) rainfall areas developed failures at a rate of about 0.6 failures per km (one failure per mi) per year.

70. Smiley, D. L. 1995. *First Year Performance of the European Concrete Pavement on Northbound I-75 - Detroit, Michigan*. Report No. FHWA-SA-95-046. Federal Highway Administration, Washington, DC.

    This report describes the performance of the I-75 European concrete pavement reconstruction project approximately one year after construction. The experimental features of the pavement design were assimilated from designs used in Germany and Austria. The objective is to determine whether innovative features of typical rigid pavement designs used in European countries can be applied cost effectively to conventional design and construction methods used for rigid pavements in the United States. The European pavement is part of a major Michigan project to reconstruct 3.7 km (2.3 miles) of the I-75 (Chrysler) freeway in downtown Detroit. I-75 is 6 to 8 lanes wide and carries about 111,000 vehicles a day (11 percent are trucks). The approximately 1.6-km (1-mile) long European pavement is located on northbound I-75 just north of I-94. It will be monitored for 5 years. The experimental pavement appears to be performing as expected, except for the disappointing results pertaining to the exposed aggregate surface as a means to reduce traffic noise levels. Specific points of interest are as follows: No surface distress features, except for minor popouts, have developed on the European pavement, in contrast to the Michigan pavement, where 50 percent of the pavement panels have one or two transverse cracks; the ethylene propylene diene terpolymer joint seals are performing satisfactorily; the exposed aggregate surface appears to have lost macrotexture in the two inner lanes of northbound I-75, where traffic was during most of 1994 while southbound I-75 was being reconstructed; surface friction increased as expected; and the exposed aggregate surface provides only a slight reduction (0.4 dBA) in exterior and interior car noise levels, as compared to a reported 4 to 5 and 3.5 to 4.5 dBA, respectively, in Europe.

71. Smith, K. D. and K. T. Hall. 1999. Concrete Pavement Design Details and Construction Practices: State of the Art Technical Digest. Federal Highway Administration, Washington, DC.

This technical digest has been prepared to concisely present current recommendations and guidelines on design and construction details for concrete pavements. Emphasis is given to JPCP, although a separate chapter on special design considerations for JRCP and CRCP is provided. This document is intended for use by design and construction engineers to assist in the development and construction of concrete pavements that perform better and last longer. Key topics covered include: concrete pavement type selection; subgrade preparation and characterization; drainage design; base and subbase considerations; thickness design; joint design; shoulder considerations; construction considerations; and special design considerations for reinforced concrete pavements. The recommendations and guidelines presented herein are based on recent research reports and concrete pavement performance studies. A detailed annotated bibliography of applicable documents from the past 10 years is included as an appendix.

72. Smith, K. D., A. L. Mueller, M. I. Darter, and D. G. Peshkin. 1990. Performance of Jointed Concrete Pavements, Volume II—Evaluation and Modification of Concrete Pavement Design and Analysis Models. Report No. FHWA-RD-89-137. Federal Highway Administration, Washington, DC.

A major national field and analytical study was conducted into the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the country. This volume investigates the accuracy of several pavement performance models and illustrates the usefulness of several concrete design and analysis procedures. Performance models evaluated include AASHTO, PREDICT, PEARDARP, and PFAULT; design and analysis models evaluated include PMARP, JSLAB, ILLI-SLAB, CMS, Liu-Lytton, JCP, JCS, and BERM. Based upon the data collected from this and other studies, new prediction models were developed for selected performance indicators.

73. Smith, K. D., D. G. Peshkin, M. I. Darter, and A. L. Mueller. 1990. *Performance of Jointed Concrete Pavements, Volume III—Summary of Research Findings*. Report No. FHWA-RD-89-138. Federal Highway Administration, Washington, DC.

A major national field and analytical study was conducted into the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the United States. This volume gives a broad overview of the work performed in this study. Summaries of the effect of various design features on concrete pavement performance are reviewed and performance trends identified. The accuracy of various prediction models and analysis methods are examined using the field performance data. From that evaluation, new prediction models were developed and a cost-effectiveness evaluation was performed.

74. Smith, K. D., D. G. Peshkin, M. I. Darter, A. L. Mueller, and S. H. Carpenter. 1990a. *Performance of Jointed Concrete Pavements, Volume I—Evaluation of Concrete Pavement Performance and Design Features*. Report No. FHWA-RD-89-136. Federal Highway Administration, Washington, DC.

    A major national field and analytical study examined the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the country. This volume presents a brief introduction to the data collection activities and presents a description and performance evaluation of the pavement sections in the study. Documentation is presented on the effects of slab thickness, base type, joint spacing, reinforcement, joint orientation, load transfer, dowel bar coatings, longitudinal joint design, joint sealant, tied shoulders, and subdrainage on jointed concrete pavement performance.

75. Smith, K. D., D. G. Peshkin, M. I. Darter, A. L. Mueller, and S. H. Carpenter. 1990. *Performance of Jointed Concrete Pavements, Volume IV—Appendix A: Project Summary Reports and Summary Tables*. Report No. FHWA-RD-89-139. Federal Highway Administration, Washington, DC.

    A major national field and analytical study examined the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the United States. This volume provides detailed summary reports documenting the design, construction, and performance of the 95 concrete pavement sections in the study. Comprehensive summary tables compile the design and performance data from each section into a convenient format.

76. Smith, K. D., D. G. Peshkin, M. I. Darter, A. L. Mueller, and S. H. Carpenter. 1990c. *Performance of Jointed Concrete Pavements, Volume V—Appendix B: Data Collection and Analysis Procedures*. Report No. FHWA-RD-89-140. Federal Highway Administration, Washington, DC.

    A major national field and analytical study examined the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the country. This volume provides documentation of the data collection and analysis procedures used. The field data collection activities, including deflection testing and coring/boring, are discussed, as are the weigh-in-motion data collection procedures. Traffic computations and backcalculation methodologies are presented, along with the basis for the assessment of drainage conditions.

77. Smith, K. D., D. G. Peshkin, A. L. Mueller, E. Owusu-Antwi, and M. I. Darter. 1991. *Evaluation of Concrete Pavements in the Phoenix Urban Corridor, Volume I—Final Report*. Report No. FHWA-AZ91-264-I. Arizona Department of Transportation, Phoenix, AZ.

A comprehensive evaluation of various concrete pavement designs in the Phoenix, AZ, urban corridor was conducted. This volume summarizes the performance of the designs. Performance data are then used to evaluate the suitability of available pavement design models to Arizona conditions. The performance data were also used in developing design recommendations and in determining suggested rehabilitation activities for each section.

78. Smith, K. D., M. J. Wade, D. G. Peshkin, and M. I. Darter. 1997. *Performance of Concrete Pavements. Volume I: Field Investigation*. Interim Report. Report No. FHWA-RD-94-177. Federal Highway Administration, McLean, VA.

With the goal of improving future concrete pavement design and construction practices, this project evaluated the performance of 303 inservice concrete pavement sections located throughout North America. An extensive field-testing program, consisting of pavement condition surveys, drainage surveys, FWD testing, coring/boring operations, and roughness testing, was conducted to collect the information needed for analysis. Because many of these pavement sections are part of State-level studies on concrete pavements, a range of design variables (e.g., load transfer, slab thickness, joint spacing, drainage) thought to affect concrete pavement performance are present. Over one-third of the sections was evaluated under a preceding FHWA study; 5-year performance trends thus are available for some sections. Additional pavement performance data are also available for 96 European concrete pavement sections and for 21 Chilean concrete pavement sections. The average age and average cumulative ESALs for the North American sections are 16 years and 7.1 million, respectively, compared to 21 years and 21.8 million for the European sections, and 9 years and 5.9 million for the Chilean sections. This volume describes the field data collection activities conducted under the study and also summarizes the individual performance of the 303 concrete pavement sections evaluated. Pertinent design and construction data are given for each project, and key performance data are presented. Where available, 5-year performance trends are also summarized.

79. Smith, K. D., M. J. Wade, D. G. Peshkin, L. Khazanovich, H. T. Yu, and M. I. Darter. 1998. *Performance of Concrete Pavements, Volume II—Evaluation of Inservice Concrete Pavements.* Report No. FHWA-RD-95-110. Federal Highway Administration, Washington, DC.

With the goal of improving future concrete pavement design and construction practices, this project evaluated the performance of 303 inservice concrete pavement sections throughout North America. This volume examines the performance of the various concrete pavement sections in the study. This examination primarily consists of an evaluation of the effect of concrete pavement design features on concrete pavement performance. Design features investigated include slab thickness, joint spacing, joint orientation, load transfer, joint sealant, base type, drainage, shoulder type, reinforcement, and pavement type. The results of an examination of the backcalculation results are also presented, as are the significant findings of an evaluation conducted on the performance of European and Chilean concrete pavements.

80. Smith, K. D., M. J. Wade, H. T. Yu, and M. I. Darter. 1997. "Design Considerations for Improved JPCP Performance." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance*. Purdue University, West Lafayette, IN.

This paper presents recommendations for various aspects of concrete pavement design, based on the results of an evaluation of the performance of more than 300 concrete pavement sections in North America. Recommendations are presented for subgrade preparation, drainage, base type selection, shoulder/edge support type selection, concrete strength requirements, concrete thickness design, and joint design and construction.

81. Strauss, P. J. and M. F. Mitchell. 1989. "The Performance of Jointed Concrete Pavement as Affected by Factors Other Than Traffic-Induced Stresses." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

Design and construction techniques as developed overseas have been used in South Africa on PCC pavements with little variation based on local experience. This paper describes the methodology by which jointed unreinforced concrete pavements are being evaluated with the aim of improving local design and construction techniques. Indications are that failures found on the pavements can primarily be attributed to inadequate construction techniques and control. Although the pavements are performing very well (only 11 percent show cracking outside the joints), it is felt that an improvement in performance is possible if subgrades are compacted properly to reduce settlement, nonerodible subbases are constructed, load transfer at joints is enhanced by low-shrinkage concrete, joints are kept properly sealed, and concrete shoulders are used.

82. Tayabji, S. D. and G. T. Korovesis. 1993. *Principles and Practices of Concrete Pavement Design*. Final Report, Contract DTFH61-90-00050. Federal Highway Administration, Washington, DC.

This manual provides pavement design engineers with background information and recommended guidelines and considerations for the design and construction of concrete pavements. Items discussed include subgrade, bases and subbases, concrete materials, drainage, temperature and moisture effects, thickness design, joint design, and critical construction features. The approach presented incorporates consideration of all critical features during the design process, in contrast to focusing on thickness design only. Successful local experience and good construction practices are emphasized.

83. Tayabji, S. D., O. Selezneva, and Y. J. Jiang. 1999. *Preliminary Evaluation of LTPP Continuously Reinforced Concrete (CRC) Pavement Test Sections*. Final Report. Report No. FHWA-RD-99-086. Federal Highway Administration, McLean, VA.

As part of the study reported here, analysis of data from the LTPP GPS-5 test sections was conducted to identify factors that influence long-term crack spacing in CRC pavements and to determine the effect of crack spacing on pavement performance. Data from the 85 test sections from the GPS-5 experiment were analyzed. Due to limitations of the available data and the lack of certain key data, the study did not produce definitive findings on factors that affect long-term crack spacing and CRC pavement performance. Lack of early-age cracking due to ambient weather conditions at the time of construction will continue to limit the value of GPS-5 to produce meaningful data on factors affecting early-age cracking. Continued monitoring of GPS-5 sites and subsequent data analysis should yield information on how CRC pavement cracking and performance changes with time, loading, and other factors. It is expected that as additional data from the GPS-5 experiment become available, more in-depth analysis of the test data to derive definitive results will be possible. Results to date, as presented in this report, do indicate that CRC pavements have the potential to provide long-term, low-maintenance service life as evidenced by the many well-performing sections in the LTPP GPS-5 experiment.

84. Tayabji, S. D., P. J. Stephanos, J. S. Gagnon, and D. G. Zollinger. 1998. *Performance of Continuously Reinforced Concrete Pavements, Volume II: Field Investigations of CRC Pavements*. Report No. FHWA-RD-94-179. Federal Highway Administration, Washington, DC.

This is one of a series of reports prepared as part of a recent study (sponsored by FHWA) to update the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRC pavements. In this report, the details of a comprehensive field investigation of 23 inservice CRCP sections are presented. The investigation included crack mapping and distress survey, profile and roughness measurements, FWD testing, and materials sampling and testing for 305-m (332-yard) long in-service test sections. Detailed, site-by-site design, construction, and performance summaries are presented for each project.

85. Tayabji, S. D., D. G. Zollinger, G. T. Korovesis, P. J. Stephanos, and J. S. Gagnon. 1998. *Performance of Continuously Reinforced Concrete Pavements, Volume I: Summary of Practice and Annotated Bibliography*. Report No. FHWA-RD-94-178. Federal Highway Administration, Washington, DC.

This is one of a series of reports prepared as part of a recent study (sponsored by FHWA) to update the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRC pavements. This volume presents a comprehensive bibliography on CRC pavements, along with a summary of the current practice. The summary of practice provides brief details on the extent and practice of CRC pavement use in the United States and other countries.

86. Tayabji, S. D., D. G. Zollinger, J. R. Vederey, and J. S. Gagnon. 1998. *Performance of Continuously Reinforced Concrete Pavements, Volume III: Analysis and Evaluation of Field Test Data*. Report No. FHWA-RD-94-180. Federal Highway Administration, Washington, DC.

This is one of a series of reports prepared as part of a recent study (sponsored by FHWA) to update the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRC pavements. In this report, the results of a comprehensive field investigation of 23 in-service CRCP sections are presented; they show that there is a strong interaction among the steel content, concrete strength, and crack spacing. Steel contents in the range of 0.6 to 0.7 percent appear to provide desirable long-term average crack spacing in the range of 0.9 to 1.5 m (3 to 5 ft). Tied concrete shoulders did not appear to contribute to CRCP mainline performance, and the effect of base type on CRCP performance was not pronounced. The use of epoxy-coated reinforcement resulted in no undesirable cracking patterns.

87. Tayabji, S. D., D. G. Zollinger, J. R. Vederey, and J. S. Gagnon. 1998. *Performance of Continuously Reinforced Concrete Pavements, Volume V: Maintenance and Repair of CRC Pavements*. Report No. FHWA-RD-98-101. Federal Highway Administration, Washington, DC.

    This is one of a series of reports prepared as part of a recent study (sponsored by FHWA) to update the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRC pavements. In this report, the critical distress types associated with CRC pavements are reviewed, and then maintenance and repair techniques are presented that address those critical distresses. Preventive maintenance methods presented include joint and crack sealing, undersealing, cathodic protection, and edge drains; repair methods presented include conventional full-depth patching and alternative patch types.

88. Titus-Glover, L., E. B. Owusu-Antwi, and M. I. Darter. 1999. *Design and Construction of PCC Pavements. Volume 3. Improved PCC Performance Models*. Report No. FHWA-RD-98-113. Federal Highway Administration, McLean, VA.

    This study was conducted to evaluate and analyze PCC pavements to develop recommendations for the design and construction of long-lived concrete pavements. It involved a detailed evaluation and analysis of the PCC pavement data in the LTPP database using a variety of methods to determine design features and practices that have beneficial effects on long-term performance. Emphasis was placed on identifying specific design features that can be included during design to improve the performance of PCC pavements under various combinations of environmental and traffic loading conditions, and for different subgrade support conditions. The study focused on the development of practical recommendations that can be implemented by highway agencies to increase pavement life. This volume describes and provides information on improved distress and roughness prediction models developed as part of the study. A key focus was to develop distress and roughness prediction models that incorporate mechanistic principles but are still practical for use by SHAs.

89. Titus-Glover, L., E. B. Owusu-Antwi, T. Hoerner, and M. I. Darter. 1998. *Design and Construction of PCC Pavements, Volume 2. Design Features and Practices That Influence Performance of Pavements*. Report No. FHWA-RD-98-127. Federal Highway Administration, McLean, VA.

    This study was conducted to evaluate and analyze PCC pavements to develop recommendations for the design and construction of long-lived concrete pavements. It involved a detailed evaluation and analysis of the PCC pavement data in the LTPP database using statistical techniques to determine design features and construction practices that have a beneficial effect on long-term performance.

90. Verhoeven, K. 1993. "Cracking and Corrosion in Continuously Reinforced Concrete Pavements." *Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

CRC pavements have been used in Belgium since the 1950s. Over that time, the longitudinal reinforcement content has been reduced. Transverse reinforcement is placed obliquely to prevent ill-timed initial cracking above the transverse rods. A 6-cm sandwich layer of a bituminous mixture is placed between the concrete slab and the lean concrete base. The depth of the reinforcement in early projects was 6 cm; this was later increased to 9 cm. Overall, CRC pavements have performed very well, with no punchouts, minimal crack breakdown, and minimal reinforcement corrosion.

91. Vyce, J. M. 1988. *A Summary of Experimental Concrete Pavements in New York.* Research Report 141. New York State Department of Transportation, Albany, NY.

This report summarizes a 22-year study of concrete pavement design features that included constructing a test road with numerous design variables, two roads with several major design changes, and several roads among the first to incorporate a major change in load transfer devices across transverse joints. Performance of each item is discussed, along with a number of minor changes. Many findings have been implemented and have improved the performance and cost-effectiveness of rigid pavements in New York State.

92. Wade, M. J., G. D. Cuttell, J. M. Vandenbossche, H. T. Yu, K. D. Smith, and M. B. Snyder. 1997. *Performance of Concrete Pavements Containing Recycled Concrete Aggregate.* Report No. FHWA-RD-96-164. Federal Highway Administration, Washington, DC.

This report documents the field performance of nine concrete pavement projects that incorporate recycled concrete aggregate (RCA) in the construction of the pavement. Multiple sections were evaluated on many of the nine projects due to perceived differences in performance levels or variations in pavement design (such as the use of virgin aggregate or the inclusion of dowel bars). All told, a total of 17 sections (of which 12 contain RCA) were subjected to an extensive field testing program consisting of pavement condition surveys, drainage surveys, FWD testing, coring, and serviceability assessments. Each section is described in detail. Performance observations and results from the FWD and laboratory testing are presented, with emphasis on evaluating the effect of RCA on pavement performance. An overall summary synthesizes the findings and conclusions of the field tests.

93. Wade, M. J., K. D. Smith, H. T. Yu, and M. I. Darter. 1995. "Monitoring of European Concrete Pavements." *Transportation Research Record 1478*. Transportation Research Board, Washington, DC.

As part of the European Concrete Pavement Evaluation System (COPES) program, 77 concrete pavement sections from France, Italy, the United Kingdom, and Belgium were monitored and the data evaluated with the objective of providing continual improvements to the design, construction, and maintenance of concrete pavements. An overview of the data collected under the European COPES program is presented, along with a general look at overall performance trends. The European sections are all characteristic of the wet-freeze environmental region. Because of higher legal axle weights and longer design periods, European pavements are often exposed to many more ESAL applications than pavements in the United States. Of the sections evaluated, the most common pavement type is jointed plain concrete pavement. Extensive uses of stabilized bases, positive drainage features, and dowel bars are also evident. A qualitative analysis was conducted using present serviceability rating (PSR), age, and traffic as the principal parameters. The use of lean concrete bases and incorporation of a greater number of modern elements (e.g., dowel bars and positive drainage) improved the performance of the pavement sections. Models predicting the PSR of the pavement sections were also developed.

94. Wade, M. J., K. D. Smith, H. T. Yu, M. I. Darter, and C. J. Wienrank. 1998. *Performance of Concrete Pavements, Volume IV—Appendices.* Report No. FHWA-RD-95-112. Federal Highway Administration, Washington, DC.

With the goal of improving future concrete pavement design and construction practices, this project evaluated the performance of 303 in-service concrete pavement sections throughout North America. This volume includes supporting documentation used in the research study. Summary tables containing design, construction, and performance data for the concrete pavement sections evaluated under this study are presented in appendix A. Appendix B presents a summary report prepared on the performance of 77 European concrete pavement sections. Appendix C presents a summary report prepared on the performance of 21 concrete pavement sections in Chile.

95. Washington State Department of Transportation (WSDOT). 1995. *WSDOT Pavement Guide, Volume 2—Pavement Notes*. Washington State Department of Transportation, Olympia, WA.

This comprehensive guide was prepared for WSDOT personnel in design, constructing, and maintaining highway pavement structures. A broad overview of pavement engineering is presented, followed by detailed chapters on key pavement areas: fundamental design parameters (including materials characterization, traffic loading, and climatic factors), pavement evaluation procedures (including condition surveys, roughness testing, structural evaluation, and skid resistance), pavement response determination, AASHTO new and overlay design procedures, pavement rehabilitation methodologies, life-cycle cost analysis procedures, subsurface pavement drainage considerations, and specific pavement construction considerations.

96. Wells, G. K. 1993. *Summary Report To Improve Jointed Plain Concrete Pavement (JPCP) Performance*. Caltrans Report. California Department of Transportation, Sacramento, CA.

This report summarizes the efforts to date within Caltrans to improve JPCP performance. It recommends changes to transverse joint spacing, load transfer design, joint sealing, longitudinal joint design, and base type and selection. One important finding is the effect of climate on JPCP faulting and cracking performance. Recommendations to reduce climatic effects on JPCP performance are given.

97. Wells, G. K. and W. A. Nokes. 1991. *Synthesize PCCP Design Parameters Researched by Caltrans and Others*. Minor Research Report 65328-637391-31111. California Department of Transportation, Sacramento, CA.

This report provides background on previous research efforts by Caltrans and others in the area of PCCP design and performance that can be used to improve current rigid pavement performance. Much work has been done in this area, and a review of these findings is useful in updating current rigid pavement design standards. Based on the review here, it is recommended that skewed joints spaced at 3.7, 4.6, 4.0, 4.3 m (12, 15, 13, and 14 ft) intervals be maintained; that 760-mm (30-inch), No. 5 tiebars be installed along all longitudinal joints at 760-mm (30-inch) centers; that all transverse joints be sealed with silicone sealant material; that treated permeable bases be used when feasible (pending the results of further studies); and that tied PCC shoulders be employed on all new designs.

98. Wimsatt, A. J. 1993. "Concrete Pavement Design and Construction Practices by the Texas DOT." *Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

The Texas DOT is a decentralized organization consisting of 24 geographical districts. Each district makes its own decision on what pavement type will be designed and constructed. However, the Department's use of concrete pavements in highway projects has increased significantly over the past decade, especially in urban areas and in high-truck-traffic corridors. A paper presented at the Fourth Purdue conference in 1989 described and analyzed the use of the 1986 *AASHTO Design Guide* in designing concrete pavements in Texas. This paper discusses other aspects of Texas DOT concrete pavement design and construction practices, as well as research underway to improve design and construction methods.

99. Won, M., B. F. McCullough, and W. R. Hudson. 1988. *Evaluation of Proposed Texas SDHPT Design Standards for CRCP*. Report No. FHWA/TX-88+472-1. Texas State Department of Highways and Public Transportation, Austin, TX.

The primary factors to be considered in the design of CRCP are the structural responses: crack spacing, crack width, and steel stress. These responses are the outcome of the interactions among materials characteristics, environmental conditions, and traffic loading. Since at least three-quarters of the concrete volume is occupied by aggregate, critical concrete properties (such as thermal coefficient, tensile strength, modulus of elasticity, and drying shrinkage) are different for concrete made with different types of coarse aggregate. In this study, proposed CRCP design standards were evaluated for concretes made with various coarse aggregate types, namely siliceous river gravel (SRG) and limestone (LS). Generally, the standard provided a good design, although it was determined that two-layer reinforcement is desirable for SRG concrete at slab thicknesses greater than 325 mm (13 inches), and for LS concrete at slab thicknesses greater than 225 mm (11 inches). An analysis of the CRCP deformed wire standard indicated that wire spacings of 450 mm (18 inches) or greater are required.

100. Yoder, E. J. and M. W. Witczak. 1975. Principles of Pavement Design. Second Edition. John Wiley & Sons, Inc. New York, NY.

This widely used textbook presents the basic principles of pavement design for both flexible and rigid pavement structures. It includes a discussion of theories of stress distribution for both pavement types, and presents a summary of key design factors and considerations: subgrade, drainage, paving materials, traffic, and climate. Design methodologies for both pavement types are also presented; and pavement distress types and rehabilitation strategies are discussed.

101. Yrjanson, W. A. 1988. "Concrete Pavements U.S.A.: State of the Art." *Transportation Research Record 1182*. Transportation Research Board, Washington, DC.

This paper describes the current state of the art of concrete pavement construction in the United States, along with changes made to accommodate the increases in traffic volumes and loadings. Typical concrete pavement designs used for primary highway paving, airfield paving, and parking lots and truck terminals are presented. Developments in paving equipment over the past 30 years are described. Design changes that improve the performance of PCC pavements are reviewed, along with examples of innovative technology.

102. Yu, H. T., K. D. Smith, M. I. Darter, J. Jiang, and L. Khazanovich. 1998. *Performance of Concrete Pavements, Volume III—Improving Concrete Pavement Performance*. Report No. FHWA-RD-95-111. Federal Highway Administration, Washington, DC.

With the goal of improving future concrete pavement design and construction practices, this project evaluated the performance of 303 inservice concrete pavement sections throughout North America. This volume presents pavement performance prediction models that were developed from the data collected under this study. Prediction models are presented for transverse joint faulting (doweled and nondoweled), transverse cracking (JPCP and JRCP), transverse joint spalling (JPCP and JRCP), pavement serviceability (JPCP and JRCP), and pavement roughness (JPCP only). Based on the results of the models and on the results of the field evaluation findings, guidelines are presented for the improved design of concrete pavements.

103. Yu, H. T., L. Khazanovich, M. I. Darter, and A. Ardani. 1998. "Analysis of Concrete Pavement Responses to Temperature and Wheel Loads Measured from Instrumented Slabs." *Transportation Research Record 1639*. Transportation Research Board, Washington, DC.

The structural response of jointed plain concrete pavement slabs was evaluated using data obtained from instrumented slabs that were a part of newly constructed JPCP on I-70 in Colorado. The instrumentation consisted of dial gauges for measuring curling deflections at the slab corner and longitudinal edge and surface-mounted strain gauges for measuring load strains at the longitudinal midslab edge. The through-thickness temperature profiles in the pavement slabs were also measured at 30-minute intervals during the field test. Analysis of the field data showed that the instrumented slabs had a considerable built-in upward curling and that concrete slabs on a stiff base can act completely independent of the base or monolithically with the base, depending on the loading condition. The built-in upward curling of the slabs has the same effect as negative temperature gradients. These findings suggest that the effects of temperature gradients on the critical edge stresses may not be as great as previously thought and that the corner loading, in some cases, may produce more critical conditions for slab cracking. Another important finding of the study is that a physical bond between pavement layers is not required to obtain a bonded response from concrete pavements.

104. Zollinger, D. G. 1996. "Design, Construction, and Performance of CRC Pavement." Proceedings, Workshops on High Occupancy Vehicle Facilities, Pavement Management Systems, and Rigid Pavement Design and Construction. Taipei, Taiwan.

This paper discussed current CRC design concepts as well as proposed modifications and improvements to the current design procedure. Current CRC design methodology consists of the prediction of crack spacing by contraction restraints, the selection of limiting criteria to avoid punchout failures, and the estimation of the required pavement thickness based on these limits. This approach, however, does not take into account shear and load transfer across the transverse cracks. Therefore, this procedure alone is insufficient for characterization of failure modes including punchouts.

The load transfer efficiency is an important parameter in selecting a pavement thickness. This efficiency depends on loading conditions, as well as required performance conditions. Crack spacing, crack width variability, and the load transfer efficiency all must be included to sufficiently describe the failure mode of the slab. Each of these topics, as well as their usage in the modified CRC design, is discussed.

105. Zollinger, D. G. and E. J. Barenberg. 1990. *Continuously Reinforced Pavements: Punchouts and Other Distresses and Implications for Design*. Report No. FHWA/IL/UI 227. Illinois Department of Transportation, Springfield, IL.

Causes for distress in CRCPs are presented. Primary types of distress leading to a reduction in service life are punchouts and crack spalling. Causes for both the punchout and spalling distresses were determined to be loss of load transfer across transverse cracks. Loss of load transfer over time was correlated with the crack opening and the loss of support due to erosion and pumping of the subbase. Crack opening was correlated with crack spacing. A design procedure is proposed based on the capability of the system to transfer load across the cracks, which in turn is dependent upon crack spacing. Consequently, it is necessary to be able to predict crack spacing before the design procedure can be applied. While crack spacing can be controlled to some extent by the amount of reinforcing steel used and steel placement, the dominant factor in crack spacing appears to be climatic conditions at the time of construction.

106. Zollinger, D. G., N. Buch, D. Xin, and J. Soares. 1999. *Performance of Continuously Reinforced Concrete Pavements. Volume VI - CRC Pavement Design, Construction, and Performance*. Final Report. Report No. FHWA-RD-97-151. Federal Highway Administration, McLean VA.

This report is one of a series of reports prepared as part of a recent study sponsored by FHWA aimed at updating the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRCP. The scope of work of the FHWA study included: (1) conduct a literature review and prepare an annotated bibliography on CRCP and CRCP overlays; (2) conduct a field investigation and laboratory testing related to 23 existing in-service pavement sections to evaluate the effect of various design features on CRCP performance, to identify any design- or construction-related problems, and to recommend procedures to improve CRCP technology; (3) evaluate the effectiveness of various maintenance and rehabilitation strategies for CRCP; and (4) prepare a summary report on the current state-of-the-practice for CRCP. Each of the above four items is addressed in a separate report. This report, Volume VI in the series, addresses several factors associated with CRCP performance, as well as several recently suggested improvements to the design, construction, and evaluation of CRCP.

107. Zollinger, D. G. and J. Soares. 1999. *Performance of Continuously Reinforced Concrete Pavements, Volume VII – Summary*. Final Report. Report No. FHWA-RD-98-102. Federal Highway Administration, McLean VA.

This report is one of a series of reports prepared as part of a recent study sponsored by FHWA aimed at updating the state-of-the-art of the design, construction, maintenance, and rehabilitation of CRCP. The scope of work of the FHWA study included the following: (1) conduct of a literature review and preparation of an annotated bibliography on CRCP and CRCP overlays; (2) conduct of a field investigation and laboratory testing related to 23 existing in-service pavement sections to evaluate the effect of various design features on CRCP performance, to identify any design or construction related problems, and to recommend procedures to improve CRCP technology; (3) evaluate the effectiveness of various maintenance and rehabilitation strategies for CRCP; and (4) prepare a summary report on the current state-of-the-practice for CRCP. Each of the above four items is addressed in a separate report. This report, Volume VII in the series, serves as a synthesis of CRCP performance, design, construction, and rehabilitation.

Drainage

1. Anderson, D. A., R. S. Huebner, J. R. Reed, J. C. Warner, and J. J. Henry. 1998. *Improved Surface Drainage of Pavements*. Final Report, NCHRP Project 1-29. NCHRP Web Document 16. Transportation Research Board, Washington, DC.

The primary objective of this project was to identify improved methods for draining rainwater from the surface of multi-lane pavements. Improved methods for draining water from the surface of multi-lane pavements are needed because of the important role that drainage plays in the mitigation of hydroplaning and tire splash and spray. Because the tendency for hydroplaning and tire splash and spray depend on the thickness of the film of water on the pavement, an interactive computer program (PAVDRN) was developed for

predicting the flow and flow path length of rainwater flowing across the pavement surface. Based on the computer program, surface drainage recommendations were developed (see PTI 1998 reference, number 12, in this section).

2. Baumgardner, R. H. 1993. "Overview of Pavement Drainage Systems." *Western States Drainable PCC Pavement Workshop—Summary Report*. FHWA-SA-94-045. Federal Highway Administration, San Francisco, CA.

   This paper presents an overview of drainable pavement systems for use on PCC pavements. Critical elements of the drainable pavement system include the permeable base (drainage layer), the separator layer, and the edgedrain system. The permeable base must provide adequate permeability to allow the flow of water through the material, sufficient stability to support paving operations, and adequate strength to contribute to the structural design of the pavement. The separator layer is placed between the permeable base and the subbase/subgrade to prevent subgrade soil particles from contaminating the base. The edgedrain system is designed to remove the water from the permeable base and outlet it to the ditches.

3. Cedergren, H. R., J. A. Arman, and K. H. O'Brien. 1973. *Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections.* FHWA/RD-73/14. Federal Highway Administration, Washington, DC.

   This report presents results of interviews with State highway department personnel, field reconnaissance of pavements in nine States, and nine case studies of selected pavements. Problems with existing drainage design concepts and construction methods are noted, showing why many pavements remain saturated for extended periods of time. This saturation contributes to considerable shortening of pavement life. A new method of design is presented to take care of the infiltration of surface water. Inflow-outflow analyses and highway geometrics are included in a procedure using a two-layer graded filter to rapidly remove water entering the structural layers.

4. Christopher, B. R. and V. C. McGuffey. 1997. *Pavement Subsurface Drainage Systems*. NCHRP Synthesis of Highway Practice 239. Transportation Research Board, Washington, DC.

   This synthesis reviews the current practices in pavement subsurface drainage and presents critical design factors and appropriate design methods for pavement subsurface drainage. It focuses on the development of consistent practices in the drainage components of pavement design and discusses the effects of good and poor subsurface drainage. Also reviewed is the impact of decisions in planning, budgeting, procurement, construction, and maintenance on drainage performance. Results of a survey of state transportation agencies on current pavement drainage strategies are interjected throughout the discussion to emphasize the important issues that influence design decisions. Critical subdrainage design details, proper construction techniques, and planned subdrainage maintenance activities are also described to ensure the functionality of the subdrainage system.

5. Daleiden, J. 1998. *Video Inspection of Highway Edgedrain Systems*. Report No. FHWA-SA-98-044. Federal Highway Administration, Washington, DC.

This report documents the results of 287 video inspections of highway edgedrain systems in 29 States. Findings indicated not only that the equipment was quite effective in identifying edgedrain performance concerns, but also the widespread nature of concerns about edgedrain performance. Almost one-third of the systems inspected had nonfunctional outlets, and another one-third either had nonfunctional mainlines or the mainlines could not be inspected due to physical obstructions. Only one-third of the systems inspected were found to be performing as intended. Based on the results of the study, recommendations are provided for edgedrain design improvements to facilitate performance of the systems and their inspections and also to improve quality control during construction. A draft guide specification for video edgedrain inspection is provided in an appendix.

6. Elfino, M. K., D. G. Riley, and T. R. Baas. 2000. *Key Installation Issues Impacting the Performance of Geocomposite Pavement Edgedrain Systems*. ASTM Special Technical Publication 1390.

This paper addresses key installation issues affecting the performance of geocomposite pavement edgedrain systems. This includes maintaining the verticality of the drain panel in the trench; proper positioning of the drain panel within the trench; backfilling with open-graded coarse aggregate; timely installation of outlet fittings and pipe; and the use of outlet pipes with adequate pipe stiffness. Three highway rehabilitation projects involving the installation of approximately 120 km (400,000 linear feet) of geocomposite edgedrains in Virginia and Ohio are investigated, and lessons learned documented in this paper. Actual cost savings, up to 50 percent, were realized from the use of geocomposite edgedrain compared to conventional edgedrains. Conclusions and recommendations are presented in support of successful installations.

7. Federal Highway Administration (FHWA). 1992. *Drainable Pavement Systems, Participant Notebook*. FHWA-SA-92-008. Federal Highway Administration, Washington, DC.

This course notebook presents state-of-the-art guidelines and recommendations on the selection, design, construction, and maintenance of drainable pavement systems. With the focus on rigid pavements, it presents background information on the detrimental effects of moisture on pavement performance and describes a methodology for computing effective drainage times of a pavement structure. Guidelines on the design and construction of both treated and nontreated permeable bases are provided, along with detailed information on separator layer requirements and on longitudinal edge drain design. Recommended drainage maintenance practices are also described.

8. Holtz, R. D., B. R. Christopher, and R. R. Berg. 1995. *Geosynthetic Design and Construction Guidelines*. FHWA-HI-95-038. Federal Highway Administration/National Highway Institute, Washington, DC.

This manual is an updated version of the *Geotextile Design and Construction Guidelines* manual used in the FHWA/NHI training course *Geosynthetics Engineering Workshop*. The

update was performed to reflect current practices and codes for geotextile design, and has been expanded to address geogrid and geomembrane materials. The manual was prepared to enable the highway engineer to correctly identify and evaluate potential applications of geosynthetics as an alternative to other construction methods and as a means to solve construction problems. With the aid of the text, the highway engineer should be able to properly design, select, test, specify, and construct with geotextiles, composite drains, geogrids, and related materials in drainage, sediment control, erosion control, roadway, and embankment on-soft-soil applications. Steepened slope and retaining wall applications also are addressed. Applications of geomembranes and other barrier materials to highway works are also summarized.

9. Koerner, R. M., G. R. Koerner, A. K. Fahim, and R. F. Wilson–Fahmy. 1994. *Long-Term Performance of Geosynthetics in Drainage Applications*. NCHRP 367. Transportation Research Board, Washington, DC.

   This report contains the results of a thorough study of geosynthetics in highway drainage applications. Ninety-one geosynthetics drainage systems in 17 States were exhumed and inspected, with the results incorporated into a database. The performance of the applications was compared against design predictions and construction techniques. These evaluations showed that the existing design methodology is acceptable for granular soils but that the criteria for fine-grained soils may need to be evaluated on a site-specific basis. Also, specific recommendations have been made in regard to the construction practices associated with prefabricated, geocomposite edge drains.

10. Larrivee, S., J. Lafleur, and Y. Savard. 1998. *Hydraulic Behaviour of Geosynthetics in Freezing Conditions*. Proceedings of the International Conference on Cold Regions Engineering 1998. ASCE, Reston, VA.

    In northern climates, subsurface road drainage involves the presence of materials with large pores within the frost-penetrated zone. One road design uses horizontal open-graded drainage layers (OGDL) directly under the pavement and connected to edge drains. In some instances, thin geotextiles with pores smaller than those of OGDL have been used as drainage layers. These drains are subject to extreme weather conditions: the proper evacuation of water from pavement cracks, joints or unpaved shoulders can be hindered if it freezes inside and blocks the system. This paper presents the results of a testing program designed to simulate these conditions using a needle-punched geocomposite. The premise is that the input of heat from the water flowing through the fibers is sufficient to compensate the influx of cold from the confining medium. A polypropylene geocomposite, consisting of coarse filaments sandwiched between two filter layers, was tested. The samples were confined by materials of different thermal conductivities: concrete slab or sand layer with degrees of saturation of 0, 70 and 100 percent. Water was circulated vertically and horizontally through the sample for 30 days inside a freezing cabinet. The applied flow rates of 100 and 600 ml/min per linear meter corresponded to the minimum (winter) and maximum (thaw) observed in a field-test section. Intermittent flows were also applied to simulate alternating daily freeze-thaw conditions. The tests showed that total blockage by ice is unlikely inside vertical edge drain but possible in horizontal blanket layers.

11. Moulton, L. K. 1980. *Highway Subdrainage Design.* FHWA-TS-80-224. Federal Highway Administration, Washington, DC.

This report, in five chapters, provides detailed guidelines on the design of subsurface drainage systems. Chapter I is devoted to a general discussion of the adverse effects of subsurface moisture, the sources of subsurface moisture, and the types of subsurface drainage installations to control this moisture. Chapter II lists the data requirements for analysis and design and presents recommended procedures for assembling these data. Chapter III presents methods and recommended criteria for the control of groundwater and infiltration in pavement structural sections. Chapter IV deals with the more general control of groundwater away from the pavement. Chapter V discusses the construction and maintenance aspects of subdrainage systems.

12. Pennsylvania Transportation Institute (PTI). 1998. *Proposed Design Guidelines for Improving Pavement Surface Drainage*. NCHRP Project 1-29. Transportation Research Board, Washington, DC.

This document provides guidance on how to modify highway pavement designs to minimize the potential for hydroplaning. Recommendations are provided in each of four principal design areas: environmental conditions, surface geometry, pavement properties, and appurtenances. The recommendations must first be field tested and eventually may be considered for inclusion as AASHTO design standards.

13. Ray, M. and J. P. Christory. 1989. "Combatting Concrete Pavement Slab Pumping – State of the Art and Recommendations." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

This paper summarizes the recommendations on drainage and erodibility of concrete pavement foundations presented in the 1987 PIARC publication "Combatting Concrete Pavement Slab Pumping – State of the Art." Classifications were established for criteria that have a major impact on the long-term behavior of concrete pavements, namely: (1) drainage of infiltration water at slab-subbase-shoulder interface, (2) use of low-erodibility materials at interfaces, and (3) optimization of interface drainage and waterproofing of pavements. The gains in service life achieved with improved drainage and low-erodibility materials can be objectively assessed.

14. Wyatt, T., W. Barker, and J. Hall. 1998. *Drainage Requirements in Pavements, User's Manual for Microcomputer Program*. FHWA-SA-96-070. Federal Highway Administration, Washington, DC.

This user's manual provides instructions on the operation of the new Microsoft® Windows®-based microcomputer program entitled DRIP (Drainage Requirements in Pavements). It provides detailed instructions on the operation, use, and application of the software, and also contains general information on drainage design methodology. Example problems are provided to fully illustrate and demonstrate the capabilities of the program; these include performing drainage designs for flexible and rigid pavements, calculating time-to-drain and depth of flow in the drainage layer, performing separator layer and geotextile designs, and performing edgedrain and geocomposite fin drain design.

Joint Design

1. Ambroz, J. K., W. J. Seiler, and M. I. Darter. 1997. *Load Transfer Design and Benefits for Portland Cement Concrete Pavements*. Report No. 96-128-E1. American Highway Technology, Kankakee, IL.

   The highest level of stress and deflection in PCC pavement is found at the joints. In highway pavements, the stress is highest along the longitudinal joints, and deflection is highest at the corners. In airport pavements, the highest stresses may occur at either the transverse or longitudinal joints, depending on the aircraft wheel configuration and the pass-to-coverage ratios. Because the joints experience the highest amount of stress or deflection, cracking, pumping, and faulting usually start at the joints. Several methods have been developed to enhance performance at transverse and longitudinal joints. Some of the more common methods are increasing slab and base course thickness to improve aggregate interlock, protecting the base and subgrade against water intrusion, installing permeable bases, reducing joint spacing, and installing load-transfer devices. Industry practice and research have determined that smooth, round, corrosion-resistant dowel bars are typically most effective in maintaining load transfer throughout the life of a pavement. This guide provides a summary of the benefits and design procedures applicable when dowel bars are used as a load-transfer device.

2. American Concrete Pavement Association (ACPA). 1991. *Design and Construction of Joints for Concrete Highways*. TB–010.0 D. American Concrete Pavement Association, Arlington Heights, IL.

   This publication addresses the design and construction of joint systems for concrete highway pavements (which typically range in thickness from 200 to 350 mm (8 to 14 inch)). The need for joints in concrete pavements is first discussed, including a description of the mechanisms of natural crack development due to thermal and shrinkage stresses. The various types of joints are described, and special emphasis is placed on the design of transverse joints, including recommendations for spacing, skewing, load transfer, and construction (dowel placement, sawing, sealing). The design and construction of other joint types (construction joints, expansion joints, longitudinal joints) are also described.

3. Bischoff, D. L. 1996. *Random Skewed Joints With and Without Dowels*. Final Report, WI 85-01. Wisconsin Department of Transportation, Madison, WI.

   The objective of this study was to compare the performance of a nonreinforced concrete pavement with randomly spaced, skewed contraction joints using dowel bars versus one without dowel bars. A control section and a test section, each 1.6 km (1 mi) in length, were incorporated into a highway improvement project located just west of Menomonie, WI, in Dunn County. Construction took place in 1984. The doweled test section has performed well, while the nondoweled control section has experienced progressive deterioration, primarily in the form of faulting. Due to the poor performance, in 1994, after 10 years of service, the entire project length of nondoweled pavement, including the control section, was diamond ground. A recent field survey showed the 12-year-old doweled pavement to be in good condition, while the ground nondoweled pavement is beginning to show signs of reoccurring faulting. It has been estimated that the nondoweled pavement will require

grinding twice to attain a service life equivalent to that of the doweled pavement. The conclusions from this project are as follows: (1) the doweled pavement continues to perform better than the nondoweled pavement; (2) the life of the doweled pavement is estimated to be approximately 2.5 times longer than the nondoweled pavement prior to any maintenance or rehabilitation; (3) the epoxy-coated dowel bars in the test section remained intact (i.e., no corrosion); (4) the use of dowel bars increases initial concrete pavement cost by approximately 7.8 percent; (5) over a 25-year service life, a nondoweled pavement would cost approximately 13.1 percent more than a doweled pavement; (6) the use of dowel bars in concrete pavements currently saves WisDOT approximately $6,000,000 per year; and (7) the employment of dowel bars is a cost effective method of extending the service lives of concrete pavements while enhancing the pavement performance and reducing user inconvenience.

4. Bock, B. T. and P. A. Okamoto. 1989. "Evaluation of Dowel Placement Using a Dowel Bar Inserter." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation*. Purdue University, West Lafayette, IN.

   This paper presents the results of several field investigations to evaluate the effectiveness of automatic dowel bar inserter equipment to properly place dowel bars in rigid pavements. A commercially available radar system capable of locating steel embedded in concrete was used in the evaluation of dowel bar alignment on projects using the dowel bar inserter. Overall, the inserter performed well compared to the basket assembly construction, with dowel depth, misalignment (vertical and horizontal), and longitudinal displacement being comparable between the two methods.

5. Brink, R. H. 1979. *Joint-Related Distress in PCC Pavement—Cause, Prevention, and Rehabilitation*. NCHRP Synthesis of Highway Practice 56. Transportation Research Board, Washington, DC.

   Joints are constructed in concrete pavement to control stresses and cracking and thus to prevent pavement damage and loss of riding quality. Unfortunately, much of the maintenance required for jointed pavements is necessitated by the joint itself. Common joint-related distresses include pumping, spalling, blowups, and faulting, although joint designs or construction practices often may lead to other pavement distresses (e.g., late sawing can lead to slab cracking). This synthesis presents background information on the causes of these common joint-related distress types, and describes recommended design and construction practices to prevent their occurrence. Detailed rehabilitation methods for addressing joint-related distresses are also described, ranging from temporary maintenance measures intended to maintain pavement serviceability to permanent rehabilitation techniques in which the joint is restored to a condition where it can perform its intended function for the remaining life of the pavement.

6. Cable, J. K. and L. I. Wosoba. 2000. "Matching Load Transfer to Traffic Needs." *Mid-Continent Transportation Symposium 2000*, May 15-16, 2000. Iowa State University, Ames, IA.

   Current pavement design in Iowa calls for the inclusion of load-transfer dowels in transverse joints in both State and local pavements. The dowels have been included to protect the pavement against faulting of the joints and other forms of distress resulting from erosion of the soils from beneath the joints. Faulting has been found to be present mostly at the outer edges of the driving lane. Iowa Highway Research Board Project TR-420 is directed at the evaluation of placing alternative numbers of dowels in the transverse joints of the pavement. One rural and one urban pavement were selected for the test sites on county highways near Creston, IA. The sites include subsections containing zero dowels in the transfer joint, three or four dowels in the outer wheel path only, and a full basket of dowels across the joint. This paper discusses the results of deflection testing in both wheel paths in both pavement directions on the rural and urban sections. Fault measurements, joint opening widths, and visual distress surveys have been conducted twice per year on each of the projects. The construction projects are now 12 months old, and the response to load in each case can now be evaluated.

7. Darter, M. I. and K. D. Smith. 1990. "Design of Joints to Control Faulting." *Proceedings, Second International Workshop on the Theoretical Design of Concrete Pavements.* Sigüenza, Spain.

   A procedure to design transverse joints to control faulting has been developed. The procedure is based on two design models (one for doweled and one for nondoweled pavements) incorporating mechanistic concepts and calibrated with field performance data. The models consider many important design factors known to influence joint faulting, including traffic loading, dowel diameter and spacing, joint spacing, temperature factors, subdrainage, base type, subgrade soil, shoulder type, and the concrete thermal coefficient of expansion.

8. Evans, L. D. 1999. *SHRP Joint Study: A Seven Year Look*. Preprint No. 990843. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

   In 1991 and 1992, test sites were constructed to evaluate the performance of joint seal materials and installation methods in new and old concrete. Five joint resealing sites were installed under the Strategic Highway Research Program (SHRP) project H-106 using 12 materials and 4 installation methods. Additionally, 6 new joint sealing sites were installed under the SHRP SPS-4 supplemental testing program using 20 materials and 5 installation methods. Yearly rigorous evaluation of the effectiveness of these seals has been conducted, providing 7 years of performance data regarding adhesion and cohesion failure, spall distress, and compression seal failures.

   This paper summarizes the final analysis results from these studies, providing material effectiveness rankings, life-cycle cost evaluations, installation method rankings, and other performance results.

9. Federal Highway Administration (FHWA). 1990. *Concrete Pavement Joints*. FHWA Technical Advisory T 5040.30. Federal Highway Administration, Washington, DC.

   This advisory provides guidance and recommendations relating to the design and construction of joints in conventional PCC pavements. The various joint types found in PCC pavement are defined; guidelines and recommendations on their use, design, and construction are then presented. Information on transverse contraction joints (spacing, load transfer, joint reservoir design), construction joints (transverse and longitudinal), longitudinal contraction joints, and expansion joints is provided.

10. Federal Highway Administration (FHWA). 1996. *Joint Sealing Roundtable*. Minutes of the December 11–12, 1996 Meeting. Federal Highway Administration, Washington, DC.

    This document summarizes a roundtable discussion of the effects of joint sealing on pavement performance. Various highway agencies and representatives from the FHWA, academia, industry, and the consulting field discussed the relative benefits of joint sealing and exchanged viewpoints. Many questions were brought to the table, and it was suggested that a synthesis-type document be prepared to fully summarize the issues surrounding joint sealing and its cost-effectiveness.

11. Federal Highway Administration (FHWA). 2000. *LTPP Findings Pay Off for Pennsylvania*. Report No. FHWA-RD-00-064. Federal Highway Administration, McLean, VA.

    The Pennsylvania Department of Transportation (PennDOT) decided to change its practice of using skewed joints after reviewing the results of a LTPP program analysis project. The project analyzed LTPP pavement performance data to identify what worked and what didn't work to control the development of joint faulting. As of calendar year 1999, Pennsylvania policy specified perpendicular joints for any limited-access, four-lane concrete pavement highway projects. By changing its pavement joint design standard, PennDOT can reduce the occurrence of joint faulting and realize the following benefits: a smoother ride for motorists; reduced construction problems and related costs; reduced maintenance requirements; and fewer maintenance-related disruptions to traffic.

12. Fuchs, F. and A. Jasienski. 1997. "The Namur Test Road: Behavior of Various Types of Contraction Joints After 17 Years of Traffic." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance*. Purdue University, West Lafayette, IN.

    This paper describes an experiment, conducted by the PIARC Technical Committee on Concrete Roads, on the Namur Test Road in Belgium, to observe the behavior of six different types of contraction joints: sealed and unsealed joints with evenly spaced dowels; sealed and unsealed joints with variably spaced dowels; and sealed and unsealed undoweled joints. These joints were in a concrete pavement constructed on top of an existing asphalt pavement. On average for all joint types, only about one-third of the joints were found to be active, which discounts the validity of predictions of joint widths for individual joints. Nonetheless, the average joint opening over lengths of 20 slabs was consistent with values predicted as a function of thermal expansion and shrinkage. No influence of sealing on joint behavior was observed.

13. Gurjar, A. H., T. Tang, and D. G. Zollinger. 1997. *Evaluation of Joint Sealants of Concrete Pavements*. Report No. FHWA/TX-98/187-27. Federal Highway Administration, Washington, DC.

This project continued investigative efforts of the earlier project 1371. The project completed laboratory tests including relaxation tests, bond-strength tests, and fatigue tests for various sealant materials, and also established a field test site of a variety of joint sealants to monitor performance. The researchers constructed a material behavior model based on finite-deformation viscoelasticity as a function of temperature, deformation, and age effects. They studied the correlation between natural and artificial weathering and proposed a sealant-performance model. Based on this model, this report provides a procedure for estimation of the service life of a sealant in concrete pavement joints. Material and pavement engineers can use this procedure for design and maintenance purposes. This report also proposes a specification and a test protocol for joint sealant materials, which incorporate performance-prediction procedures. The proposed guidelines can be verified and refined through continuous observation of the field test sections. Project researchers expect that application of the research products will lead to an improved sealant selection process and reduced maintenance cost for concrete pavements.

14. Hammons, M. I. and A. M. Ioannides. 1997. "Mechanistic Design and Analysis Procedure for Doweled Joints in Concrete Pavements." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance*. Purdue University, West Lafayette, IN.

This paper provides a rational, mechanistic method for analysis, design, and evaluation of doweled joints in concrete pavements. The required inputs to the analytical model are the slab thickness, modulus of subgrade reaction, and the radius of the loaded area. All other model inputs can be set at default values or modified at the designer's discretion. Dowel bar diameters and spacings can then be interactively modified by the designer to yield a given level of load transfer capability at the joint. The same relationships can be used to evaluate the load transfer efficiency of in-service joints by entering FWD-measured joint deflections. The method can be used to backcalculate joint material and structural properties, as well as stress load transfer at the joint. The design and analysis procedures presented in this paper ignore the effects of curling and warping. Obviously, daily and seasonal temperature and moisture cycles have a significant influence on pavement response. Further investigation of the effects of environmentally induced responses is needed.

15. Ioannides, A. M. and G. T. Korovesis. 1990. "Aggregate Interlock: A Pure-Shear Load Transfer Mechanism." *Transportation Research Record 1286*. Transportation Research Board, Washington, DC.

A finite-element investigation was made of the behavior of jointed or cracked pavement systems equipped with a pure-shear load transfer mechanism, such as aggregate interlock. Dimensional analysis was used in the interpretation of the data, leading to a general definition of the relative joint stiffness of the pavement system in terms of its structural characteristics. Results of this study were verified by comparisons with earlier published field, laboratory, and analytical information. The investigation demonstrated that deflection load transfer efficiency is related to stress-load transfer efficiency, and that this relationship is sensitive to the size of the applied load (or to the gear configuration). A simple backcalculation procedure is outlined to evaluate the in situ joint stiffness of such pavements. Pure-shear load transfer devices are shown to be particularly desirable under a combined externally applied and thermal loading condition because they offer no additional restraint to longitudinal curling.

16. Ioannides, A. M., Y. H. Lee, and M. I. Darter. 1990. "Control of Faulting Through Joint Load Transfer Design." *Transportation Research Record 1286*. Transportation Research Board, Washington, DC.

    This paper describes and evaluates the development of mechanistic-empirical algorithms for more realistic estimates of anticipated faulting in concrete pavements. Earlier theoretical investigations are considered, interpreted through more recent finite-element analysis results, and calibrated using an extensive database of field observations. A factor influencing faulting is the dowel-concrete bearing stress, for which an improved method of determination is presented. A procedure is outlined for assessing the need for dowels in both plain and jointed reinforced concrete pavements, and determining the bar diameter needed to prevent significant faulting. Application of the procedure is facilitated through use of the program PFAULT, which can be implemented on a personal computer.

17. Jiang, Y. J. and M. I. Darter. 1997. "Prevention of Joint Faulting Based on Field Performance Modeling." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance*. Purdue University, West Lafayette, IN.

    This paper describes the development of faulting prediction models for doweled and undoweled joints, based on mechanistic concepts as well as analysis of field data. Site conditions (traffic, climate, and subgrade) and several design features (dowel diameter, subdrainage, joint spacing, base type, and slab widening) were found to enter significantly into the faulting prediction models.

18. Kelleher, K. and R. M. Larson. 1989. "The Design of Plain Doweled Jointed Concrete Pavement." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

    This paper summarizes findings of past research and experience in the United States and foreign countries in the design of doweled JPCP. The need for dowels on medium and heavy truck traffic routes is emphasized. Recommendations are given on the design of doweled joints, dowel diameter, dowel length, dowel spacing, bondbreaking, corrosion protection, contraction joint spacing, dowel alignment tolerances, and dowel placement.

19. Larralde, J. "Feasibility of Class C FRP Load Transfer Devices for Highway Jointed Concrete Pavements." *Serv Durability Construction Materials, Proceedings of the First Materials Engineering Congress*. Published by ASCE, Boston Society of Civil Engineers Section, Boston, MA.

    The objective of this paper is to analyze the feasibility of using corrosion-free fiberglass reinforced plastic (FRP) devices in lieu of steel tie bars in the longitudinal joints of highway concrete pavements. The FRP devices are designed to provide the same shear transfer capability as the currently used steel tie bars. FRP devices consisting of bars, channel, and I-beam shapes are considered. In terms of cross-sectional area, the amount required for FRP devices is greater than that for steel bars because the modulus of elasticity of the FRP is lower than that of steel. In terms of cost of materials, FRP devices are more expensive than steel tie bars. However, prevention of deterioration due to corrosion may extend the service life of the joints and therefore that of the pavement. More research is needed to accurately define the increase of service life when corrosion is prevented.

20. Larson, R. M. 1990. "The Need for Dowel Bars in Jointed Concrete Pavements." *Proceedings, Second International Workshop on the Theoretical Design of Concrete Pavements.* Sigüenza, Spain.

    FHWA has been involved in a number of research activities to determine the need for dowel bars in jointed PCC pavements. Recent activities are summarized, and the most recent models developed to predict transverse joint faulting of doweled and nondoweled pavements are presented. The faulting models are recommended for use in checking the adequacy of proposed project designs. Future research activities pertaining to load transfer design are also discussed.

21. McGhee, K. H. 1995. *Design, Construction, and Maintenance of PCC Pavement Joints*. NCHRP Synthesis of Highway Practice 211. Transportation Research Board, Washington, DC.

    PCC pavements require joints to control the natural cracking associated with shrinkage caused by drying and with movements caused by changes in temperature and moisture conditions. This report records the state of the practice with respect to the design, construction, and maintenance of PCC pavement joints. An overview of concrete pavement jointing is presented, including a description of current practices used by highway agencies. This is followed by general joint design considerations, such as load transfer needs, joint spacing requirements, and joint reservoir and sealant design. A discussion on current joint construction practices and quality control considerations is also provided to illustrate critical construction requirements, and a summary of recommended joint repair and maintenance practices is presented.

22. Moore, R. K. 1994. Analysis, Design, and Construction of Transverse Joint Load Transfer Systems for Rigid Pavements. KU-93-6. Kansas Department of Transportation, Topeka, KS.

    This report discusses the evolutionary developments in mechanistic dowel behavior theory over the past 70 years. New findings relating to dowel bar behavior obtained from finite-element modeling are described. A sampling of pavement performance models that use empirical or mechanistic-empirical statistical regressions to estimate load transfer performance (expressed as joint faulting) is presented. The findings from a limited number of field performance and laboratory studies are summarized. Recommended construction procedures associated with transverse joint construction are also presented.

23. Morian, D. A. and S. M. Stoffels. 1998. *Joint Seal Practices in the United States: Observations and Considerations*. Preprint No. 981346. Seventy-Seventh Annual Meeting of the Transportation Research Board, Washington, DC.

In the past few years, the subject of sealing joints in rigid pavements has become a rather controversial issue. Many highway agencies have made or are considering significant modifications to the rigid pavement joint sealing practices, namely leaving joints unsealed or sealing narrow joint sawcuts. This paper reviews the current rigid pavement joint sealing practices and summarizes the most recent information on the effectiveness of joint sealing. It is observed that, in terms of subgrade erosion, joint sealing may not be cost effective in pavement locations with free-draining, coarse-grained subgrade material, or in a location where pavement joints are thermally locked. However, neither of these cases adequately addresses the issue of reducing the infiltration of pavement compressibles and the resultant buildup in expansive pressures that can lead to joint spalling and blowups.

24. Morian, D. A., N. Suthahar, and S. Stoffels. 1999. *Evaluation of Rigid Pavement Joint Seal Movement*. Preprint No. 991497. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

The subject of sealing concrete pavement joints has been studied for many years, and a wealth of technology exists for successfully installing pavement joint seals. However, in practice, a great deal of inadequate performance has been observed by highway agencies in the United States in recent years. One primary reason for the observed problems is inadequate control of construction processes. Another very significant factor affecting the performance of joint seals is climatic conditions. This paper examines the effects of climate on the movement of rigid pavement joints.

25. Shober, S. F. 1997. "The Great Unsealing: A Perspective of Portland Cement Concrete Joint Sealing." *Transportation Research Record 1597*. Transportation Research Board, Washington, DC.

Joint and sealant studies of PCC pavements must address whether joint sealing enhances total pavement performance and is cost effective, and, if so, what sealant system should be used. WisDOT has been studying the effect of PCC joint/crack sealing on total pavement performance for 50 years. By 1967, there was substantial documentation that filling and refilling of contraction joints had no beneficial effect on pavement performance. By 1984, it was concluded that pavements with unsealed joints had better overall performance (distress, rid, materials integrity) than pavements with sealed joints. In 1990, WisDOT passed a policy eliminating all PCC joint sealing in new construction and maintenance. This "no seal" policy has saved the State $6,000,000 annually with no loss in pavement performance and with increased customer safety and convenience. The entire PCC sealing issue is beginning to be addressed at the national level, ensuring no false assumptions and with the customers' needs in view.

26. Steffes, R. 1999. *Preformed Phoenix Ethylene Propylene Diene Monomer (EPDM) Compression Joint Seals*. Final Report. Report No. MLR-93-2. Iowa Department of Transportation, Ames, IA.

There is an ongoing drive toward improvements and achieving success in effective and long-term sealing of PCC pavement contraction joints. A variety of joint sealing products and procedures have been applied in Iowa in search of improvements in seal performance. Hot-poured rubberized asphalt products were mainly used for sealing all joints in earlier years for highways. In the 1980s, silicone sealant products were becoming popular, especially for major highways. As a high level of sealant performance was not achieved from silicones in Iowa conditions, other sealing products were tried. Preformed neoprene compression seals are being tried as a substitution for silicone sealants. Due to high costs of materials and installation with neoprene seals, the search for improvements through other joint sealing products and procedures continued. An agreement was made to provide and install preformed ethylene propylene diene monomer compression joint seals. The research site was a 183 m (600 ft) test section of northbound I-29 in Pottawattamie County, IA. Seal installation was done on August 20, 1992. Seal performance has been good over the past 7 years and the seals still show no significant signs of decreasing performance.

27. Tayabji, S. D. 1986. *Dowel Placement Tolerances*. FHWA/RD-86/042. Federal Highway Administration, Washington, DC.

This report presents results of an investigation conducted to develop placement tolerances for dowels in concrete joints. A theoretical analysis of dowel misalignment was attempted, but because of the complexity of incorporating the three-dimensional nature of dowel misalignment, the theoretical analysis was not completed. The effect of dowel misalignment was then investigated in the laboratory by conducting pull-out tests on sections of concrete slabs incorporating dowels with different levels of misalignment. Results of this laboratory testing showed that pullout loads are relatively low for dowel misalignment levels of less than 25 mm per 450-mm (1 in per 18-inch) length of dowel bar.

28. Tayabji, S. D. and B. E. Colley. 1986. *Improved Rigid Pavement Joints*. FHWA-RD-86-040. Federal Highway Administration, Washington, DC.

This report contains the results of a study conducted to improve rigid pavement joint systems. The study included a literature review, development of a finite-element analysis computer program, laboratory testing, and evaluation of various methods to improve rigid pavement joints. The conclusion: solid steel dowel bars are the most cost effective load-transfer device, and nonuniformly spaced dowel bars are recommended to be used on a trial basis. The use of tied PCC shoulders and widened PCC slabs is also recommended to improve joint performance.

29. Wu, C. L., J. W. Mack, P. A. Okamoto, and R. G. Packard. 1993. "Prediction of Faulting of Joints in Concrete Pavements." *Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation*. Purdue University, West Lafayette, IN.

In 1984, the Portland Cement Association revised its concrete pavement thickness design procedure for highways and streets to incorporate consideration of erosion as well as fatigue. The erosion criteria were added because heavy axle loads cause deflections at slab edges and corners and edges, which cause pumping, erosion, voids, and faulting. The erosion criteria correlated well to all of the in-service performance data available at that time, including data from the AASHO road test and several States. These criteria were viewed as first-generation general guidelines that could be refined as field performance data became available on specific climatic and drainage conditions. The objective of this study described in this paper was to collect performance data that have become available since 1984 and refine the existing erosion criteria to represent different climatic areas and drainage conditions.

30. Voigt, G. 1995. "Joint Sealant Considerations for Transverse Contraction Joints in Concrete Highway Pavements." Paper Presented at the 1995 Annual Meeting of the American Concrete Pavement Association. American Concrete Pavement Association, Skokie, IL.

This report discusses the use of joint sealants and provides background information for considering joint sealing in concrete pavements. It contains the current practices of each State and some new performance information on hot-pour, silicone, and preformed compression seals. The report also defines the primary considerations that would accompany elimination of joint sealing. Further investigation and deliberation beyond this report are necessary to reach a final conclusion.

31. Voigt, J. April 1998. "Overview of Joint Sealants for Concrete Pavements." *Public Works*. Volume 129, Issue 4.

Joint sealant is a material that minimizes both infiltration of surface water and incompressible material into the joint system. Sealants also reduce the potential for dowel bar corrosion by reducing the entrance of de-icing chemicals. The other function of joint sealants is to prevent incompressible material from entering the joint reservoir. Many advances have been made in joint sealant materials; the current research determined whether they improve pavement performance. Common joint sealants in pavement applications include hot-pour liquid sealants, silicone sealants, and compression seals.

Maintenance

1. Ceran, T. and R. B. Newman. 1992. *Maintenance Considerations in Highway Design*. NCHRP Report 349. Transportation Research Board, Washington, DC.

This report contains a summary of the current practice for considering maintenance concerns in the highway design process, including those involved in pavement design. Strengths and weaknesses of the various methods are described and areas needing particular improvement are identified. A process designed to explicitly recognize the maintenance implications of designs and design elements is presented, along with the demonstration results gained from the trial implementation of the process in an SHA. Finally, common highway design details that create significant maintenance problems are listed with recommended solutions to accommodate maintenance concerns.

2. Geoffroy, D. N. 1996. *Cost-Effective Preventive Pavement Maintenance*. NCHRP Synthesis of Highway Practice 223. Transportation Research Board, Washington, DC.

This synthesis describes the state of the practice with respect to setting a coherent strategy of cost-effective preventive maintenance for extending pavement life. It discusses the practices of State, local, and Provincial transportation agencies that are attempting to minimize life-cycle costs of pavements and are identifying, during the design of pavement rehabilitation, reconstruction, or construction projects, the future preventive maintenance treatments and the timing and funding for those treatments. It includes a review of domestic literature and a survey of current practices in North America. The appendices include a primer on pavement design and construction, the benefits of preventive maintenance of pavements, a summary of the questionnaire data collected, a simulation of pavement management strategies, and an example process to demonstrate the cost-effectiveness of preventive maintenance.

3. O'Brien, L. G. 1989. *Evolution and Benefits of Preventive Maintenance Strategies*. NCHRP Synthesis of Highway Practice 153. Transportation Research Board, Washington, DC.

This report describes the preventive maintenance practices of several highway agencies, along with the rationale for these practices. It covers the history of preventive maintenance and describes critical issues such as funding considerations, strategy development, and cost analysis. Recommendations regarding current practices and future funding schemes are also included.

4. Permanent International Association of Road Congresses (PIARC). 1992. *Evaluation and Maintenance of Concrete Pavements*. Report 07.04.B. Permanent International Association of Road Congresses, Paris, France.

Well-designed and well-constructed concrete roads generally require little maintenance, which is one of their main assets. However, maintenance is still required for the road's design life to be fully realized. Furthermore, inadequate designs or poor construction practices can hamper the performance capabilities of the concrete pavement, thereby further necessitating the need for timely and effective concrete pavement maintenance. Because it has been proven that the durability of concrete structures depends on the quality of the maintenance, it is now advisable to locate as early as possible those areas exhibiting evidence of wear and fatigue. Procedures to assist in this identification process are included in this document. In addition, a range of diagnostic and rehabilitation equipment is now available for concrete pavements, and can be used to assess and predict the performance of a network or a part of a network. Various solutions are available to improve or repair deteriorated concrete pavements and are presented in this document.

5. Peshkin, D. G., K. D. Smith, K. A. Zimmerman, and D. N. Geoffroy. 1999. *Pavement Preventive Maintenance, Reference Manual*. FHWA-HI-00-004. Federal Highway Administration, Washington, DC.

This document serves as the participant's reference manual for a FHWA/NHI training course on pavement preventive maintenance. Preventive maintenance, often summed up as

"applying the *right* treatment to the *right* pavement at the *right* time," is becoming increasingly popular in highway agencies interested in overall pavement preservation. The objectives of this manual and course are to introduce the components of a pavement preventive maintenance program, to define potential treatment techniques and materials, to describe the relationship between pavement management and pavement preventive maintenance, and to explain cost/benefit concepts of preventive maintenance to decision makers.

The material is organized into seven modules that are intended to meet the above-stated objectives. The first module is an overview of pavement preventive maintenance. This is followed by background information on the current status of preventive maintenance, appropriate definitions, objectives of preventive maintenance programs, and barriers to success. The next module introduces the most commonly used maintenance treatments for both asphalt concrete surfaced and PCC pavements. Because economic analyses are so important to evaluating the cost-effectiveness of treatments, a module on cost analyses is included.

Reinforcement

1. American Society of Civil Engineers. 1995. "Coating Protection for Reinforcement: State of the Art Report." American Society of Civil Engineers. Thomas Telford Limited, London, England.

This expert report provides a comprehensive survey of hot-dip galvanizing and epoxy-or-PVC-coating protection systems for steel reinforcement. It examines influences on materials and application in manufacturing, performance in concrete environments, and practical experience. It also offers guidance on the choice of protection systems.

2. Iwama, S. 1964. *Experimental Studies on the Structural Design of Concrete Pavement*. Public Works Research Institute, Volume 117. Ministry of Construction, Tokyo, Japan.

This report documents the performance of experimental concrete pavements in Japan. A conventionally designed jointed reinforced pavement has been constructed that contains a deformed bar running longitudinally near the outside edge of the slab. These pavements have shown very good performance, with the additional bar controlling crack widths and crack deterioration.

3. Kunt, M. M. and B. F. McCullough. 1990. "Evaluation of the Subbase Drag Formula by Considering Realistic Subbase Friction Values." *Transportation Research Record 1286*. Transportation Research Board, Washington, DC.

A modification of the reinforcement formula that considers the realistic frictional characteristics of subbase types is presented. The objective of this study is not to abandon the current formula but to arrive at a better formula, one that considers the field observations. Rational reinforcement design is important because the amount of reinforcement affects the restraint on the movement of a pavement section, or slab, and the long-term performance. The reinforcement formula was modified in accordance with the experimental results obtained concerning subbase frictional resistance. The new formula

represents the actual components of frictional resistance at the interface: adhesion, bearing, and shear. The formula calculates the steel requirement for the middle of the slab; in other words, the calculated value is the maximum requirement, and the locations between the free end and the middle of the slab will require less reinforcement. Further experimental study is necessary to calibrate the new formula.

4. McCullough, B. F. 1977. "Design Procedure for CRCP Based on Laboratory and Field Observations." *Proceedings, First International Conference on Concrete Pavement Design*. Purdue University, West Lafayette, IN.

   In this paper, a CRCP design procedure is presented along with the necessary limiting criteria. This procedure is derived from theoretical and empirical models developed in connection with numerous field and laboratory studies. The first basic concept is to provide adequate longitudinal steel considering the specific conditions of a project to achieve a desirable crack width, crack spacing, and steel stress. These limiting criteria are shown in this paper. A check is also made to limit spalling and other distress manifestations from occurring. The *AASHO Guide for Design of Pavement Structures* is recommended for pavement thickness design.

5. McCullough, B. F. and C. L. Saraf. 1989. "Effect of Coarse Aggregates on the Design and Performance of CRCP in Texas." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation*. Purdue University, West Lafayette, IN.

   This paper describes a study that was conducted to investigate the effect of coarse aggregate types on the performance of CRCP built in Texas. Long-term performance studies of CRCP in Texas show that performance is a function of coarse aggregate type. A laboratory-testing program was conducted to determine the effects of coarse aggregate type on concrete properties. Hypothetical pavement sections varying in thickness and steel contents were then analyzed with a mechanistic model to obtain estimates of performance. From the laboratory and performance analyses, reinforcing steel design charts were developed for pavements built with either limestone or siliceous river gravel aggregates.

6. Nakamura, T. and T. Iijama. 1994. "Evaluation of Performance and Structural Design Methods of Cement Concrete Pavements in Japan." *Proceedings, Seventh International Symposium on Concrete Roads*. Vienna, Austria.

   This report reviews the performance of concrete pavements constructed in Japan and also reports on available structural design methods. The performance of reinforced and nonreinforced pavements is evaluated and the suitability of current structural design methods to the design conditions in Japan is explored.

7. Ozyildirim, C., C. Moen, and S. Hladky. 1997. "Investigation of Fiber-Reinforced Concrete for Use in Transportation Structures." *Transportation Research Record 1574*. Transportation Research Board, Washington, DC.

   Results are presented of a laboratory investigation to determine the properties of fiber-reinforced concretes (FRCs) with steel (hooked-end), polypropylene (monofilament and fibrillated), and the recently introduced polyolefin fibers (monofilament) for application in pavements and bridge deck overlays. Concrete properties in the unhardened and hardened states were evaluated and compared. Although the ultimate splitting tensile strength, compressive strength, and first-crack strength were higher in most of the FRCs, when strength values were adjusted for changes in air content, only a few batches had higher strengths. The addition of fibers resulted in great improvements in flexural toughness and impact resistance. Three FRC pavement overlays were applied in Virginia in 1995. The FRCs used in the projects were similar to those used in the laboratory investigation, with similar fiber volumes, types, and sizes. To implement the findings of the study successfully, the performance of the FRC pavement overlays is being monitored.

8. Raja, Z. I. and M. B. Snyder. 1991. "Factors Affecting Deterioration of Transverse Cracks in Jointed Reinforced Concrete Pavements." *Transportation Research Record 1307*. Transportation Research Board, Washington, DC.

JRCP develops transverse cracks as the drying and thermal shrinkage of the concrete is resisted by the friction with the supporting layers. These cracks deteriorate with time and traffic because of the loss of load transfer capacity. However, rapid deterioration of these cracks has been observed on some recently constructed projects. This rapid crack deterioration leads to accelerated maintenance requirements and shortened service lives. A synthesis of factors that may reduce aggregate interlock load transfer is presented and current research efforts aimed at evaluating the relative effects of these factors are described. Test variables selected for study include coarse aggregate type, coarse aggregate gradation, coarse aggregate treatment (virgin, recycled, blended), coarse aggregate source, foundation support, reinforcement content, and type and amount of slab tension.

9. Ramakrishnan, V. and N. S. Tolmare. 1998. *Evaluation of Non-Metallic Fiber-Reinforced Concrete in New Full Depth PCC Pavements*. Final Report. Report No. SD96-15-F. South Dakota Department of Transportation, Pierre, SD.

This final report presents the construction and performance evaluation of a new full-depth pavement, constructed with a new type nonmetallic fiber-reinforced concrete (NMFRC). The mixture proportions used, the quality control tests conducted for the evaluation of the fresh and hardened concrete properties, and the procedure used for mixing, transporting, placing, consolidating, finishing, tining, and curing of the concrete are described. Periodic inspection of the full-depth pavement was done and this report includes the results of these inspections. The feasibility of using this NMFRC in the construction of highway structures has been discussed. The new NMFRC with enhanced fatigue, impact resistance, modulus of rupture, ductility and toughness properties is suitable for the construction of full-depth pavements. However, a life-cycle cost analysis shows that NMFRC is not a favorable choice because of its high initial cost.

10. Snyder, M. B. 1994. "Effects of Reinforcement Design and Foundation Stiffness on the Deterioration of Transverse Cracks in Jointed Reinforced Concrete Pavements." *Third International Workshop on the Design and Evaluation of Concrete Pavements,* Krumbach, Austria.

Research was conducted to evaluate the relative effects of several factors on the deterioration of transverse cracks in jointed reinforced concrete pavement. Thirty-five large-scale pavement sections were cracked at an early age and subjected to repeated applications of simulated heavy vehicle loads. Test variables included coarse aggregate type, source, and grading (including use of recycled concrete, slag, and blends of manufactured and natural aggregates); slab tension (simulated varying panel lengths and foundation friction); reinforcing steel type (smooth and deformed wire mesh and deformed bars); reinforcement quantity; and foundation support. Deflection, load transfer, and crack width data were collected at predetermined test intervals. This paper discusses the research findings related to the effects of reinforcement design and foundation stiffness on transverse crack performance. Test results indicate that the deterioration of transverse cracks can be mitigated by providing strong foundation support, thereby reducing the magnitude of the relative vertical displacements of the two slab fragments at the crack. The use of slab reinforcing designs that hold the cracks more tightly closed also provided improved crack performance.

11. Suh, Y. C. and B. F. McCullough. 1994. "Factors Affecting Crack Width of Continuously Reinforced Concrete Pavement." *Transportation Research Record 1449*. Transportation Research Board, Washington, DC.

    Crack width is an important factor affecting the behavior and performance of CRCP. Wide cracks can lead to various pavement distresses, including spalling, punchouts, and steel rupture. Various factors affecting CRCP crack width were evaluated on the basis of width measurements of 208 transverse cracks randomly selected from a series of experimental test sections constructed in Houston. The crack widths were measured at various times and slab temperatures by using a microscope with a graduated eyepiece. It was found from a statistical analysis of the collected data that the factors that significantly affect crack width are construction season, coarse aggregate type, amount of steel, and time of crack occurrence. Thus, hot weather placement produced much wider cracks than cool weather placement. The use of siliceous river gravel resulted in cracks wider than those associated with the use of limestone, and the difference was larger at lower temperatures. The greater the amount of longitudinal steel, the narrower the crack width. Cracks occurring during the first 3 days of construction were significantly wider than those that occurred later. Finally, the effect of crack spacing on crack width was found to be insignificant.

12. Vandenbossche, J. M. 1995. *An Analysis of the Longitudinal Reinforcement in a Jointed Reinforced Concrete Pavement*. M.S. Dissertation. Michigan State University, East Lansing, MI.

    This study identified sources of stress in the longitudinal steel of JRCP, evaluated the effectiveness of current longitudinal design procedures, and included an analysis of the stress state of the longitudinal steel that can be used to check the adequacy of any reinforcing design. The analysis revealed that even pavements containing a sufficiently high amount of longitudinal reinforcing, such that all stresses are within their allowable limits, are failing prematurely. One reason for this may be because neither current design procedures nor the proposed mechanistic analysis considers fatigue of the steel. A mechanistic design procedure that accounts for fatigue can be developed after the effects of multi-axle loading and varying load magnitudes are determined.

13. Xu, P., C. Yi, C. M. Fan, and R. C. Joshi. 1998. *Performance of Fiber-Reinforced Concrete with Respect to Frost Resistance: A Case Study*. Proceedings of the International Conference on Cold Regions Engineering 1998. ASCE, Reston, VA.

Results of laboratory and field tests conducted to study the freeze-thaw durability of FRC are presented. Test data suggest that FRC is more resistant to freeze-thaw environment than plain concrete, irrespective of the number of cycles. The short metallic fibers apparently contribute to frost resistance of FRC due to their positive effect on the bond strength between fibers and matrix. A prototype FRC slab pavement constructed in the field exhibited good performance. Visual inspection of the prototype pavement indicated that FRC is sufficiently durable and resistant to spalling and scaling by frost action commonly observed in plain concrete pavement in cold regions of China.

14. Zollinger, D. G. and E. J. Barenberg. 1990. "Field Investigation of Punchout Distress in Continuously Reinforced Concrete Pavement." *Transportation Research Record 1286*. Transportation Research Board, Washington, D.C.

Most maintenance activities on CRC pavements are related in one way or another to punchout distress. Over several years of observation of CRC pavement, several symptoms related to the structural aspects of punchout distress have been noted, including, but not limited to: close crack spacing, surface widening and spalling of transverse cracks, development of longitudinal cracking, loss of load transfer, and subbase and subgrade pumping. Literature reviews have elaborated on punchout-related factors with respect to pavement performance. However, the punchout mechanism relating the various factors is not completely defined. Some questions exist concerning the sequence of events leading to the loss of load transfer across transverse cracks as a prerequisite to the development of a punchout. This sequence of events relates to the role reinforcement plays in the punchout process and whether the loss of aggregate interlock requires rupturing of the steel. If rupturing of the steel occurs, the question is whether it occurs before or after the loss of aggregate interlock. Factors related to punchout distress (as noted by this investigation and others) are reviewed and a possible mechanism of punchout distress is addressed.

15. Zollinger, D. G. and E. J. Barenberg. 1990. "Mechanistic Design Considerations for Punchout Distress in Continuously Reinforced Concrete Pavement." *Transportation Research Record 1286*. Transportation Research Board, Washington, D.C.

A study was undertaken at the University of Illinois to develop a mechanistic design approach for CRCP to account for punchout distress. A mechanism relating to the loss of load transfer and the progressive development of punchout-related distress is presented. Analysis procedures, demonstrated to implement the mechanism as a rationally based thickness design procedure for CRCP, suggest that the optimal crack interval is between 0.9 and 1.2 m (3 and 4 ft). Current CRCP design methodologies focus on limiting cracking intervals, crack width, and stress in the reinforcement. Load-transfer mechanisms have not been considered in the limiting design criteria and consequently are not included in these design procedures. These methods attempt to determine the design pavement thickness based on the combined effects of environmental and load-related stress on the final crack spacing, which must be limited to the design cracking criteria. However, past experience has indicated that a certain percentage of crack spacing usually falls below the specified minimum crack interval. These data suggest a greater tendency for punchouts to develop within this lower range of crack spacing. How pavement thickness, percent reinforcement, and crack spacing may be considered with respect to pavement spalling and loss of load transfer in the process of punchout development are outlined.

16. Zwerneman, F. J., R. C. Donahey, H. S. Syed, and S. R. Gunna. 1995. "Cracking of Concrete Pavement Continuously Reinforced with Epoxy-Coated Steel." *ACI Materials Journal*. Volume 92, Issue 6, Nov-Dec.

    Two sections of interstate highway in central Oklahoma were recently reconstructed. On both projects, northbound lanes were reinforced with epoxy-coated steel and southbound lanes were reinforced with uncoated steel. Two crack surveys were conducted over the entire length of both sections of new pavement. The results of these surveys show that epoxy coating has no significant effect on crack spacing. Additional variables evaluated include concrete strength, air temperature, and air content. The data indicate that air temperature and air content have the greatest impact on crack spacing. A second phase of the study involved the construction of laboratory specimens with either uncoated or epoxy-coated reinforcement. Cracks were artificially induced in specimens by placing the specimens in tension. No significant differences were measured in either crack spacing or crack width.

Ride Specifications

1. Hancock, J. and M. Hossain. 2000. "An Update on Kansas' Experience with PCCP Smoothness Specifications and Incentives." *Mid-Continent Transportation Symposium 2000*, May 15-16, 2000. Iowa State University, Ames, IA.

    The smoothness or riding comfort of PCC pavements is the highest indicator of quality from the user's perspective. Therefore, the smoothness of newly constructed PCC pavements is of high interest. Since its development in 1990, the Kansas PCC pavement smoothness specification has undergone several revisions. The 1996 revision changed the incentive/disincentive payment from a percent of bid unit cost for the PCC pavement paving basis to a dollar-based value. This revision of the PCC pavement smoothness specification is primarily an attempt to make this smoothness specification more compatible with the asphalt concrete smoothness specification, which has been based on dollar value. This paper primarily outlines the current PCC pavement smoothness specifications in use in Kansas and also updates this development.

2. Karamihas, S. M., T. D. Gillespie, R. W. Perera, and S. D. Kohn. 1999. *Guidelines for Longitudinal Pavement Profile Measurement*. NCHRP Report 434. Transportation Research Board, Washington, DC.

    Most pavement management activities include the use of devices that measure longitudinal profile for assessment of surface roughness. Although technology has been available for measuring longitudinal profile for decades, its still has not fully matured. In fact, a prevailing sense exists in the highway community that if each agency measured the same road with its device, there would be a variety of results. This project sought to improve the accuracy and consistency of roughness measurement through the development of guidelines

for network-level and project-level measurement of longitudinal pavement profile. The resulting guidelines are based on the determination of factors that affect roughness measurements, the quantification of the effect of these factors on repeatability and accuracy, and the determination of how and when these factors can be controlled. The research also sought to explain the underlying causes of common profile measurement problems to assist agencies charged with measuring longitudinal profiles in maximizing the quality of their pavement management system's roughness estimates.

3. Ksaibati, K., R. Staigle, and T. M. Adkins. 1995. *Evaluating the Effectiveness of Pavement Smoothness Specifications*. MPC Report No. 95-37B. Mountain-Plains Consortium. North Dakota State University, Fargo, ND.

   SHAs throughout the United States use smoothness specifications to ensure they are providing the public with quality roads. Some SHAs even provide monetary incentives to contractors for building smoother roads. The effectiveness of such incentive policies, though, has not been studied. This report describes a study to determine whether the initial roughness of a pavement section has any influence on its long-term performance. Statistical and graphical analyses were performed on asphalt and concrete test sections in Wyoming. The findings indicate that the initial roughness of concrete pavements based on the Profilograph Index do not correlate with the long-term performance of these pavements. However, initial roughness measurements based on the International Roughness Index do correlate with future roughness measurements.

4. Smith, K. L., K. D. Smith, L. D. Evans, T. E. Hoerner, M. I. Darter, and J. H. Woodstrom. 1997. *Smoothness Specifications for Pavements*. NCHRP Web Document 1. Final Report, NCHRP Project 1-31. National Cooperative Highway Research Program. Transportation Research Board, Washington, DC.

   This report describes the findings and results of research conducted to determine (1) the effect of initial pavement smoothness on the future smoothness and future life of various pavement types; (2) the effect of smoothness specifications on initial pavement smoothness; (3) the cost-effectiveness of smoothness specifications (including incentives/disincentives); and (4) the most suitable types of smoothness measuring equipment and smoothness reporting indices for use in future smoothness specifications. Detailed analyses of comprehensive time-series smoothness data showed that initial pavement smoothness has a significant effect on the future smoothness of the pavement in 80 percent of new construction (both AC and PCC pavements) and in 70 percent of AC overlay construction. Furthermore, using two different analysis techniques, it was illustrated that added pavement life can be obtained by achieving higher levels of initial smoothness. Combined results of both roughness model and pavement failure analyses indicated at least a 9 percent increase in life corresponding to a 25 percent increase in smoothness from target profile index (PI) values of 0.11 and 0.08 m/km (7 and 5 inches/mi) for concrete and asphalt pavements, respectively. Cost-effectiveness analyses conducted on several pavement families showed that the most cost effective smoothness levels are considerably higher than what is typically used as the current target (PIs between 0 and 0.08 m/km (0 and 5 inches/mile) versus PIs between 0.08 and 0.16 m/km (5 and 10 inches/mi)). Moreover, theoretical pay adjustment functions for the pavement families examined showed greater maximum incentive amounts and more punitive disincentive amounts than current pay adjustment functions. A detailed evaluation of smoothness-measuring equipment resulted in recommended requirements for measured profile wavelengths, sampling interval, distance accuracy, vertical elevation accuracy, and other considerations. Reviews of various smoothness indices, in terms of their ability to correlate with user response and other smoothness indices, resulted in the international roughness index (IRI) and the PI being ranked highest, followed closely by the Michigan DOT ride quality index, the Janoff ride number, and the Sayers ride number.

5. Zaghloul, S. M. 1996. "Effect of Poor Workmanship and Lack of Smoothness Testing on Pavement Life-Cycle Costs." *Transportation Research Record 1539*. Transportation Research Board, Washington, DC.

   Pavement performance is commonly evaluated using the concept of pavement serviceability, in which pavement failure is defined by terminal serviceability instead of strict structural failure. The present serviceability index (PSI), the measure of pavement serviceability, is a function of pavement roughness, cracking, patching, and rutting. Pavement roughness is the major component of PSI and represents more than 95 percent of its value. Because roughness is such as important consideration, changes in roughness control pavement life cycles, and, therefore, construction quality, which influences roughness, performance, and life cycle, as well. A case study of a $120,000,000 project is presented. In this project, poor workmanship and lack of smoothness testing led to a notably high initial roughness. A study was conducted to quantify the long-term effects of the high initial roughness. Results indicate that the pavement service life of the project will be reduced significantly. In addition, a huge increase in the project life-cycle costs is expected. In another study, consideration was given to some of the available smoothness evaluation criteria in which roughness indices, such as the international roughness index and PSI, are used. Results of the study showed that most of these criteria are not capable of adequately addressing the high roughness associated with repeated transverse pumps. Recommendations for overcoming this inadequacy are presented.

Shoulder Design and Performance

1. American Association of State Highway and Transportation Officials. 1998. AASHTO Strategic Highway Safety Plan: A Comprehensive Plan to Substantially Reduce Vehicle-Related Fatalities and Injuries on the Nation's Highways. AASHTO, Washington, DC.

   This strategic safety plan for the Nation's highways is based on a coordinated effort between a host of public and private transportation agencies and other groups interested in highway safety. The objective of the plan is to outline strategies in 17 key emphasis areas that have the potential for saving lives in the magnitude of 5,000 to 7,000 each year, as well as substantially reducing health care costs. Key emphasis areas are young drivers, suspended/revoked drivers, older drivers, aggressive/speeding drivers, impaired drivers, keeping drivers alert, safety belts, pedestrians, bicyclists, vehicle train deaths, motorcyclists, heavy trucks, safety enhancements in vehicles, keeping vehicles on the roadway/minimizing the consequences of leaving the roadway, intersections, work zones, and survivability of severe crashes. One important strategy recommended to help keep drivers alert and to help keep vehicles on the roadway is the retrofitting of rumble strips on the shoulders of rural interstates and other facilities prone to cause driver fatigue.

2. Barksdale, R. D. and R. G. Hicks. 1979. *Improved Pavement-Shoulder Joint Design*. NCHRP Report 202. Transportation Research Board, Washington, DC.

A particularly troublesome part of the pavement-shoulder system is the joint at the interface of concrete mainline pavements and bituminous shoulders. This discontinuity has often permitted undesirable amounts of surface water to reach vulnerable bases, subbases, and foundation soils, causing them to soften, swell, or become adversely affected by freeze-thaw action. Subsequent deterioration of the shoulder surface at the joint, and often extending 0.6 m (2 ft) or more away from the joint, has been a particularly vexing problem.

A wide variety of pavement-shoulder joint systems were reviewed in 15 states and detailed longitudinal joint movements recorded at several sites. These field observations, coupled with the results of other field and laboratory studies, provided the bases for a number of recommendations for longitudinal joint and shoulder design and construction.

3. Benekohal, R. F., K. T. Hall, and H. W. Miller. 1990. "Effect of Lane Widening on Lateral Distribution of Truck Wheels." *Transportation Research Record 1286*. Transportation Research Board, Washington, DC.

Past field studies of lateral distribution of trucks on highway pavements are limited in their relevance to current design practices and truck size limits. In particular, little information is available on the effect of widened concrete slabs on lateral distribution of trucks. The Illinois DOT has constructed test sections of "widened-lane" (or more properly called widened-slab) pavements with 46-cm (18-in and 51-cm (20-in extensions on I-57. Truck wheel placements on these test sections were compared with those observed on nearby conventional 3.65-m (12-ft) pavement slabs, in a study conducted at the University of Illinois. Continuous filming of truck wheel positions was performed with an 8-mm camera mounted on bridges over the highway. Wheel positions were determined to within approximately 12 mm (0.5 in precision by scaling distances measured on the films to known dimensions on the pavements. The mean placement of the wheels of over 900 trucks observed on the control sections was about 56 cm (22 inches) from the slab edge. About 2.5 percent of the wheels passed within 15 cm (6 inches) of the slab edge. On the widened-slab sections, the mean placement of truck wheels was about 5 cm (2 inches) closer to the lane edge (marked by the paint stripe) but still 96 to 102 cm (38 to 40 inches) away from the slab edge. No slab edge loadings were observed among more than 1,300 observations of truck wheel placements on the widened-slab pavement sections. The results suggest that slab widening is likely to be a cost-effective design improvement for concrete pavements that otherwise would be vulnerable to transverse fatigue cracking as a predominant mode of failure.

4. Federal Highway Administration (FHWA). 1990. *Paved Shoulders*. Technical Advisory T5040.29. FHWA, Washington, DC.

This advisory outlines recommended practices for the design and construction of paved shoulders. Guidance is provided on the selection of shoulder types with specific design recommendations presented for both asphalt and concrete shoulders.

5. Griffith, M. S. 1999. *Safety Evaluation of Continuous Shoulder Rumble Strips Installed on Freeways*. Preprint No. 991062. Seventy-Eighth Annual Meeting of the Transportation Research Board, Washington, DC.

Single vehicle run-off-the-road crashes result in approximately one-third of all highway fatalities and one-half million people injured annually, with a societal cost of $80 billion each year. Continuous shoulder rumble strips (CSRS) are one countermeasure used to address this significant safety problem. This study extracted data for two States (California and Illinois) from the Highway Safety Information System to estimate the safety effects of CSRS on freeways. Before-after evaluations of CSRS projects with the use of different comparison groups were conducted. The results from the evaluations estimate that CSRS reduce single-vehicle run-off-the-road crashes on average by 18.3 percent on all freeways (no regard to urban/rural classification) and 21.1 percent on rural freeways. Two potential adverse effects related to safety with CSRS were analyzed: (1) the crash risk that CSRS may present due to driver startle/panic response, and (2) the effect of CSRS on crash migration. Findings show that these potential adverse effects are insignificant.

6. Harwood, D. W. 1993. *Use of Rumble Strips to Enhance Safety*. NCHRP Synthesis of Highway Practice 191. Transportation Research Board, Washington, DC.

Rumble strips provide motorists with an audible and tactile warning that their vehicle is approaching a decision point of critical importance to safety or that their vehicle has partially or completely left the road. Rumble strips can be installed either in the traveled way of a roadway or on the roadway shoulder. This synthesis describes the current state of the practice on the use and effectiveness of rumble strips in both applications. Recommended designs and guidelines on the uses of rumble strips are also provided.

7. Korfhage, G. R. 1988. *Effect of Concrete Shoulders, Lane Widening, and Frozen Subgrade on Concrete Pavement Performance*. Report No. FHWA/MN/RD-88/02. Minnesota Department of Transportation, St. Paul, MN.

This is the fourth and final report of a study to determine the effect of concrete shoulders, lane widening, and frozen subgrade on concrete pavement performance. In this portion of the study, nondestructive deflection testing was conducted over a 2-year period to determine seasonal effects on pavement deflections. Deflections taken during the frozen portion of the year are about one-tenth as great as those taken during the nonfrozen portion of the year. Tied concrete shoulders appeared to reduce edge deflections by about 15 percent. The use of widened PCC slabs reduced deflections at the pavement lane edge by 27 to 46 percent depending on the width of the widening.

8. Majidzadeh, K. and G. J. Ilves. 1986. *Structural Design of Roadway Shoulders—Final Report*. Report No. FHWA/RD-86/089. Federal Highway Administration, Washington, DC.

This report describes the thickness design of roadway shoulders and is based on mechanistic principles of stress-strain analysis. Both flexible and rigid pavements can be designed with this method. The shoulders may be adjacent to either rigid or flexible pavements. All shoulder–pavement type combinations are possible, including the use of a widened rigid mainline lane with a flexible shoulder. The inner and outer edges of the shoulder are designed by computing stress-strain responses due to encroaching or parked vehicles and then applying fatigue distress functions to determine thickness requirements. Drainage design is also considered as part of the shoulder design process.

9. Perrillo, K. 1998. *The Effectiveness and Use of Continuous Shoulder Rumble Strips*. Technical Report. Federal Highway Administration Regional Office, Albany, NY.

   Continuous shoulder rumble strips create noise and vibration that alert wayward drivers that they have wandered from the main traffic lanes. Many highway agencies have adopted CSRS on many paving projects. This report examines the use of CSRS and evaluates their effectiveness.

10. Sehr, M. 1989. *Lateral Load Distribution and the Use of PCC Extended Slabs for Reduced Fatigue*. Final Report. Federal Highway Administration, Portland, OR.

    This paper summarizes data concerning lateral wheel distributions and examines the advantages of extended (or widened) PCC slabs in terms of their effect on stress, strain, deflection, and PCC pavement deterioration. Widened slabs are increasingly used by many highway agencies due to their beneficial effect on pavement performance. Present wheel load distribution was determined to be an average of 50 to 55 cm (20 to 22 inches) away from the edge stripe on 4.6-m (12-ft) shoulders. Detrimental edge loads were found to be reduced significantly at 40 to 50 cm (16 to 20 inches) away from the PCC slab edge. A PCC slab widened 45 to 60 cm (18 to 24 inches) and striped with a 12-ft lane can expect a 20- to 30-percent increase in pavement fatigue life.

11. Transportation Research Board. 1979. Design and Use of Highway Shoulders. NCHRP Synthesis of Highway Practice 63. Transportation Research Board, Washington, DC.

    Shoulder design practices have been surveyed and synthesized in this report. General design information is summarized for the shoulder types used by the SHAs, including cross section design data. Similar performance problems have been noted from various highway agencies, particularly in the sealing and maintenance of the longitudinal joint between a concrete pavement and a bituminous shoulder.

12. Yu, H. T., K. D. Smith, and M. I. Darter. 1995. *Field and Analytical Evaluation of the Effects of Tied PCC Shoulder and Widened Slabs on Performance of JPCP*. Final Report. Colorado Department of Transportation, Denver, CO.

    This study evaluated the effects of widened slabs and tied concrete shoulders on the performance of PCC pavements. Three test sections were constructed on I-70 in 1994 consisting of a widened PCC slab with tied PCC shoulder, a widened PCC slab with nontied PCC shoulder, and a standard-width PCC slab with tied PCC shoulder. After construction, these sections were instrumented with dial gauges and surface-mounted strain gauges to measure the temperature and load-induced deflections and strains. Follow-up testing was conducted with the FWD after 1 year to evaluate the structural response of the pavement sections. Results from the investigation indicated the development of a built-in negative temperature gradient such that the pavement is actually curled up at a zero

temperature gradient. Furthermore, some separation of the PCC slab from the AC base apparently is occurring due to significant differences in the theoretical and measured responses. An analysis of the amount of slab widening to obtain structural benefits revealed that a 6-m (2-ft) widening is appropriate.

Subgrade

1. American Concrete Institute (ACI). 1985. "Guide for Design of Foundations and Shoulders for Concrete Pavements." *ACI Manual of Concrete Practice,* ACI 325.3R-85. American Concrete Institute, Detroit, MI.

   This guide suggests methods for material selection, moisture control, and compaction or treatment of soils and materials to ensure volume stability and uniform support for concrete pavement. Various environments are considered and appropriate methods of subgrade preparation are outlined. Subbase functions are defined and adaptability of types of subbase are discussed. Placement of materials to aid in subbase moisture control is emphasized in shoulder design. A section on recognition of causes of deficiencies in existing pavements is included to alert the engineer to the consequences of improper construction or adverse environment.

2. Asphalt Institute (AI). 1993. *Soils Manual for the Design of Asphalt Pavement Structures*. MS-10. Asphalt Institute, Lexington, KY.

   This manual provides practical guidance on soils inspection, characterization, and strength testing for pavement design purposes. Among the topics covered are the AASHTO and Unified soil classification systems, pedologic soil classification, and the California Bearing Ratio, plate bearing (k value), resistance (R value), and resilient modulus tests.

3. Carpenter, S. H., M. R. Crovetti, K. L. Smith, E. H. Rmeili, and T. P. Wilson. 1991. *Soil and Base Stabilization and Associated Drainage Considerations, Volume I—Pavement Design and Construction Considerations*. Federal Highway Administration, Washington, DC.

   The primary purpose of this manual is to provide background information for engineers responsible for utilizing soil stabilization as an integral part of a pavement structure. The importance of materials properties is discussed, and guidelines provided on the selection of an appropriate stabilizer to use with specific soil types. Construction procedures and equipment for each stabilization method also are described. Information is also included to assist the engineer in evaluating the drainage problems of a pavement. Specific details of drainage design, including permeable base design, are given, along with guidelines on the construction of drainage systems.

4. Darter, M. I., K. T. Hall, and C. M. Kuo. 1995. *Support Under Portland Cement Concrete Pavements*. NCHRP Report 372. Transportation Research Board, Washington, DC.

   This report documents a comprehensive study on the effect of structural support conditions on the performance of PCC pavements. An exhaustive review of the characterization of subgrade support in PCC pavement design is presented, along with an examination of the causes of loss of support beneath PCC slabs. The report also documents several major deficiencies related to concrete pavement support that were found to exist in the current AASHTO design procedure. It was determined that the commonly used "top-of-the-base" k value is unrealistically high. The static, elastic k value of the foundation (subgrade and, if present, fill) is considered the appropriate k value for use in concrete pavement design. Guidelines are provided for three methods for determining k value for design: (1) correlation methods; (2) deflection testing and backcalculation methods; and (3) plate load testing methods.

A three-dimensional finite-element model was developed to analyze the effects of foundation support, base properties, joint spacing, slab thickness, climate, and other factors on concrete pavement performance. The stress analysis results were combined with the AASHO road test stress-performance correlation to develop a proposed revision to the AASHTO rigid pavement design model.

5. Hall, K. T., M. I. Darter, T. E. Hoerner, and L. Khazanovich. 1997. *LTPP Data Analysis, Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction.* FHWA-RD-96-198. Federal Highway Administration, Washington, DC.

Several important issues concerning the effect of slab support on concrete pavement performance were studied under NCHRP Project 1-30. However, the data available for testing and validating the results generated by that study were limited. This study was conducted to further field-verify and develop the improved support guidelines proposed in NCHRP 1-30 using the LTPP database to establish their practicality and appropriateness for use in concrete pavement design nationwide. This study was also conducted to further field-verify the proposed revisions to the AASHTO rigid pavement performance model using the design, materials, traffic, and performance data available in the LTPP database for rigid pavements. Guidelines, revised on the basis of this field verification study, are presented in the appendix in the form of a proposed addendum to the AASHTO design guide. Complete documentation on the field verification efforts is included.

6. Holtz, R. D. 1989. *Treatment of Problem Foundations for Highway Embankments.* NCHRP Synthesis of Highway Practice 147. Transportation Research Board, Washington, DC.

Construction over problem soil areas requires extensive site investigations, environmental impact studies, and detailed comparative design analyses to evaluate possible construction alternatives. These investigations are expensive, but the costs are more than offset by the potential savings in construction costs, avoidance of environmental problems, improved pavement performance, reduced maintenance costs, and so on. A number of construction alternatives are available for treating problem soils, and the characteristics, benefits, limitations, and costs of these alternatives are described in detail. Factors to be considered in selecting an appropriate construction alternative are also provided.

7. Johnson, T. C., R. L. Berg, K. L. Carey, and C. W. Kaplar. 1974. *Roadway Design in Seasonal Frost Areas.* NCHRP Synthesis of Highway Practice 26. Transportation Research Board, Washington, DC.

Highway engineers in seasonal frost areas have been aware for decades of the damaging effects that frost in the underlying support layers can have on pavement performance. Pavement heaving and cracking resulting from frost action and thaw-induced breakups are familiar problems. In the highway environment, the factors of climate, soil, water, pavement structure, and traffic are known to interact in freezing and thawing situations to the detriment of the pavement. The exact nature of the physical processes that take place, the sensitivities of the several contributing factors, and the magnitude of the responses to freezing action are not well understood. Nevertheless, usable criteria exist and are reported in this synthesis for identifying frost susceptibility, and for selecting measures that will avoid harmful pavement reaction.

This report describes and assesses the merit of current roadway design practice in seasonal frost areas. Information is presented on the mechanisms of frost heaving and thaw weakening, the factors that contribute to frost problems, criteria for estimating frost susceptibility, surveying practices for locating areas of detrimental frost action, subgrade treatment and drainage to minimize the effects of frost action, and the structural design of pavements to accommodate the influences of frost action. In addition, it identifies research needs in the area.

8. Laguros, J. G. and G. A. Miller. 1997. *Stabilization of Existing Subgrades to Improve Constructibility During Interstate Pavement Reconstruction*. NCHRP Synthesis of Highway Practice 247. Transportation Research Board, Washington, DC.

This synthesis is intended to provide information to engineers and other transportation officials on methods to evaluate and improve subgrade conditions to meet the constructibility requirements of a reconstruction project. Information was gathered through the use of a questionnaire sent to 65 transportation agencies in the United States and Canada and by a review of pertinent literature. A number of laboratory and in situ testing methods for evaluating subgrade soil properties are presented. In addition to traditional methods of laboratory testing of subgrade soils (e.g., Proctor compaction and California Bearing Ratio (CBR)), there appears to be an increased reliance on nondestructive methods such as the FWD for subgrade evaluation of existing pavements. Furthermore, the dynamic cone penetrometer appears to have great potential for evaluating subgrade stability before, during, and after pavement construction. Detailed information is also presented on the current use of drainage systems (including edge and lateral drains, drainage layers, and geotextile filters) as well as common remediation measures for solving subgrade problems.

9. Portland Cement Association (PCA). 1992. *PCA Soil Primer*. Engineering Bulletin EB007.05S. Portland Cement Association, Skokie, IL.

This introduction to soil properties and characteristics emphasizes how those soil properties influence the design, construction, and performance of concrete pavements. Traditional soil tests, such as Proctor moisture-density tests, triaxial compression test, CBR test, stabiliometer test, and plate load test, are completely described with discussion on the practical meaning and application of each test. Recommended procedures for conducting detailed soil surveys and performing representative soil sampling are also described, highlighted by examples illustrating soil survey and analysis procedures in various highway applications.

10. Thompson, M. R. and B. J. Dempsey. 1977. "Subgrade Soils: An Important Factor in Concrete Pavement Design." *Proceedings, International Conference on Concrete Pavement Design*. Purdue University, West Lafayette, IN.

    The importance of subgrade support evaluation in the design, analysis, and construction of concrete pavement has been demonstrated. Subgrade soil strength and stiffness properties significantly influence the structural response (concrete pavement stresses, surface deflection) and the performance of concrete pavement systems. Moisture and temperature are the major climatic parameters that influence soil strength and stiffness. The effects of moisture and temperature factors on fine-grained soil behavior are documented. The necessity of considering spatial and seasonal variability in subgrade support is emphasized. Techniques for predicting temperature and moisture conditions in pavement systems are described.

11. Transportation Research Board. 1987. *Lime Stabilization—Reactions, Properties, Design, and Construction*. State of the Art Report 5. Transportation Research Board, Washington, DC.

    Many significant engineering properties of soils are beneficially modified by lime treatment. Although lime is primarily used to treat fine-grained soils, it can also be used to modify the characteristics of the fine fraction of more granular soils, to expedite construction, modify subgrade soils, and improve strength and durability of subgrade soils. This document presents the state-of-the-art in lime treatment of paving materials, including a description of the soil-lime reactions, a summary of the properties and characteristics of lime-treated soils, and recommended guidelines for the design and construction of soil-lime mixtures.

12. Transportation Research Board. 1990. *Guide to Earthwork Construction*. State of the Art Report 8. Transportation Research Board, Washington, DC.

    This document provides construction engineers and technicians with information on all aspects of earthwork construction. Although not intended as a design manual, it does contain considerable background information on the design concepts necessary for good earthwork construction. Most sections contain information on specific field problems, and a number of references are included to provide engineers with additional detailed information. Earthwork topics included in this guide are compaction, earthwork construction, drainage, embankment foundations, earthwork for retaining structures, environmental considerations, special soil deposits and embankment materials, and instrumentation for embankments.

Surface Texture and Noise

1. Ardani, A. 1996. "Portland Cement Concrete Pavement Texturing Methods." *Transportation Research Record 1544*. Transportation Research Board, Washington, DC.

   The testing and construction details of nine test sections with varying textural characteristics are described. The effects of the textures on the frictional and noise characteristics of the pavement surface were examined. Skid numbers were acquired according to ASTM E274 with ribbed and smooth tires at 65, 80, and 105 km/hr (40, 50, and 65 mi/hr) for all sections. Six types of texture-measuring devices were used to measure and compare the amount of texture in each section. To examine the noise properties of the test sections, noise data were required in three locations: inside the vehicle, 7.6 m (25 ft) from the centerline, and near the right rear tire of the vehicle. The smooth tire showed more sensitivity to micro- and macrotexture than the ribbed tire, and is recommended as the means of acquiring skid numbers. The sand patch test to measure texture depth showed excellent correlations to smooth-tire skid numbers and is also recommended. Finally, longitudinal textures were found to be much quieter than transverse textures.

2. Ardani, A. and W. Outcalt. 2000. *PCCP Texturing Methods (January 2000)*. Report No. CDOT-DTD-R-00-1. Colorado Department of Transportation, Denver, CO.

   This report documents the noise properties and the frictional characteristics of various concrete pavement textures. It describes the testing and construction details of nine test sections with varying textural characteristics. Included is an overview of the methodologies used to texture concrete pavement surface and a discussion of frictional attributes of various surface textures at different speeds and their impact on noise properties. There is also a description of the state-of-the-art equipment used to acquire sound pressure levels, texture-measuring devices and texture installing equipment, plus a discussion of data acquisition.

3. Drakopoulos, A., T. H. Wenzel, S. F. Shober, and R. B. Schmiedlin. "Crash Experience on Tined and Continuously Ground Portland Cement Concrete Pavements." *Transportation Research Record 1639*. Transportation Research Board, Washington, DC.

   Crash rates were compared between 290 km (180 mi) of continuously ground and 115 km (71 mi) of transversely tined PCC pavements in Wisconsin. All 11,219 reported crashes at the study sites during the 6-year period (1988 to 1993) were analyzed. Continuously ground surfaces were found to have lower crash rates than tined surfaces under dry and wet conditions during daytime and nighttime as well as under all four combinations of pavement and light conditions. Ground pavements had 58 percent the crash rates of tined pavements under dry and wet conditions; the ratio was 84 percent when snow or ice was present on the pavement; however, relatively limited vehicular travel occurred under such conditions and these results are viewed as preliminary. Ground pavements had 57 percent the crash rates of tined pavements during daytime, and 73 percent at night. On the basis of the available data, a hypothesis of increasing crash rates with time (based on frictional property deterioration with pavement age, cumulative vehicle passes since construction, or both) could not be confirmed for either type of pavement texture.

4. Hibbs, B. O. and R. M. Larson. 1996. *Tire Pavement Noise and Safety Performance, PCC Surface Texture Working Group*. Report No. FHWA-SA-96-068. Federal Highway Administration, Washington, DC.

The annoying noise frequencies produced from tire/pavement interaction on some (usually transverse tined) PCC pavements have concerned both nearby residents and traveling motorists. A Technical Working Group (TWG) was formed to investigate the problem by conducting a review of previous research and by evaluating the results of ongoing research. The goal of the TWG was to recommend PCC pavement surface textures that will reduce the annoying noise frequencies without compromising safety.

Noise-reducing construction methods that work most effectively for new pavements are to randomly space (10 to 40 mm (0.4 to 1.6 inch)) the transverse tines/grooves, construct longitudinal tines/grooves (either according to AASHTO guidelines or to the Spanish plastic brushing method), or construct an exposed aggregate surface. Existing PCC pavements that produce an annoying noise should be retextured (diamond grooving, diamond or carbide grinding, or shotblasting) or resurfaced (PCC overlay or surface laminate, microsurfacing, or a dense- or open-graded asphalt concrete overlay).

5. Kuemmel, D. A., J. R. Jaeckel, A. Satanovsky, S. F. Shober, and M. M. Dobersek. 1996. "Noise Characteristics of Pavement Surface Texture in Wisconsin." *Transportation Research Record 1544*. Transportation Research Board, Washington, DC.

Twelve PCC pavement test sections were constructed to compare with standard PCC pavement and AC pavement to quantify the effects of the pavement surface texture on noise, safety, and winter maintenance. Asphalt pavements studied include an SHRP asphalt, stone matrix asphalt, and Wisconsin standard asphalt. A dependency between the pavement textures and their noise characteristics was observed. Noise measurements indicated that uniformly transversely tined PCC pavement created dominant noise frequencies that were audible adjacent to the road and inside the test vehicles. However, careful design and construction of transversely tined PCC pavement can reduce tire-road noise. No significant acoustical advantages of open-graded asphalts over the standard dense-graded asphalt were found.

6. Kuemmel, D. A., R. C. Sonntag, J. A. Crovetti, Y. Becker, J. R. Jaeckel, and A. Satanovsky. 2000. *Noise and Texture on PCC Pavement-Results of a Multi-State Study.* Report Number WI/SPR-08-99. Wisconsin Department of Transportation, Madison, WI.

This paper represents the second phase of a project researching the texture and noise characteristics of PCC pavements. Ten new PCC pavements were built on Highway 29 east of the 1994 Phase I test sites and evaluated; re-evaluations were also conducted on previously evaluated sections in North Dakota, Minnesota, Iowa, Colorado, and Michigan. Interior and exterior noise was measured using the Fast Fourier Transform (FFT) method with a Larson-Davis tow channel real time acoustical analyzer. Texture on all sites was measured with the Road Surface Analyzer (ROSAN). Sand patch tests were also performed.

Generally, the longitudinal tined PCC and asphaltic concrete (AC) pavements exhibited the lowest exterior noise levels. No significant advantage was found regarding exterior noise levels for special textures such as the European, Skidabrader, or diamond ground PCC pavements. The lowest interior noise levels were those of the longitudinal, skewed, European PCC, and AC pavements. The ROSAN mean profile depth and estimated texture depth correlated very closely with the sand patch and friction measurements, especially when analyzed with a certain texture type.

7. Larson, R. M. and B. O. Hibbs. 1997. "Tire Pavement Noise and Safety Performance - PCC Surface Texture Technical Working Group." *Proceedings, Sixth International Purdue Conference on Concrete Design and Materials for High Performance*. Purdue University, West Lafayette, IN.

A TWG representing SHAs, industry, academia, and FHWA met over a 3-year period to update guidance on methods to obtain high pavement surface friction values while minimizing tire/pavement noise. The TWG reviewed the current guidance on surface texture as it relates to safety (friction and vehicle control including effects of splash and spray), noise (inside and outside of the vehicle), drainage (cross slope, effect of longitudinal and transverse tining, and other surface textures), durability, ride (profile), texture quality (measurement), and economy of construction. This paper summarizes the findings as well as the recommendations of the TWG.

8. Wayson, R. L. 1998. *Relationship Between Pavement Surface Texture and Highway Traffic Noise*. NCHRP Synthesis of Highway Practice 268. Transportation Research Board, Washington, DC.

Pavement/tire noise has been studied for well over 30 years; several large databases have been compiled in the past decade. This synthesis is a summary of the research findings on this extensively studied topic. Summaries of selected sample sets are included to allow comparisons of the results and reports. Because the reporting is voluminous, care was taken to include up-to-date reports and those that summarize ideas from groups of individuals. In addition, a survey of current practices was conducted to help guide the synthesis.

9. Wu, C. L. and M. A. Nagi. 1995. *Optimizing Surface Texture of Concrete Pavement*. Research and Development Bulletin RD111T. Portland Cement Association, Skokie, IL.

This report summarizes current and past research and development work related to surface texturing technology. It begins with experiments on skid resistance in the early 1930s in Iowa and Ohio and carries up through the recent developments in exposed aggregate and porous concrete in Europe. Reports and publications are summarized on the work done in this field in the past 60 years with emphasis on concrete pavement friction and tire/road noise issues. Various surface texturing techniques discussed in this report include burlap dragging, transverse tining, longitudinal tining, exposed aggregate, chip sprinkling, surface dressing, and porous.

APPENDIX B. QUESTIONNAIRE SURVEY FORMS

As described previously, two surveys were produced, one targeted at highway agencies (to solicit relative performance data) and one targeted at PCC paving contractors (to solicit relative cost data). This appendix presents the actual questionnaire survey forms used in the conduct of both the relative performance and relative cost surveys.

RELATIVE PERFORMANCE DATA COLLECTION FORM

Purpose

The purpose of this form is to document the effects of various portland cement concrete (PCC) pavement design features on pavement performance. This information will be combined with relative cost information obtained from concrete paving contractors to illustrate the cost-effectiveness of various PCC pavement design features.

This research is being performed for the Innovative Pavement Research Foundation (IPRF) with the support of the Federal Highway Administration (FHWA).

Overview

The underlying concept in this form is that various pavement design features or options are selected by designers to provide a performance benefit. Such a benefit (for example, reduced edge stress, better joint load transfer, or a smoother ride) might be measured in terms of an extension in pavement service life or additional traffic capacity. A standard pavement cross section has been identified, and volunteers are being asked to evaluate the change in life, in terms of the expected increase (or decrease) in 18-kip equivalent single axle loads (ESALs), that the feature provides to the pavement. Your response should be based on experience at your agency. If your agency has no experience with a particular design feature, please indicate this by entering "n/a" in the space provided.

Since each agency uses different design and performance periods, we are asking for your estimate of the performance life of the standard pavement section in ESALs for your State's climate zone. The expected increase (or decrease) in ESALs that the design feature provides should be expressed as a percent increase or decrease from the ESALs that you would expect the standard pavement section to carry.

Information gathered during this process will only be reported in summary form, so individual respondents cannot be identified. Individual responses will be kept confidential.

Instructions

It is estimated that this process will take 4 to 6 hours. At the end of the form, please let us know how long the process took. This information will be used to improve any possible research in this area.

At the beginning of the form, a standard concrete pavement section is described. Changes to each section are also described. After reviewing the standard section and the changes, please estimate the expected ESALs for the standard section and the relative change in performance (in terms of relative increase or decrease in allowable ESALs) attributable to each design feature. If you feel that the feature under evaluation allows the pavement to carry 8 percent more ESALs, it would receive a performance rating of 1.08.

Several States have multiple climate zones as defined in the LTPP research program. The four climate zones identified by LTPP are listed in table 1. For example, a region that has a high potential for moisture throughout the year and low temperatures that are not considered a problem would be classified as a wet-freeze climate zone. If your agency has different design procedures or performance expectations for different climate zones, please complete a form for each zone. Please indicate the climate zone on the form.

Table 1. LTPP Climatic Regions.

LTPP Climatic Regions
Wet Freeze
Dry Freeze
Wet Non-Freeze
Dry Non-Freeze

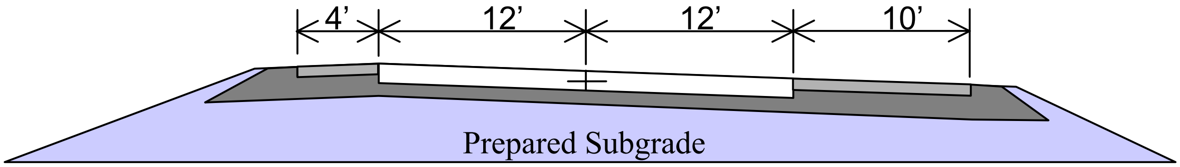
Construction Section

For the purposes of this project, assume the following conditions:

- Rural mainline divided highway construction or reconstruction.
- Average daily traffic (ADT) is 20,000 vehicles per day in each direction with 15% trucks. This is approximately 1,000,000 ESALs per year in the design lane. Assume no growth in ESALs during the life of the pavement.
- Project located within 50 miles of mobilization center.
- 5 miles in length.
- 4 lanes of traffic (2 lanes in each direction).
- The project will be constructed at grade with no earthwork and all paving is on longitudinal slopes less than 5%.
- Construction is performed during normal daytime business hours.
- Standard slipform paving equipment used for construction.
- Portable concrete batch plant will be utilized.
- The concrete for this project is a typical local mix.
- Local environmental regulations will govern the project.
- Local specifications for construction apply for highway work.
- Local load restrictions will be used.

DESIGN FEATURE PERFORMANCE FORMS

Reference/Standard Pavement Section **Performance___________**



- Two lanes, 10-in thick PCC, 12 ft wide lanes tied with #5 deformed tie bars spaced 30 in o.c.
- Transverse joints at 15-ft spacing with no skew and 1.25 in diameter by 18 in long epoxy-coated dowel bars at 12 in o.c.
- Joints have a 1/8 in control sawcut at 1/3 the depth of the concrete slab, a 3/8 in widening sawcut at 1.75 in depth, and backer rod is used before placing a hot-poured asphalt sealant with a 1:1 aspect ratio.
- 6-in dense-graded crushed aggregate base layer.
- Prepared subgrade scarified to a depth of 6 in and recompacted at optimum moisture content.
- Concrete mix design calls for type I cement with a target strength of 650 psi flexural and 4,500 psi compressive (28-day, ASTM C78, third-point loading).
- No drainage layer, no underdrains.
- 6-in asphalt shoulders over 10-in dense-graded aggregate base, 10 ft wide on the truck lane and 4 ft wide on the passing lane.
- Initial smoothness requirements are 7 to 9 inches per mile with no incentive or penalty with a 0.20-in blanking band. Grinding is required on all areas outside of tolerance at no additional cost.

Climate zone: ________

SUBGRADE

Lime Treated Subgrade **Relative Performance___________**
- Eliminate 6-in dense-graded aggregate base course.
- Add 12-in lime stabilized subgrade.

BASE/SUBBASE

Directly on Subgrade **Relative Performance___________**
- Eliminate 6-in dense-graded aggregate base course.
- Pavement constructed directly on prepared subgrade.

Cement Stabilized **Relative Performance___________**
- Eliminate 6-in dense-graded aggregate base course.
- Add 6-in cement stabilized base.

Asphalt Stabilized **Relative Performance___________**
- Eliminate 6-in dense-graded aggregate base course.
- Add 6-in asphalt stabilized base.

DRAINAGE

Open-Graded Aggregate (nonstabilized) Base plus underdrain system

Relative Performance____________

- 6-in open-graded, nonstabilized granular drainage layer.
- 6-in dense-graded, crushed aggregate base layer.
- Open-graded trench edge drains wrapped with geotextile with 6-in flexible pipe and rigid pipe outflows at 500 ft spacing.

Cement-Treated Permeable Base (CTPB) plus underdrain system

Relative Performance____________

- 6-in CTPB layer.
- 6-in dense-graded, crushed aggregate base layer.
- Open-graded trench edge drains wrapped with geotextile with 6-in flexible pipe and rigid pipe outflows at 500 ft spacing.

Asphalt-Treated Permeable Base (ATPB) plus underdrain system

Relative Performance____________

- 6-in ATPB layer.
- 6-in dense-graded, crushed aggregate base layer.
- Open-graded trench edge drains wrapped with geotextile with 6-in flexible pipe and rigid pipe outflows at 500 ft spacing.

CTPB with daylight drainage into ditch (no underdrain system)

Relative Performance____________

- 6-in CTPB layer.
- 6-in dense-graded, crushed aggregate base layer.

ATPB with daylight drainage into ditch (no underdrain system)

Relative Performance____________

- 6-in ATPB layer.
- 6-in dense-graded, crushed aggregate base layer.

THICKNESS/SLAB SIZE

12-inch PCC

Relative Performance____________

- 12-in PCC pavement with 18-ft transverse joint spacing.
- Transverse jointing details remain the same.
- Longitudinal jointing remains the same.

8-inch PCC

Relative Performance____________

- 8-in PCC pavement with 12-ft transverse joint spacing.
- Transverse jointing details remain the same.
- Longitudinal jointing remains the same.

SHOULDERS

16-inch Gravel **Relative Performance____________**

- Gravel shoulder on truck lane and passing lane.

Tied PCC Partial-Depth **Relative Performance____________**

- 6-in tied PCC, partial-depth, with 10-in dense-graded aggregate base.
- Tied shoulder on truck lane, standard shoulder on passing lane.
- 10 ft wide on the truck lane, 4 ft wide on the passing lane.

Tied PCC Full-Depth **Relative Performance____________**

- 10-in tied PCC, full-depth, with 6-in dense-graded aggregate base.
- Tied shoulder on truck lane, standard shoulder on passing lane.
- 10 ft wide on the truck lane, 4 ft wide on the passing lane.

Widened truck lane (24 inches) plus standard asphalt shoulder **Relative Performance____________**

- The truck lane is paved 24 in wider, but striped at the conventional width.

PAVEMENT CROSS SECTION

Trapezoidal Cross Section **Relative Performance____________**

- Trapezoidal cross section 8 in thick at the outside edge of the passing lane and 11 in thick on the outside of the truck lane.

Thickened Edge **Relative Performance____________**

- Thickened edge from 8 in at the centerline of the 2 lanes to 11 in at the outside edge of the passing lane and truck lane.

JOINTS/LOAD TRANSFER

10-inch PCC, Epoxy-Coated Dowel Bars, Skewed Joints **Relative Performance____________**

- 10-in PCC, 15-ft skewed transverse joint spacing, 1.25-in diameter by 18 in long epoxy-coated dowel bars at 12 in on-center. Joints are skewed counter-clockwise at 2 ft per 12-ft lane.

10-inch PCC, No Dowel Bars **Relative Performance____________**

- 10-in PCC, 15-ft perpendicular transverse joint spacing, with aggregate interlock and no dowel bars.

10-inch PCC, Dowel Bars, No Epoxy **Relative Performance____________**

- 10-in PCC, 15-ft perpendicular transverse joint spacing, 1.25-in diameter by 18 in long dowel bars at 12 in on-center. No epoxy coating on the dowel bars.

10-inch PCC, Epoxy-Coated Dowel Bars (reduced number of dowel bars) **Relative Performance____________**

- 10-in PCC, 15-ft perpendicular transverse joint spacing, 1.25-in diameter by 18 in long epoxy-coated dowel bars with 3 bars centered in each wheel path at 12 in on-center.

12-inch PCC, Epoxy-Coated
Dowel Bars **Relative Performance**____________

- 12-in PCC, 20-ft perpendicular transverse joint spacing, 1.50-in diameter by 18 in long epoxy-coated dowel bars at 12 in on-center.

9.5-inch Continuously Reinforced Concrete (CRC),
Epoxy-Coated **Relative Performance**____________

- 9.5-in CRC with #6 deformed bars 8 in on-center longitudinally and 36 in on-center transversely. All steel is epoxy coated.

9.5-inch CRC, No Epoxy **Relative Performance**____________

- 9.5-in CRC with #6 deformed bars 8 in on-center longitudinally and 36 in on-center transversely. The deformed bars have no epoxy coating.

10-inch Jointed Reinforced Concrete Pavement (JRCP),
Epoxy-Coated Dowel Bars **Relative Performance**____________

- 10-in JRCP with W12x W5.5—6x12 welded wire fabric mesh placed on chairs at mid-height. 30-ft perpendicular transverse joint spacing, 1.25-in diameter by 18 in long epoxy-coated dowel bars at 12 in on-center.

JOINT SEALING

Silicone Sealant **Relative Performance**____________

- 15-ft transverse joint spacing with silicone sealant.
- 1/8-in initial control sawcut at h/3.
- 3/8-in widening cut, 1.75 in deep to form joint sealant reservoir.
- Backer rod is used before placing silicone sealant with a 2:1 aspect ratio (w:d).

Preformed Compression Sealant **Relative Performance**____________

- 15-ft transverse joint spacing with preformed compression sealant.
- 1/8-in initial control sawcut at h/3.
- 3/8-in widening cut to form joint sealant reservoir.

No Sealant **Relative Performance**____________

- 15-ft transverse joint spacing with no sealant material.
- 1/8-in initial control sawcut at h/3 with no widening.

Hot-Pour (no joint widening cut) **Relative Performance**____________

- 15-ft transverse joint spacing with hot-pour asphaltic sealant.
- 1/8-in initial control sawcut at h/3 with no widening.
- No backer rod is used before filling joint with hot-pour joint sealant.

Silicone Sealant (no joint widening cut) **Relative Performance**____________

- 15-ft transverse joint spacing with silicone sealant.
- 1/8-in initial control sawcut at h/3 with no widening.
- No backer rod is used before filling joint with silicone sealant.

CONCRETE STRENGTH/MATERIALS

750 psi Flexural Strength **Relative Performance____________**

- Additional strength is obtained by using additional Type 1 cement.

High-Early Strength **Relative Performance____________**

- Opening to traffic within 48 hours (a.k.a. Fast-Track).
- 550 psi flexural strength at 48 hours, 650 psi flexural acceptance strength. (High-early strength is obtained by using Type III cement).

Well-Graded Mix **Relative Performance____________**

- Changing the mix from a gap-graded mix to a well-graded mix. (If the local concrete mix design uses well-graded aggregate, please note that on the survey.)

INITIAL SMOOTHNESS/RIDE

The following table lists several initial smoothness/ride quality ranges and base layer types. For each initial smoothness/ride quality range, please calculate the expected performance increase (or decrease) for the change in base layers. The standard section has been given a performance of "1.0."

Smoothness Criteria, inches/mi. (0.20 in. blanking band)	Aggregate Base	Econocrete/ Cement Stabilized Base	AC Stabilized Base	Open Graded Stabilized Drainage Layer
7 to 9	1.0			
5 to 7				
3 to 5				
1 to 3				
< 1				

Standard Type of Profilograph:_____________________

IMPACT RANKING

General

To complete the analysis portion of this project, please "force rank" the concrete pavement design features in the table below based on their overall importance to pavement performance. Since pavement performance is based on several factors, a ranking is required for each one.

Please rank the pavement design features below from 1 to 10 for each pavement performance measure, with 1 indicating the feature that has the greatest impact and 10 the least. For example, for *Cracking*, if you believe *Subgrade* has the greatest impact, you would enter *1* for that feature under *Cracking*. You would enter *2* for the feature that you believe has the next greatest impact, and so on. There can be no ties, so please do not use the same number twice.

Pavement Design Feature	**Pavement Performance Measures**			
	Cracking	**Spalling**	**Faulting**	**Smoothness***
Subgrade				
Base/Subbase				
Drainage				
Thickness/Slab Size				
Shoulders				
Pavement Cross Section				
Joints/Load Transfer				
Joint Sealing				
Concrete Strength/Materials				
Initial Smoothness/Ride				

* Although smoothness is not a specific performance measurement used in most performance models, smoothness is considered a general indicator of the overall pavement performance. In addition, smoothness is representative of the traveling public's perception of the quality of highways. As such, please rank smoothness along with the more specific performance measures.

What basis did you use for filling out this form:
______Engineering experience/judgment only
______Performance modeling
______Some performance modeling along with judgment

Please provide a short description of your State's standard concrete pavement maintenance activities:

RELATIVE COST FORMS

Purpose

The purpose of this project is to document the construction costs of different design features for portland cement concrete (PCC) pavements. This information will be combined with performance information obtained from State Highway Agencies to illustrate the relationship between the costs of various concrete pavement design features.

This research is being performed for the Innovative Pavement Research Foundation (IPRF) with the support of the Federal Highway Administration (FHWA).

Overview

The underlying concept in this project is that various pavement design features or options are selected by designers to provide a performance benefit. Such a benefit (for example, reduced edge stress, better joint load transfer, or a smoother ride) might be measured in terms of an extension in pavement service life or additional traffic capacity. A standard pavement cross section has been identified, and you are being asked to provide information on the cost of various design features.

Since construction costs vary from State to State, we are asking for your estimate of the relative change in cost. This should be expressed as a percent increase (or decrease) from the expected construction cost of the standard pavement section.

Information gathered during this process will only be reported in summary form, so individual respondents cannot be identified. Individual responses will be kept confidential.

Instructions

Please review the entire form. It is estimated that this process will take 4 to 6 hours to complete. Please let us know how long the process took. This information will be used to improve any possible future projects.

At the beginning of the form, a standard concrete pavement section is described. Changes to each section are also described. After reviewing the standard section and the changes, please calculate the relative change in construction cost that is appropriate for your company and these concrete pavement design features. The standard section is given a cost of "1.0." So, if you feel that the feature under evaluation increases the construction cost of the pavement by 8 percent, it would receive a relative cost rating of 1.08.

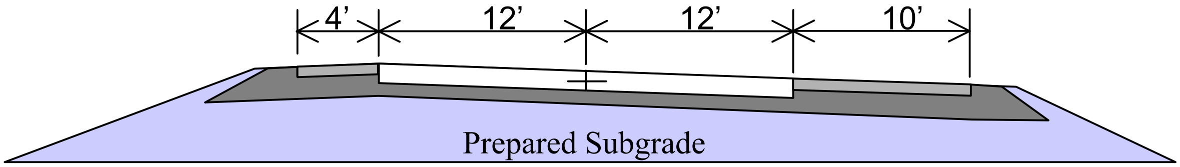
Construction Section

For the purposes of this project, assume the following conditions:

- Rural mainline divided highway construction or reconstruction.
- Project located within 50 miles of mobilization center.
- 5 miles in length.
- 4 lanes of traffic (2 lanes in each direction).
- The project will be constructed at grade with no earthwork and all paving is on longitudinal slopes less than 5%.
- Construction is performed during normal daytime business hours.
- Standard slipform paving equipment used for construction.
- Portable concrete batch plant will be utilized.
- The concrete for this project is a typical local mix.
- Local environmental regulations will govern the project.
- Local specifications for construction apply for highway work.
- Local load restrictions will be used.

RELATIVE COST FORM

Reference/Standard Pavement Section **Relative Cost______1.00___**



- Two lanes, 10-in thick PCC, 12 ft wide lanes tied with #5 deformed tie bars spaced 30 in o.c.
- Transverse joints at 15-ft spacing with no skew and 1.25 in diameter by 18 in long epoxy-coated dowel bars at 12 in o.c.
- Joints have a 1/8 in control sawcut at 1/3 the depth of the concrete slab, a 3/8 in widening sawcut by 1.75 in depth, and backer rod is used before placing a hot-poured asphalt sealant with a 1:1 aspect ratio.
- 6-in dense-graded crushed aggregate base layer.
- Prepared subgrade scarified to a depth of 6 in and recompacted at optimum moisture content.
- Concrete mix design calls for type I cement with a target strength of 650 psi flexural and 4,500 psi compressive (28-day, ASTM C78, third-point loading).
- No drainage layer, no underdrains.
- 6-in asphalt shoulders over 10-in dense-graded aggregate base, 10 ft wide on the truck lane and 4 ft wide on the passing lane.
- Initial smoothness requirements are 7 to 9 inches per mile with no incentive or penalty with a 0.20-in blanking band. Grinding is required on all areas outside of tolerance at no additional cost.

SUBGRADE

Lime Treated Subgrade **Relative Cost____________**

- Eliminate 6-in dense-graded aggregate base course.
- Add 12-in lime stabilized subgrade.

BASE/SUBBASE

Directly on Subgrade **Relative Cost____________**

- Eliminate 6-in dense-graded aggregate base course.
- Pavement constructed directly on prepared subgrade.

Cement Stabilized **Relative Cost____________**

- Eliminate 6-in dense-graded aggregate base course.
- Add 6-in cement stabilized base.

Asphalt Stabilized **Relative Cost____________**

- Eliminate 6-in dense-graded aggregate base course.
- Add 6-in asphalt stabilized base.

DRAINAGE

Open-Graded Aggregate (nonstabilized)
Base plus underdrain system **Relative Cost____________**

- 6-in open-graded, nonstabilized granular drainage layer.
- 6-in dense-graded, crushed aggregate base layer.
- Open-graded trench edge drains wrapped with geotextile with 6-in flexible pipe and rigid pipe outflows at 500 ft spacing.

Cement-Treated Permeable Base (CTPB)
plus underdrain system **Relative Cost____________**

- 4-in CTPB layer.
- 6-in dense-graded, crushed aggregate base layer.
- Open-graded trench edge drains wrapped with geotextile with 6-in flexible pipe and rigid pipe outflows at 500 ft spacing.

Asphalt-Treated Permeable Base (ATPB)
plus underdrain system **Relative Cost____________**

- 6-in ATPB layer.
- 6-in dense-graded, crushed aggregate base layer.
- Open-graded trench edge drains wrapped with geotextile with 6-in flexible pipe and rigid pipe outflows at 500 ft spacing.

CTPB with daylight drainage into ditch
(no underdrain system) **Relative Cost____________**

- 6-in CTPB layer.
- 6-in dense-graded, crushed aggregate base layer.

ATPB with daylight drainage into ditch
(no underdrain system) **Relative Cost____________**

- 6-in ATPB layer.
- 6-in dense-graded, crushed aggregate base layer.

THICKNESS/SLAB SIZE

12-inch PCC **Relative Cost____________**

- 12-in PCC pavement with 18-ft transverse joint spacing.
- Transverse jointing details remain the same.
- Longitudinal jointing remains the same.

8-inch PCC **Relative Cost____________**

- 8-in PCC pavement with 12-ft transverse joint spacing.
- Transverse jointing details remain the same.
- Longitudinal jointing remains the same.

SHOULDERS

16-inch Gravel **Relative Cost____________**

- Gravel shoulder on truck lane and passing lane.

Tied PCC Partial-Depth **Relative Cost____________**

- 6-in tied PCC, partial-depth, with 10-in dense-graded aggregate base.
- Tied shoulder on truck lane, standard shoulder on passing lane.
- 10 ft wide on the truck lane, 4 ft wide on the passing lane.

Tied PCC Full-Depth **Relative Cost____________**

- 10-in tied PCC, full-depth, with 6-in dense-graded aggregate base.
- Tied shoulder on truck lane, standard shoulder on passing lane.
- 10 ft wide on the truck lane, 4 ft wide on the passing lane.

Widened truck lane (24 inches) plus standard asphalt shoulder **Relative Cost____________**

- The truck lane is paved 24 in wider, but striped at the conventional width.

PAVEMENT CROSS SECTION

Trapezoidal Cross Section **Relative Cost____________**

- Trapezoidal cross section 8 in thick at the outside edge of the passing lane and 11 in thick on the outside of the truck lane.

Thickened Edge **Relative Cost____________**

- Thickened edge from 8 in at the centerline of the 2 lanes to 11 in at the outside edge of the passing lane and truck lane.

JOINTS/LOAD TRANSFER

10-inch PCC, Epoxy-Coated Dowel Bars, Skewed Joints **Relative Cost____________**

- 10-in PCC, 15-ft skewed transverse joint spacing, 1.25-in diameter by 18 in long epoxy-coated dowel bars at 12 in on-center. Joints are skewed counter-clockwise at 2 ft per 12-ft lane.

10-inch PCC, No Dowel Bars **Relative Cost____________**

- 10-in PCC, 15-ft perpendicular transverse joint spacing with aggregate interlock and no dowel bars.

10-inch PCC, Dowel Bars, No Epoxy **Relative Cost____________**

- 10-in PCC, 15-ft perpendicular transverse joint spacing, 1.25-in diameter by 18 in long dowel bars at 12 in on-center. No epoxy coating on dowel bars.

10-inch PCC, Epoxy-Coated Dowel Bars (reduced number of dowel bars) **Relative Cost____________**

- 10-in PCC, 15-ft perpendicular transverse joint spacing, 1.25-in diameter by 18 in long epoxy-coated dowel bars with 3 bars centered in each wheel path at 12 in on-center.

12-inch PCC, Epoxy-Coated Dowel Bars **Relative Cost____________**

- 12-in PCC, 20-ft perpendicular transverse joint spacing, 1.5-in diameter by 18 in long epoxy-coated dowel bars at 12 in on-center.

9.5-inch Continuously Reinforced Concrete (CRC), Epoxy-Coated **Relative Cost____________**

- 9.5-in CRC with #6 deformed bars 8 in on-center longitudinally and 36 in on-center transversely. All steel is epoxy coated.

9.5-inch CRC, No Epoxy **Relative Cost____________**

- 9.5-in CRC with #6 deformed bars 8 in on-center longitudinally and 36 in on-center transversely. The deformed bars have no epoxy coating.

10-inch Jointed Reinforced Concrete Pavement (JRCP), Epoxy-Coated Dowel Bars **Relative Cost____________**

- 10-in JRCP with W12x W5.5—6x12 welded wire fabric mesh placed on chairs at mid-height. 30-ft perpendicular transverse joint spacing, 1.25-in diameter by 18 in long epoxy-coated dowel bars at 12 in on-center.

JOINT SEALING

Silicone Sealant **Relative Cost____________**

- 15-ft transverse joint spacing with silicone sealant.
- 1/8-in initial control sawcut at h/3.
- 3/8-in widening cut to form joint sealant reservoir.
- Backer rod is used before placing silicone sealant with a 2:1 aspect ratio (w:d).

Preformed Compression Sealant **Relative Cost____________**

- 15-ft transverse joint spacing with preformed compression sealant.
- 1/8-in initial control sawcut at h/3.
- 3/8-in widening cut to form joint sealant reservoir.

No Sealant **Relative Cost____________**

- 15-ft transverse joint spacing with no sealant material.
- 1/8-in initial control sawcut at h/3 with no widening.

Hot-Pour (no joint widening cut) **Relative Cost____________**

- 15-ft transverse joint spacing with hot-pour asphaltic sealant.
- 1/8-in initial control sawcut at h/3 with no widening.
- No backer rod is used before filling joint with hot-pour joint sealant.

Silicone Sealant (no joint widening cut) **Relative Cost____________**

- 15-ft transverse joint spacing with silicone sealant.
- 1/8-in initial control sawcut at h/3 with no widening.
- No backer rod is used before filling joint with silicone sealant.

CONCRETE STRENGTH/MATERIALS

750 psi Flexural Strength **Relative Cost____________**

- Additional strength is obtained by using additional Type 1 cement.

High-Early Strength **Relative Cost____________**

- Opening to traffic within 48 hours (a.k.a. Fast-Track).
- 550 psi flexural strength at 48 hours, 650 psi flexural acceptance strength. (High-early strength is achieved by using Type III cement.)

Well-Graded Mix **Relative Cost____________**

- Changing the mix from a gap-graded mix to a well-graded mix. (If the local concrete mix design uses well-graded aggregate, please note that on the survey.)

INITIAL SMOOTHNESS/RIDE

The following table lists several initial smoothness/ride quality ranges and base layer types. For each initial smoothness/ride quality range, please calculate the expected cost increase (or decrease) for the change in base layers. The standard section has been given a performance of "1.0."

Smoothness Criteria, inches/mi. (0.20 in. blanking band)	Aggregate Base	Econocrete/ Cement Stabilized Base	AC Stabilized Base	Open Graded Stabilized Drainage Layer
7 to 9	1.0			
5 to 7				
3 to 5				
1 to 3				
< 1				

APPENDIX C. SUMMARY OF DATA COLLECTION RESPONSES

As described in chapter 2, two surveys were produced and distributed under this project, one targeted to SHAs (to solicit relative performance data) and one for PCC paving contractors (to solicit relative cost data). Although these were separate surveys, the pavement design variables presented in each questionnaire are identical; results from each data collection effort thus can be paired directly for analysis.

Both surveys were structured so that only one design feature (from the standard design) was changed at a time, and the survey participants were then asked to assess what effect that change might have in terms of the costs (contractor questionnaire) or relative performance (agency questionnaire). In this way, the relative effects of the change in that one design feature could be determined. As described in chapter 2, all cost and performance changes were collected in terms of *ratios* that were relative to the expected cost and performance of the standard design section (i.e., cost and performance ratios associated with the design features defining the standard pavement section were 1.00). For example, if a contractor participant estimated that a particular design feature change was expected to cost 5 percent more than the original expected cost of the standard pavement section, then the participant defined the associated cost ratio as 1.05. Likewise, if an agency participant estimated that a particular design feature change would result in 3 percent decrease in performance (in comparison to the expected performance of the standard pavement section) then the participant defined the associated performance ratio as 0.97 (i.e., 1.00 – 0.03 = 0.97). Survey respondents were asked to not enter a rating if they had no experience with a particular design feature.

Table 18 presents the raw data collected from the agency (performance) surveys, and also provides a statistical summary (mean, standard deviation, median, and range) of the overall results. Table 19 presents similar information for the contractor (cost) surveys. A blank cell in either of these tables indicates a “no response” from the participant.

Table 18. Raw data and summary of agency (performance) surveys.

Feature Category	Description	Performance Survey Responses 1	2	3	4	5	6	7	8
Subgrade	Lime treated subgrade	0.95	1.00		0.50	0.90	1.05		1.09
Base	No base	0.90	0.85		0.70	0.80	0.90	0.70	0.93
	Asphalt-treated base (ATB)	1.02				0.80	1.05	1.10	1.27
	Cement-treated base (CTB)	1.03			0.75	0.80	1.00	1.05	1.27
Drainage	Open-graded, non-stabilized base (with underdrains)	1.08	1.30	1.20	1.50	1.15	1.25		1.87
	Asphalt-treated permeable base (with underdrains)	1.09	1.40		1.75	1.15	1.40		2.36
	Asphalt-treated permeable base (without underdrains)	1.05	1.30			1.20	1.10		1.76
	Cement-treated permeable base (with underdrains)	1.10	1.35		1.60	1.15	1.35	0.80	2.36
	Cement-treated permeable base (without underdrains)	1.06	1.30			1.20	1.10		1.76
Thickness/ Slab Size	200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing	0.85	0.28		0.25	0.95	0.75	0.75	0.25
	300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing	1.15	3.00		3.00	1.00	1.50	1.20	3.30
	250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing	0.98				0.80		0.75	1.00
	240-mm (9.5-in) CRCP with epoxy-coated deformed bars	1.05				1.00		1.10	
	240-mm (9.5-in) CRCP with non-coated deformed bars	1.03				0.80		1.05	
Cross Section	Trapezoidal cross section, 275 to 200 mm (11 to 8 in)	0.95				1.05			
	Thickened edge cross section, 275 mm (11 in) at edges	0.95				1.05			
Joints/Load Transfer	Non-coated dowels	0.96			1.05	0.96	0.95		1.00
	Skewed joints	1.00			0.80	0.95			1.00
	No dowels	0.87			0.60	0.75	0.80		0.43
	Reduced number of dowels	0.90				0.98	0.90	0.75	0.74
Joint Sealing	Hot-poured sealant without widening cut	1.00			0.80	1.00	0.95	1.00	
	Silicone sealant with widening cut	0.97			1.30	1.10	1.05	1.00	
	Silicone sealant without widening cut	0.99			0.90	1.05	0.95	1.00	
	Preformed compression sealant	1.03			1.00	1.15	1.25	1.00	0.90
	No sealant	0.90			0.70	0.95	0.90	1.00	0.60
Shoulders	400-mm (16-in) gravel shoulder	0.90				0.95			1.00
	150-mm (6-in) tied PCC shoulder	1.08			1.20	1.05		1.05	1.25
	250-mm (10-in) tied PCC shoulder	1.10	1.15		1.30	1.10	1.10	1.05	1.58
	0.6-m (2-ft) widened PCC slab	1.10			1.35	1.10	1.10	1.10	1.58
Strength/ Materials	5.2-Mpa (750 lb/yd^2) flexural strength	1.00			1.00	0.90	1.05		1.63
	High-early strength	1.00	1.00		0.90	0.90	1.05		1.00
	Well-graded mix	1.00	1.20		n/a	1.05	1.05		2.00
Initial Smoothness	79 to 110 mm/km (5 to 7 in/mi)		1.00		1.10				1.01
	47 to 79 mm/km (3 to 5 in/mi)		1.02		1.20				1.02
	16 to 47 mm/km (1 to 3 in/mi)		1.03		1.30				1.03
	< 16 mm/km (1 in/mi)		1.03						1.05

Table 18. Raw data and summary of agency (performance) surveys (continued).

Feature Category	Description	Performance Survey Responses 9	10	11	12	Mean	Standard Deviation	Median	High	Low
Subgrade	Lime treated subgrade				0.50	0.86	0.25	0.95	1.09	0.50
Base	No base				0.30	0.76	0.21	0.83	0.93	0.30
	Asphalt-treated base (ATB)		1.20		1.30	1.11	0.17	1.10	1.30	0.80
	Cement-treated base (CTB)		0.75		0.40	0.88	0.26	0.90	1.27	0.40
Drainage	Open-graded, non-stabilized base (with underdrains)	1.35			1.20	1.32	0.24	1.25	1.87	1.08
	Asphalt-treated permeable base (with underdrains)			1.80	1.20	1.52	0.43	1.40	2.36	1.09
	Asphalt-treated permeable base (without underdrains)		1.20			1.27	0.26	1.20	1.76	1.05
	Cement-treated permeable base (with underdrains)				0.50	1.28	0.56	1.25	2.36	0.50
	Cement-treated permeable base (without underdrains)					1.28	0.28	1.20	1.76	1.06
Thickness/ Slab Size	200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing			0.25	0.80	0.57	0.30	0.75	0.95	0.25
	300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing			3.30	1.00	2.05	1.06	1.50	3.30	1.00
	250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing				1.00	0.91	0.12	0.98	1.00	0.75
	240-mm (9.5-in) CRCP with epoxy-coated deformed bars					1.05	0.05	1.05	1.10	1.00
	240-mm (9.5-in) CRCP with non-coated deformed bars	2.03			0.70	1.12	0.53	1.03	2.03	0.70
Cross Section	Trapezoidal cross section, 275 to 200 mm (11 to 8 in)					1.00	0.07	1.00	1.05	0.95
	Thickened edge cross section, 275 mm (11 in) at edges					1.00	0.07	1.00	1.05	0.95
Joints/Load Transfer	Non-coated dowels	1.00		1.00	0.80	0.97	0.07	0.98	1.05	0.80
	Skewed joints		1.00	1.00	1.00	0.96	0.07	1.00	1.00	0.80
	No dowels	0.43	0.40	0.55	0.50	0.59	0.18	0.55	0.87	0.40
	Reduced number of dowels				0.80	0.85	0.10	0.85	0.98	0.74
Joint Sealing	Hot-poured sealant without widening cut		1.20		1.00	0.99	0.12	1.00	1.20	0.80
	Silicone sealant with widening cut	1.00		1.05	1.00	1.06	0.11	1.03	1.30	0.97
	Silicone sealant without widening cut				1.00	0.98	0.05	1.00	1.05	0.90
	Preformed compression sealant				1.00	1.05	0.12	1.00	1.25	0.90
	No sealant				1.00	0.86	0.15	0.90	1.00	0.60
Shoulders	400-mm (16-in) gravel shoulder	1.00		1.00	1.00	0.98	0.04	1.00	1.00	0.90
	150-mm (6-in) tied PCC shoulder				1.20	1.14	0.09	1.14	1.25	1.05
	250-mm (10-in) tied PCC shoulder	1.57	1.40	1.75	1.20	1.30	0.24	1.20	1.75	1.05
	0.6-m (2-ft) widened PCC slab	1.57	1.30	1.75	1.40	1.34	0.24	1.33	1.75	1.10
Strength/ Materials	5.2-Mpa (750 lb/yd^2) flexural strength	1.63			1.00	1.17	0.32	1.00	1.63	0.90
	High-early strength	1.00			0.90	0.97	0.06	1.00	1.05	0.90
	Well-graded mix					1.26	0.42	1.05	2.00	1.00
Initial Smoothness	79 to 110 mm/km (5 to 7 in/mi)		1.00	1.05	1.00	1.03	0.04	1.01	1.10	1.00
	47 to 79 mm/km (3 to 5 in/mi)		1.05	1.20	1.00	1.08	0.09	1.04	1.20	1.00
	16 to 47 mm/km (1 to 3 in/mi)		1.10	1.25	1.00	1.12	0.13	1.07	1.30	1.00
	< 16 mm/km (1 in/mi)		1.15	1.25	1.00	1.10	0.10	1.05	1.25	1.00

Table 19. Raw data and summary of contractor (cost) surveys.

Feature Category	Description	Cost Survey Responses										
		1	2	3	4	5	6	7	8	9	10	11
Subgrade	Lime treated subgrade	0.95	0.92	0.95	1.05	0.92	1.08	0.95	0.95	0.95	1.12	1.02
Base	No base	0.70	0.78	0.88	0.85	0.87	0.96	0.83	0.70	0.94	0.80	0.88
	Asphalt-treated base (ATB)	1.05	1.15	1.25	1.10	1.10	1.17	1.28	1.10	1.07	1.27	1.24
	Cement-treated base (CTB)	0.87	1.11	0.94	1.00	1.06	1.08	1.25	1.00	1.05	1.19	1.21
Drainage	Open-graded, non-stabilized base (with underdrains)	1.31	1.19	1.18	1.10	1.14	1.19	1.34	1.22	1.16	1.23	1.37
	Asphalt-treated permeable base (with underdrains)	1.38	1.20	1.27	1.20	1.23	1.47	1.44	1.28	1.22	1.35	1.56
	Asphalt-treated permeable base (without underdrains)	1.32	1.21	1.20	1.15	1.24	1.38	1.34	1.20	1.15	1.36	1.46
	Cement-treated permeable base (with underdrains)	1.29	1.23	1.21	1.20	1.15	1.28	1.39	1.30	1.19	1.40	1.46
	Cement-treated permeable base (without underdrains)	1.33	1.27	1.13	1.15	1.22	1.27	1.44	1.07	1.18	1.43	1.58
Thickness/ Slab Size	200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing	0.94	0.92	0.94	0.80	0.88	0.90	0.92	0.95	0.97	0.92	0.86
	300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing	1.06	1.08	1.06	1.20	1.10	1.11	1.09	1.08	1.04	1.09	1.17
	250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing	1.10	1.11	1.03	1.25	1.01	1.03	1.02	1.17	1.02	1.18	1.14
	240-mm (9.5-in) CRCP with epoxy-coated deformed bars	1.15	1.13	1.32	1.30	1.11	1.13	1.17	1.20	1.28	1.29	1.30
	240-mm (9.5-in) CRCP with non-coated deformed bars	1.11	1.09	1.13	1.25	1.05	1.10	1.13	1.17	1.24	1.25	1.23
Cross Section	Trapezoidal cross section, 275 to 200 mm (11 to 8 in)	0.98	0.98	1.00	0.95	0.98	0.97	1.00	1.10	0.99	0.95	1.00
	Thickened edge cross section, 275 mm (11 in) at edges	0.98	0.98	1.00	1.00	0.98	0.97	1.00	1.15	0.99	1.00	1.00
Joints/Load Transfer	Non-coated dowels	0.99	1.00	0.99	0.98	1.00	1.00	0.99	0.97	0.99	0.99	0.99
	Skewed joints	1.01	1.03	1.00	1.10	1.00	1.00	1.01	1.05	1.01	1.00	1.01
	No dowels	0.94	0.97	0.94	0.90	0.94	0.94	0.94	0.95	0.91	0.94	0.92
	Reduced number of dowels	0.98	0.99	0.97	0.95	0.97	0.98	0.97	0.99	0.96	0.97	0.97
Joint Sealing	Hot-poured sealant without widening cut	1.00	1.00	0.99	0.98	1.00	1.00	0.99	1.10	1.00	0.96	1.00
	Silicone sealant with widening cut	1.01	1.00	1.01	1.05	1.00	1.01	1.01	1.00	1.00	1.05	1.01
	Silicone sealant without widening cut	1.03	1.00	n/a	1.02	1.00	1.00	1.01	0.99	1.00	1.01	n/a
	Preformed compression sealant	1.02	1.05	1.01	1.15	1.01	1.07	1.02	1.05	1.01	1.08	1.02
	No sealant	0.99	0.99	0.98	0.90	1.00	0.99	0.98	0.90	1.00	0.93	0.99
Shoulders	400-mm (16-in) gravel shoulder	0.84	0.97	0.91	0.95	0.95	0.86	0.90	0.90	0.87	0.91	0.92
	150-mm (6-in) tied PCC shoulder	1.04	1.08	1.08	1.05	1.03	1.07	1.04	1.30	1.14	1.08	1.37
	250-mm (10-in) tied PCC shoulder	1.13	1.14	1.10	1.20	1.06	1.15	1.06	1.35	1.22	1.17	1.28
	0.6-m (2-ft) widened PCC slab	1.04	1.03	1.06	1.10	1.02	1.02	1.01	1.10	1.10	1.05	1.22
Strength/ Materials	5.2-Mpa (750 lb/yd^2) flexural strength	1.03	1.03	1.02	1.10	1.03	1.02	1.01	1.10	1.02	1.06	1.11
	High-early strength	1.05	1.09	1.07	1.40		1.05	1.10	1.23	1.03	1.10	1.28
	Well-graded mix	1.00	1.02	1.00	1.05	1.03	1.01	1.01	1.00	1.00	1.01	n/a
Initial Smoothness	*Aggregate Base*											
	79 to 110 mm/km (5 to 7 in/mi)		1.00	n/a	1.02		1.00	1.00	1.00	1.00		
	47 to 79 mm/km (3 to 5 in/mi)		1.01	n/a	1.02		1.02	1.00	1.00	1.00		
	16 to 47 mm/km (1 to 3 in/mi)		1.02	n/a	1.02		1.02	1.00	1.00	1.04		
	< 16 mm/km (1 in/mi)		1.04	n/a	1.10		1.04	1.01	1.00	1.07		
	Asphalt-treated base (ATB)											
	79 to 110 mm/km (5 to 7 in/mi)		1.00	n/a	1.07		1.00	1.00	n/a	1.00		
	47 to 79 mm/km (3 to 5 in/mi)		1.01	n/a	1.07		1.01	1.00	n/a	1.00		
	16 to 47 mm/km (1 to 3 in/mi)		1.02	n/a	1.07		1.02	1.00	n/a	1.04		
	< 16 mm/km (1 in/mi)		1.03	n/a	1.07		1.03	1.01	n/a	1.07		
	Cement-treated base (CTB)											
	79 to 110 mm/km (5 to 7 in/mi)		1.00	n/a	1.07		1.00	1.00	n/a	1.00		
	47 to 79 mm/km (3 to 5 in/mi)		1.01	n/a	1.07		1.01	1.00	n/a	1.00		
	16 to 47 mm/km (1 to 3 in/mi)		1.02	n/a	1.07		1.02	1.00	n/a	1.04		
	< 16 mm/km (1 in/mi)		1.03	n/a	1.07		1.03	1.01	n/a	1.07		
	Open-graded stabilized drainage layers											
	79 to 110 mm/km (5 to 7 in/mi)		1.00	n/a	1.12		1.00	1.01	1.01	1.00		
	47 to 79 mm/km (3 to 5 in/mi)		1.01	n/a	1.12		1.01	1.01	1.01	1.00		
	16 to 47 mm/km (1 to 3 in/mi)		1.02	n/a	1.12		1.02	1.01	1.01	1.04		
	< 16 mm/km (1 in/mi)		1.03	n/a	1.12		1.03	1.02	1.01	1.07		

Table 19. Raw data and summary of contractor (cost) surveys (continued).

Feature Category	Description	Cost Survey Responses					Mean	Standard Deviation	Median	High	Low
		12	13	14	15	16					
Subgrade	Lime treated subgrade	1.00	0.94	0.94	1.00	0.99	0.98	0.06	0.95	1.12	0.92
Base	No base	0.92	0.91	0.89	0.90	0.84	0.85	0.08	0.88	0.96	0.70
	Asphalt-treated base (ATB)	1.17	1.25	1.18	1.14	1.09	1.16	0.08	1.16	1.28	1.05
	Cement-treated base (CTB)	1.00	1.04	0.97	1.11	1.11	1.06	0.10	1.06	1.25	0.87
Drainage	Open-graded, non-stabilized base (with underdrains)	1.30	1.15	1.10	1.24	1.28	1.22	0.08	1.21	1.37	1.10
	Asphalt-treated permeable base (with underdrains)	1.40	1.24	1.26	1.43	1.34	1.33	0.11	1.31	1.56	1.20
	Asphalt-treated permeable base (without underdrains)	1.40	1.21	1.19	1.48	1.26	1.28	0.11	1.25	1.48	1.15
	Cement-treated permeable base (with underdrains)	1.40	1.13	1.17	1.34	1.30	1.28	0.10	1.29	1.46	1.13
	Cement-treated permeable base (without underdrains)	1.42	1.14	1.15	1.22	1.28	1.27	0.14	1.24	1.58	1.07
Thickness/ Slab Size	200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing	0.95	0.93	0.09	0.96	0.94	0.87	0.21	0.93	0.97	0.09
	300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing	1.10	1.08	1.08	1.06	1.07	1.09	0.04	1.08	1.20	1.04
	250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing	1.10	1.06	1.02	1.05	1.02	1.08	0.07	1.06	1.25	1.01
	240-mm (9.5-in) CRCP with epoxy-coated deformed bars	1.25	1.16	1.24	1.02	1.11	1.20	0.09	1.19	1.32	1.02
	240-mm (9.5-in) CRCP with non-coated deformed bars	1.20	1.11	1.20	1.00	1.07	1.15	0.08	1.13	1.25	1.00
Cross Section	Trapezoidal cross section, 275 to 200 mm (11 to 8 in)	1.00	0.98	1.00	1.00	1.03	0.99	0.03	0.99	1.10	0.95
	Thickened edge cross section, 275 mm (11 in) at edges	1.00	0.98	1.00	1.00	1.04	1.00	0.04	1.00	1.15	0.97
Joints/Load Transfer	Non-coated dowels	1.00	1.00	0.99	0.98	0.98	0.99	0.01	0.99	1.00	0.97
	Skewed joints	1.00	1.00	1.00	1.01	1.00	1.01	0.03	1.01	1.10	1.00
	No dowels	0.96	0.94	0.94	0.94	0.91	0.94	0.02	0.94	0.97	0.90
	Reduced number of dowels	0.98	0.99	0.97	0.96	0.96	0.97	0.01	0.97	0.99	0.95
Joint Sealing	Hot-poured sealant without widening cut	1.00	1.00	0.99	0.79	0.99	0.99	0.06	1.00	1.10	0.79
	Silicone sealant with widening cut	1.01	1.01	1.01	1.32	1.01	1.03	0.08	1.01	1.32	1.00
	Silicone sealant without widening cut	1.01	1.00	1.00	1.17	1.00	1.02	0.05	1.00	1.17	0.99
	Preformed compression sealant	1.02	1.01	1.02	1.58	1.02	1.07	0.14	1.02	1.58	1.01
	No sealant	1.00	0.99	0.99	0.57	0.98	0.95	0.11	0.99	1.00	0.57
Shoulders	400-mm (16-in) gravel shoulder	0.90	0.85	0.94	0.90	0.90	0.90	0.04	0.90	0.97	0.84
	150-mm (6-in) tied PCC shoulder	1.30	1.06	1.07	1.19	1.11	1.13	0.11	1.08	1.37	1.03
	250-mm (10-in) tied PCC shoulder	1.24	1.08	1.12	1.24	1.16	1.17	0.08	1.15	1.35	1.06
	0.6-m (2-ft) widened PCC slab	1.20	1.03	1.05	1.03	1.06	1.07	0.06	1.05	1.22	1.01
Strength/ Materials	5.2-Mpa (750 lb/yd^2) flexural strength	1.10	1.04	1.02	1.03	1.03	1.05	0.03	1.03	1.11	1.01
	High-early strength	1.25	1.07	1.03	1.03	1.06	1.12	0.11	1.07	1.40	1.03
	Well-graded mix	n/a	1.01	n/a	1.00	1.01	1.01	0.01	1.01	1.05	1.00
Initial Smoothness	*Aggregate Base*										
	79 to 110 mm/km (5 to 7 in/mi)		1.00	1.00	1.00	1.00	1.00	0.01	1.00	1.02	1.00
	47 to 79 mm/km (3 to 5 in/mi)		1.00	1.00	1.00	1.00	1.00	0.01	1.00	1.02	1.00
	16 to 47 mm/km (1 to 3 in/mi)		1.01	1.05	1.00	1.02	1.02	0.02	1.02	1.05	1.00
	< 16 mm/km (1 in/mi)		1.01	1.10	1.00	1.05	1.04	0.04	1.04	1.10	1.00
	Asphalt-treated base (ATB)										
	79 to 110 mm/km (5 to 7 in/mi)		1.25	1.00	1.00	1.01	1.04	0.08	1.00	1.25	1.00
	47 to 79 mm/km (3 to 5 in/mi)		1.26	1.00	1.00	1.01	1.04	0.09	1.01	1.26	1.00
	16 to 47 mm/km (1 to 3 in/mi)		1.26	1.05	1.00	1.03	1.05	0.08	1.03	1.26	1.00
	< 16 mm/km (1 in/mi)		1.27	1.10	1.00	1.06	1.07	0.08	1.06	1.27	1.00
	Cement-treated base (CTB)										
	79 to 110 mm/km (5 to 7 in/mi)		1.04	1.00	1.00	1.00	1.01	0.03	1.00	1.07	1.00
	47 to 79 mm/km (3 to 5 in/mi)		1.04	1.00	1.00	1.00	1.01	0.02	1.00	1.07	1.00
	16 to 47 mm/km (1 to 3 in/mi)		1.04	1.05	1.00	1.02	1.03	0.02	1.02	1.07	1.00
	< 16 mm/km (1 in/mi)		1.04	1.10	1.00	1.05	1.04	0.03	1.04	1.10	1.00
	Open-graded stabilized drainage layers										
	79 to 110 mm/km (5 to 7 in/mi)		1.15	1.00	1.00	1.01	1.03	0.06	1.01	1.15	1.00
	47 to 79 mm/km (3 to 5 in/mi)		1.16	1.00	1.00	1.01	1.03	0.06	1.01	1.16	1.00
	16 to 47 mm/km (1 to 3 in/mi)		1.16	1.05	1.00	1.03	1.05	0.05	1.03	1.16	1.00
	< 16 mm/km (1 in/mi)		1.17	1.10	1.00	1.06	1.06	0.05	1.05	1.17	1.00

APPENDIX D. SOFTWARE USER'S GUIDE

SECTION 1. INTRODUCTION

Pavement engineers have many design feature options when designing a PCC pavement. Feature choices such as the selection of base type, drainage type, load-transfer mechanism, slab thickness, and joint sealant type all influence both the cost and expected performance of the resulting pavement section. As pavement designers always strive to maximize performance while minimizing cost, an understanding of the cost and performance influence of each chosen feature is imperative during the design process.

The estimation of the overall cost associated with the selection of different design features is straightforward, as all design feature costs are cumulative. The estimation of overall performance is, however, more complex, as there are many performance interdependencies among design feature choices. For example, a design engineer may estimate that by itself, using a cement-treated base instead of a dense aggregate base may improve overall performance by an estimated 5 percent. In another separate case, the design engineer may estimate that using a tied PCC shoulder instead of an asphalt shoulder would result in a 15 percent increase in performance. However, in actuality, if both design features were made at the same time, an interdependency between these two design features may result in an overall increase in performance of 18 percent (i.e., they are not necessarily cumulative). This software tool utilizes a simplified methodology that allows pavement engineers and contractors to estimate the performance associated with different design feature interdependencies. The results from this software therefore, can be used to estimate the relative cost effectiveness of different combinations of design features.

The contents of this appendix are intended to:

- Introduce the software and define its general capabilities, intended usage, and limitations.
- Provide detailed descriptions of the required inputs.
- Provide guidance on the selection of specific values for the various inputs.
- Guide the user through the process of setting-up and conducting an analysis session.
- Provide guidance on interpreting the output results.

The remainder of this section includes an introduction to the general capabilities and intended uses of the software, followed by an introduction to the software's user interface structure.

Software Capabilities

This software provides a tool for pavement designers and contractors who are interested in investigating the cost and performance trade-offs associated with the selection of different design features during the PCC pavement design process. With this tool, you may define different pavement sections (i.e., different unique combinations of design features) that can then be compared to determine the relative differences in cost and performance of each section. However, before using the software, it is important to understand that it is absolutely not intended as a "design" tool. Instead, it provides a "reasonableness" check regarding the "justification" or "questioning" of the addition of different design features.

Specifically, the following two types of analysis sessions may be conducted using the software:

1) *Direct Comparison*—A *Direct Comparison* analysis session is used to compare two defined pavement sections in order to assess expected differences in cost and performance. A byproduct of this analysis type is the benefit/cost (B/C) ratio associated with each section. In a comparison of two pavement sections, the section with the largest B/C ratio is the most cost effective section to construct. Another way to interpret these B/C ratios is that the larger the B/C ratio, the more performance is achieved per dollar spent.

2) *Sensitivity Analysis*—A *Sensitivity Analysis* session is provided as a method of defining more complex analysis sessions. Specifically, the following two general types of sensitivity analyses may be defined in the software:

   - Comparison of multiple pavement sections—The first general type of sensitivity analysis that may be defined is the comparison of one pavement section to a number of other defined pavement sections (i.e., a series of sections). This series of direct section comparisons is conducted while holding all other analysis inputs constant.
   - Sensitivity of a chosen pavement section to other process inputs—The second general type of sensitivity analysis allows you to investigate the sensitivity of one defined pavement section to changes in the other inputs of the process. Specifically, these sensitivity analyses are used to compute cost and performance changes associated with subjecting a chosen pavement section to a series of different cost data sets, performance data sets, category ranking sets, or a combination of both cost and performance sets (i.e., a two-dimensional analysis).

Both of these general sensitivity analysis types are discussed in more detail later in this appendix.

It is important to note that the outputs (i.e., estimated cost and performance) of this tool are only as good as the reliability of the numerous inputs collected in the user interface. For example, the importance of choosing a representative cost data set, performance data set, and category ranking set are critical to the validity of the results. Because the default data sets provided in the software were based on collected survey data from all over the United States, it is strongly suggested that the user define cost sets, performance sets, and category ranking factor sets that reflect local experiences and conditions.

The user is also reminded that because this tool is built on simple mathematical concepts, the relative trends resulting from the analysis should be deemed more important than the actual values (i.e., the computed percent changes in cost and performance). Therefore, it is again emphasized that the output results from this tool are solely "estimates" of cost and performance associated with changing design features and, therefore, should be used with caution.

Software Structure

Before getting started using the software, it is important to understand the structure of the software's user interface and the data organization within the software.

The user interface is organized as a series of tabs. Brief descriptions of each of the tabs are provided below:

- *Introduction*—The contents of this tab are intended to provide the novice user with the information needed to effectively use the software. The interactive flow chart on the left of this tab is included as a method of introducing the analysis process and its associated user interface components. To learn more about a particular step of the approach, place the cursor over one of the boxes in the flow chart and read the associated text.
- *Section Definition*— The *Section Definition* tab houses controls that allow for the definition of one or more pavement sections. A pavement section is defined as a unique combination of design features in the following 10 design categories:

  – Subgrade.
  – Base/Subbase.
  – Drainage.
  – Thickness/Slab Size.
  – Cross Section.
  – Joints/Load Transfer.
  – Joint Sealing.
  – Shoulders.
  – Strength/Materials.
  – Initial Smoothness.

  Defined pavement sections may be named and saved in the *Pavement Section Master List*. Those saved pavement sections will then be available to the user when defining an analysis session.

  Also included within the *Section Definition* tab are controls that are used to define the inputs required to conduct a simplistic life-cycle cost analysis (LCCA) as part of a defined analysis session. When an analysis is conducted, these cost inputs combined with the expected changes in performance are used to compute simple expected life-cycle cost (LCC) streams associated with the different pavement sections being analyzed. The LCC approach is described as simplistic in that the cost stream values (annual maintenance, rehabilitation, and salvage value costs) can all be determined using simplified methods. However, because of its simplistic nature, the user of the software tool is warned that the results of the LCC analysis should be viewed with caution. While the cost trends may be realistic, the actual computed dollar values may or may not be accurate. If more accurate LCC analysis results are desired, it is recommended that a more rigorous LCC analysis be conducted using accepted methods.
- *Cost/Performance Data Sets*—All analysis computations in the software require the user to define the cost and performance impacts associated with changing individual design features. This tab provides controls that a user can use to define expected percent cost and performance changes associated with specific design feature changes. Each entered percent change is in reference to a defined default *Standard* pavement section that is defined later. Entered cost and performance data values are stored within the software as part of cost or performance data sets, respectively. Defined data sets of both types may be named and saved in the respective data set master lists. Those data sets stored in the master lists will then be available to the user when defining an analysis session.

- *Category Rankings*—The controls included in this tab are used to define customized tables that rank each of the 10 design feature categories in terms of relative importance. For a given category ranking set, all 10 categories are ranked from 1 to 10 (i.e., integer values), with 10 representing the most significant variable of the set. Because the individual percent changes in performance are not cumulative, these design category rankings are used to determine the total expected change in performance associated with changing more than one design feature for a given pavement section. All category ranking sets saved in the *Category Rankings Master List* will be available for later use when building analysis sessions under the *Analysis Setup* tab.
- *Analysis Setup*—The controls of the *Analysis Setup* tab are used to define and conduct direct comparison or sensitivity analysis sessions. After the user defines an analysis session, the analysis results can be summarized in a customizable output report that may be previewed in an on-screen preview window or printed. The detailed output report contains all details of the analysis session including a list of input values, tables of intermediate cost and performance computations, and a series of tables that summarize cost, performance, and LCC analysis results.

More detailed descriptions of each of these tabs and their purpose, usage, and inputs are contained in the remaining sections of this appendix.

SECTION 2. GETTING STARTED

System Requirements and Recommendations

To effectively use this analysis software, you will need an IBM®-compatible industry-standard personal computer with the following minimum characteristics:

- Intel Pentium Pro,® Pentium,® or 486 PC.
- Microsoft® Windows 98, Windows 95, Windows NT 4.0, or newer operating system.
- Minimum of 16 Mb of RAM.
- Pointing device.
- CD-ROM drive.
- Graphics adapter with at least 800 x 600 resolution.
- An Internet browser.

Software Installation

This is an auto-run CD-ROM (i.e., it should automatically launch when placed in your CD-ROM drive). If your current system is not set with "auto insert notification" enabled, you will have to run the SETUP.EXE in the root drive of the CD-ROM. Starting this .exe will launch the install program and/or the contents CD-ROM. Follow the on-screen instructions to complete the installation process. Upon completion, to start the software, select the “PCC Design Feature Comparison Tool” shortcut in your “Programs” list (under “All Programs” in Windows XP) under the “Start” menu.

What You See When You Start the Analysis Software

As mentioned previously, this software uses a tabbed structure as the basis of its user interface (see figure 2). When you first open the program, a blank database is opened (i.e., no user-defined pavement sections, cost sets, performance sets, etc.) with the *Introduction* tab showing. The interactive flow chart on the left of this tab is included as a method of introducing the analysis process and its associated user interface components. To learn more about a particular step of the approach, place your cursor over one of the boxes in the flow chart. The remainder of this section introduces you to the menu and toolbar items that are visible as part of the software’s interface. Each of the other tabs making up the interface is described in other sections of this appendix.

<u>Menu Bar</u>

The menu bar includes three items: *File*, *Edit*, and *Help*. To display the available commands under a specific menu heading, click on the heading of your choice. You may then click on any of the commands shown in the associated drop-down list.

File Menu

The *File* menu, shown in figure 3, contains eight standard windows commands, each of which is described briefly below.

- *New*—Creates a new blank database.

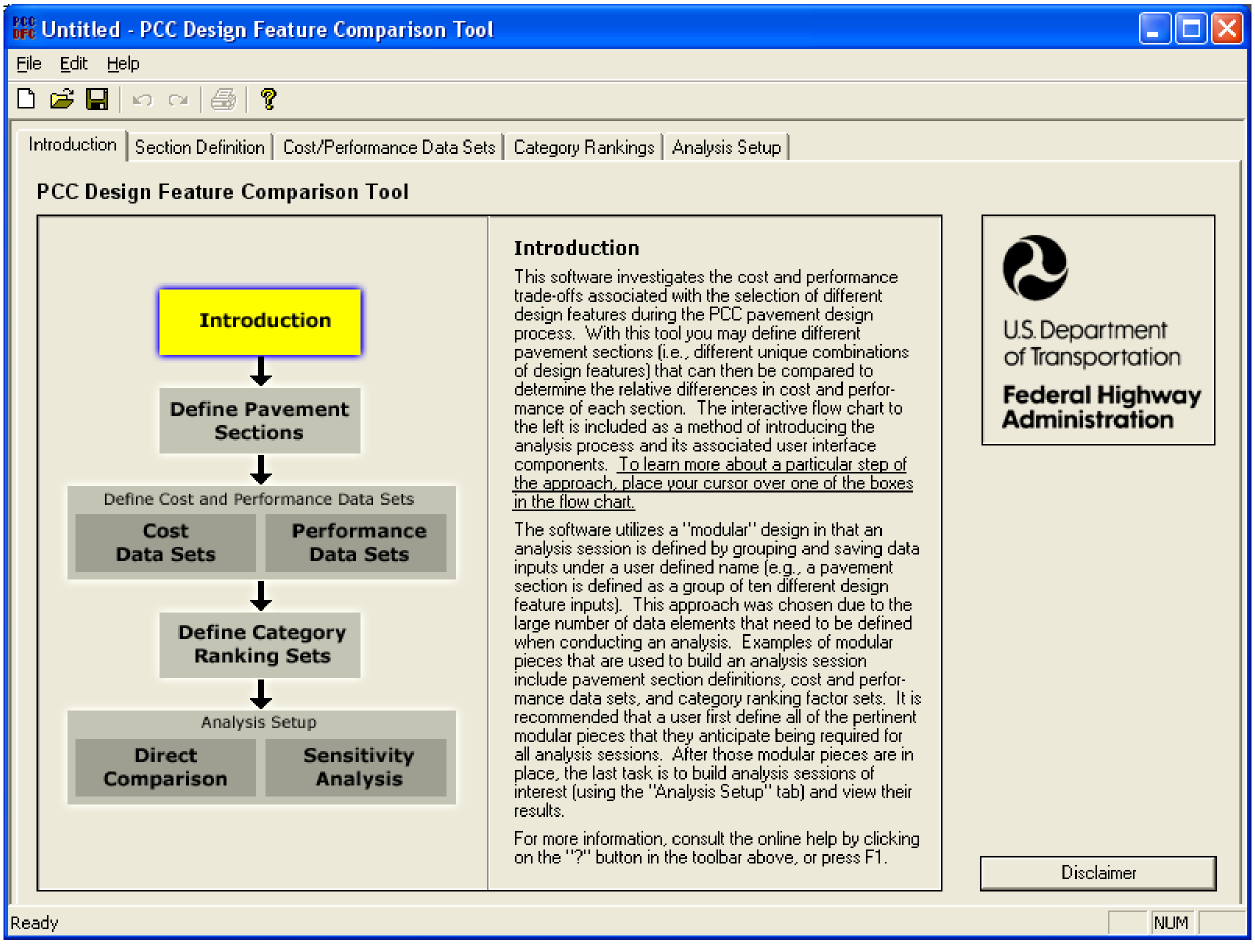


Figure 2. Software main window with the *Introduction* tab displayed.

- *Open*—Opens a previously saved database. Note: databases are saved with a .dfc (design feature comparison) file extension. When you select *Open*, by default, the software looks for all .dfc files in the "Data" directory under the created during installation.
- *Save*—Saves the active database. If the active database has been named and saved previously, the database will be saved under the previously defined file name. If the database has not been named and saved previously, the program will prompt the user to enter a file name and storage location.
- *Save As*—Allows the user to save the active database under a new name or in a new location.
- *Print*—Activates the pop-up *Print* dialog box in preparation for printing information associated with the selected data module (i.e., cost data set, performance data set, or category ranking set) on the current tab. For example, selecting the *Print* menu option while the *Section Definition* tab is activated will prepare a printable report that summarizes information describing the pavement section currently selected in the *Pavement Section Master List*.
- *Print Preview*—Shows an on-screen preview of the prepared printable report associated with the selected data module on the current tab.

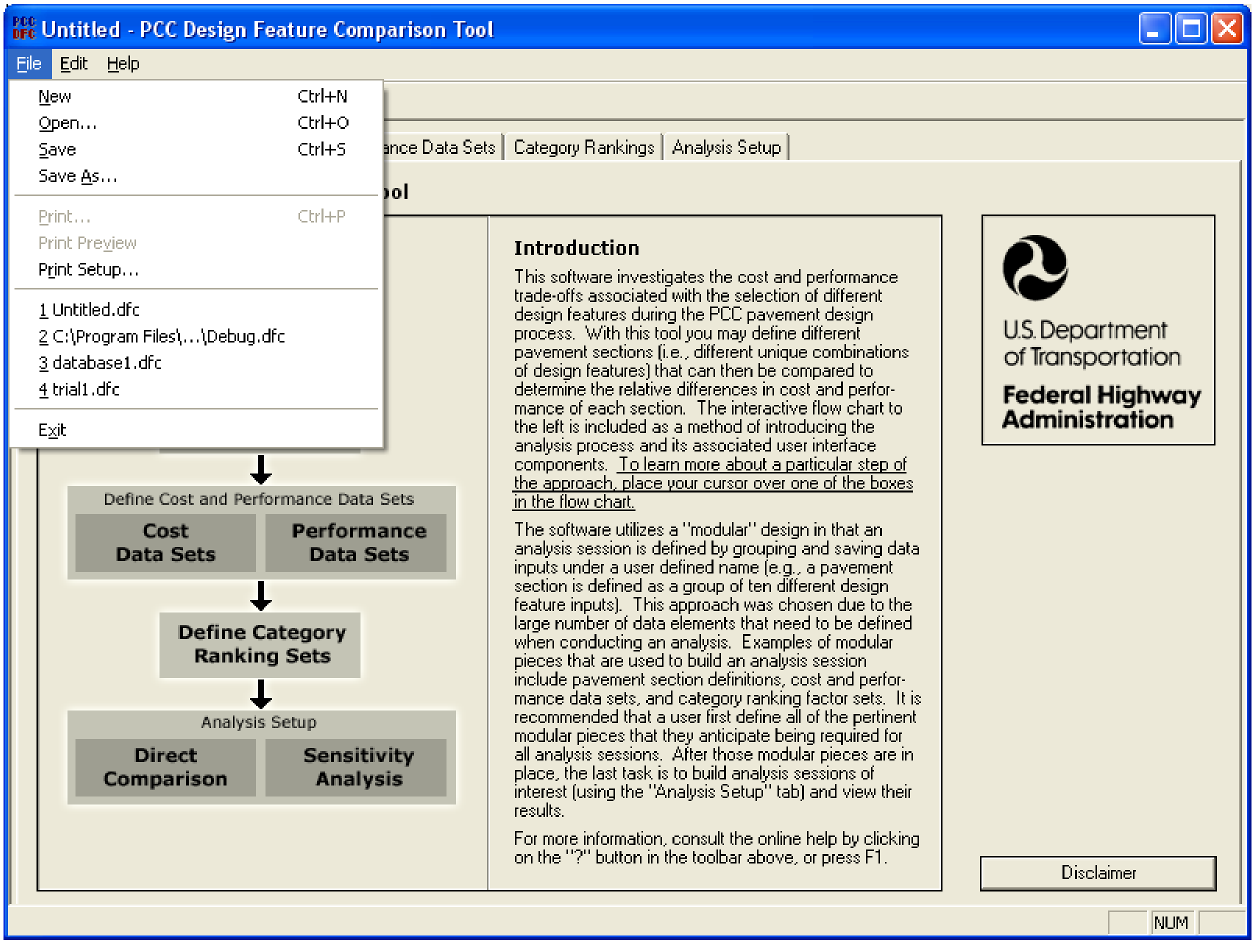


Figure 3. Contents of the *File* menu.

- *Print Setup*—Activates the standard Windows *Print Setup* pop-up dialog box that allows the user to alter printer settings.
- *Exit*—Exits the analysis software program.

In addition to the standard windows commands, a list of the last four viewed databases (if available) will also appear in this file menu. To open a recently viewed database, simply select the name from the list.

Edit Menu

The edit menu contains the following three commands:

- *Undo*—Undoes the last software action completed by the user. Note: the software has an infinite undo, which means that all previous software actions can be undone in the reverse order that they were originally completed.
- *Redo*—Restores any previously undone software actions in the reverse order that they were originally undone.
- *Global LCC Parameters*—Opens a pop-up dialog box that allows the user to change any of the four global cost-related parameters (i.e., analysis period, discount rate, initial cost

of *Standard* pavement section, and whether or not to include a salvage value in the cost computations). These four cost-related inputs are defined as global as they remain constant for all section comparisons that are investigated (i.e., they are not section-specific). Each of these global LCC inputs is described in more detail in this User's Guide in *Section 3. Defining Pavement Sections*.

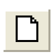
Help Menu

The help menu contains the following three commands:

- *About PCC Design Feature Comparison Tool*—Activates a pop-up dialog box that provides general information about the software. This information includes information such as the current software version, copyright information, pertinent contact information, and a general disclaimer.
- *Help on…*—Activates the software's *Help File* and displays help text that is associated with the user's current location in the software's interface.
- *Display Help Contents*—Opens the software's *Help File* and displays the Help's *Introduction* page.

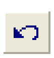
<u>Toolbar Buttons</u>

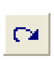
The toolbar buttons provide quick access to many of the commands housed in the software menus. Brief descriptions of each of the visible toolbar buttons are provided below. To activate a toolbar button, simply click on it.

 *New*—Creates a new database.

 *Open*—Opens an existing database.

 *Save*—Saves the current database.

 *Undo*—Undoes the last software action completed by the user.

 *Redo*—Restores any previously undone software actions in the reverse order that they were originally undone.

 *Print*—Activates the pop-up *Print* dialog box in preparation for printing information associated with the selected data module on the current tab.

 *Help*—Opens the *Help File* and displays help text that is associated with the user's current location in the software's interface.

SECTION 3. DEFINING PAVEMENT SECTIONS

As mentioned previously, the software provides the user with a tool for investigating the cost and performance implications of changing different design features in a PCC pavement. Specifically, the software allows the user to change design features organized into the following 10 categories:

- Subgrade.
- Base/Subbase.
- Drainage.
- Thickness/Slab Size.
- Cross Section.
- Joints/Load Transfer.
- Joint Sealing.
- Shoulders.
- Strength/Materials.
- Initial Smoothness.

A summary of all available feature values associated with each of these design categories is summarized in table 20.

A pavement section is defined as a unique combination of specific feature values chosen from these 10 different design feature categories. That is, defining a section requires that the user select one of the provided feature choices (see table 20) for each of the different design categories. The remainder of this section starts by introducing the concept of the *Standard* pavement section. This is followed by a more detailed discussion of the individual controls on the *Section Definition* tab, and how they are used to define and save different pavement sections.

The Default *Standard* Pavement Section

The cost and performance impacts of changing design features are all measured relative to a *Standard* pavement section. Specifically, the *Standard* section is defined as that pavement section with the specific design features summarized in table 21.

The *Standard* pavement section was used as the basis for the survey process as contractors and agencies were asked to estimate percent changes in cost and performance resulting from changing design features from the *Standard* pavement section. Specifically, survey respondents were asked to make subjective estimates of cost and performance changes resulting from changing one design feature at a time in the *Standard* pavement section. It is important to understand the detailed meaning of the *Standard* pavement section as it is referenced many times in the remaining chapters of this appendix.

Defining Pavement Sections with the *Section Definition* Tab

The *Section Definition* tab provides controls that are used to define different pavement sections. As illustrated in figure 4, the left side of the tab contains the *Pavement Section Master List* while the right portion of the tab contains two secondary tabs titled *Variables* and *Life-Cycle Costs*. Each of these areas of the *Section Definition* tab is discussed separately below.

Table 20. Available design features organized by design feature category.

Design Category	Design Feature Choices
Subgrade	• Untreated prepared subgrade (STD) • 300-mm (12-in) lime-treated subgrade
Base/Subbase	• 150-mm (6-in) dense-graded aggregate base on prepared subgrade (STD) • No base (placed directly on prepared subgrade) • 150-mm (6-in) dense-graded asphalt-treated base (ATB) • 150-mm (6-in) dense-graded cement-treated base (CTB)
Drainage	• No drainage layers, no underdrains (STD) • 150-mm (6-in) open-graded, non-stabilized aggregate base (with underdrains) • 150-mm (6-in) asphalt-treated permeable base (with underdrains) • 150-mm (6-in) asphalt-treated permeable base (without underdrains) • 150-mm (6-in) cement-treated permeable base (with underdrains) • 150-mm (6-in) cement-treated permeable base (without underdrains)
Thickness/Slab Size	• 250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing (STD) • 200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing • 300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing (38-mm (1.5-in) epoxy-coated dowels) • 250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing (32-mm (1.25-in) epoxy-coated dowels, 150- by 300-mm (6- by 12-in) mesh) • 240-mm (9.5-in) CRCP (19-mm (0.75-in) epoxy-coated deformed bars, 200 mm (8 in) on center (longitudinal), 914 mm (36 in) on center (transverse)) • 240-mm (9.5-in) CRCP (19-mm (0.75-in) non-coated deformed bars, 200 mm (8 in) on center (longitudinal), 914 mm (36 in) on center (transverse))
Cross Section	• 250-mm (10-in) uniform thickness (STD) • Trapezoidal: 200-mm (8-in) to 275-mm (11-in) thickness (left lane edge to right lane edge) • Thickened edge: 200 mm (8 in) at centerline of 2 lanes, 275 mm (11 in) at outside edges
Joints/Load Transfer	• 250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints (STD) • 250-mm (10-in) JPCP, 32-mm (1.25-in) uncoated dowels, 4.6-m (15-ft) perpendicular joints • 250-mm (10-in) JPCP, 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) skewed joints • 250-mm (10-in) JPCP, no dowels, 4.6-m (15-ft) perpendicular joints • 250-mm (10-in) JPCP, reduced number of 32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints
Joint Sealing	• Hot-poured rubberized asphalt with widening cut (4.6-m (15-ft) joint spacing) (STD) • Hot-poured asphalt without widening cut (4.6-m (15-ft) joint spacing) • Silicone sealant with widening cut (4.6-m (15-ft) joint spacing) • Silicone sealant without widening cut (4.6-m (15-ft) joint spacing) • Preformed compression sealant (4.6-m (15-ft) joint spacing) • No sealant
Shoulders	• 150-mm (6-in) HMA over 250-mm (10-in) dense graded aggregate base (STD) • 400-mm (16-in) gravel • 150-mm (6-in) partial-depth tied PCC over 250-mm (10-in) dense-graded aggregate base • 250-mm (10-in) full-depth tied PCC over 150-mm (6-in) dense-graded aggregate base • 0.6-m (2-ft) widened PCC slab and a 2.4-m (8-ft) HMA shoulder
Strength/Materials	• 4.5-MPa (650 lb/in^2) flexural strength (STD) • 5.2-MPa (750 lb/in^2) flexural strength • High early strength • Well-graded mix
Smoothness/Ride	• 110 to 142 mm/km (7 to 9 in/mi) (STD) • 79 to 110 mm/km (5 to 7 in/mi) • 47 to 79 mm/km (3 to 5 in/mi) • 16 to 47 mm/km (1 to 3 in/mi) • < 16 mm/km (1 in/mi)

Key: STD = Standard.

Table 21. Unique set of pavement design features defining the *Standard* pavement section.

Design Category	Design Features
Subgrade	Untreated prepared subgrade
Base/Subbase	150-mm (6-in) dense-graded aggregate base
Drainage	No drainage layers, no underdrains
Thickness/Slab Size	250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing
Cross Section	250-mm (10-in) uniform thickness
Joints/Load Transfer	32-mm (1.25-in) epoxy-coated dowels, 4.6-m (15-ft) perpendicular joints
Joint Sealing	Hot-poured rubberized asphalt with widening cut (4.6-m joint spacing)
Shoulders	150-mm (6-in) HMA over 250-mm (10-in) dense graded aggregate base
Strength/Materials	4.5-MPa (650 psi) flexural
Initial Smoothness	110 to 142 mm/km (7 to 9 in/mi) (measured with a 5-mm (0.2-in) blanking band)

Note: Average daily traffic (ADT) is 20,000 vehicles per day in each direction with 15% trucks. This is approximately 700,000 to 800,000 ESALs per year in the design lane. Assume no growth in annual ESALs during the life of the pavement.

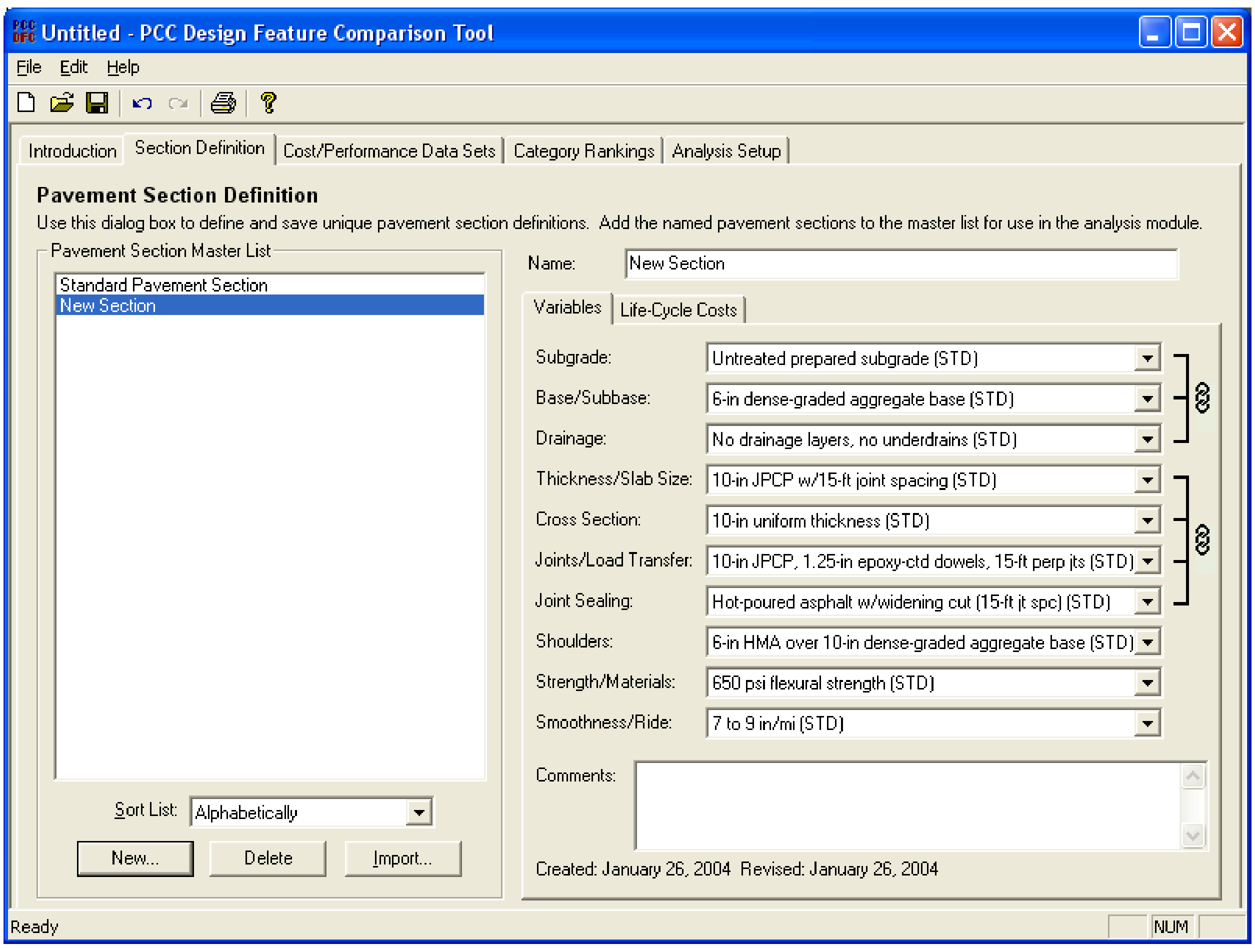


Figure 4. *Section Definition* tab with the *Variables* secondary tab displayed.

Pavement Section Master List Area

The *Pavement Section Master List* stores a complete list of the unique pavement sections defined within the current database. That is, the master list will always include the *Standard Pavement Section* and any user-defined pavement sections. You may sort this list alphabetically, by section creation date, or by section revision date by selecting the appropriate choice from the *Sort List* list box. This area also contains three buttons (*New*, *Delete*, and *Import*) that allow you to manage the contents of the master list. Specifically, the buttons perform the following functions:

Creates and adds a new pavement section to the master list. Upon clicking this button, you will be prompted to enter a name for the new pavement section being created. Upon entering a unique section name, the new section name will be added to the master list. (Note that the new section will have the same design feature properties as the pavement section that was selected in the master list when the *New* button was clicked.)

Deletes the pavement section that is currently selected in the master list. Note that the *Standard Pavement Section* pavement section cannot be deleted from the list.

Imports a previously defined pavement section from another .dfc file. Upon clicking the *Import* button, use the controls of the pop-up Windows Explorer dialog box to locate the .dfc file from which you wish to import a pavement section. After selecting a .dfc file, click the *Open* button to bring up the *Import* dialog box (see figure 5). Select from the list the sections you wish to import and click *OK*. Those selected sections will be added to the master list.

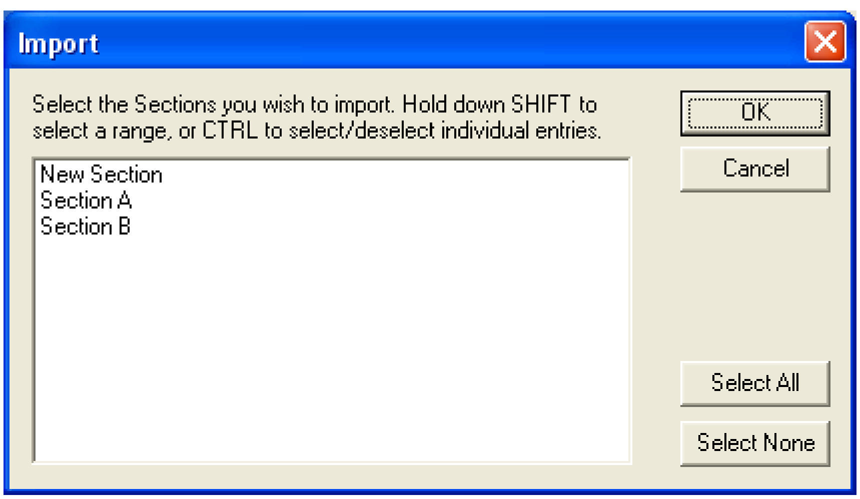


Figure 5. Example of the *Import* pop-up dialog box.

Variables Secondary Tab

The controls of the *Variables* secondary tab (see figure 4) are used to select the specific design features associated with a given pavement section. To edit the feature details of a given section, you must first select the pavement section of interest from the *Pavement Section Master List*. To define the specifics of the selected section, choose a desired design feature from each of the list boxes associated with the 10 different design feature categories. You will notice that the last three items in each design category list box are user-definable custom names. Changes to these custom names may only be made when defining cost or performance data sets under the *Cost/Performance Data Sets* tab.

It must be noted that some design feature categories are inherently dependent on the settings of other categories (i.e., they are linked). Specifically, dependencies exist between the design categories of 1) subgrade, base/subbase, and drainage, and 2) thickness/slab size, cross section, joints/load transfer, and joint sealant. Each of these dependency groups is indicated in the user interface by the brackets and the associated (link) symbol (note: clicking on the symbol in the user interface will open a pop-up dialog with a brief explanation of its meaning). The purpose of the link symbol is to alert the user that changing one of the design categories within a dependent set may automatically modify the settings of one or more of the other linked categories.

An example of a dependency within the first group of design feature categories is observed if you choose *300-mm (12-in) lime treated subgrade* as your subgrade. When this subgrade selection is made, the base/subbase value will automatically be set to *(No base; placed directly on 300-mm (12-inch) stabilized subgrade)* and the drainage value will automatically be set to *(No drainage layers; 300-mm (12-inch) Lime Treated Subgrade)*. These dependencies reflect the limited design feature choices that were allowed in the survey. All of the specific design feature dependencies between subgrade, base/subbase, and drainage are summarized in table 22.

An example of a dependency in the second set of linked fields is observed when the thickness/slab size is set to *250-mm (10-inch) JRCP with 9.1-m (30-ft) joint spacing (32-mm (1.25-inch) epoxy-coated dowels, 150-mm (6-inch) by 300-mm (12-inch) mesh)*. When the JRCP pavement type is chosen, the joint/load transfer field is automatically set to *(Defined under Thickness/Slab Size)*. In addition, the *Joint Sealing* field is set to the first custom joint-sealing value (e.g., *Custom 1*) as all of the default values in the joint sealing list are specific to a 4.6-m (15-ft) joint spacing. Table 23 summarizes all of the specific design feature dependencies between thickness/slab size, cross section, joints/load transfer, and joint sealant.

The bottom of the *Variables* secondary tab provides more general section-related feedback to the user. Use the *Comments* box to enter any general information you wish to save as part of the section definition. Also, note that for your convenience, the section definition creation and revision dates are included to indicate when the section definition was first and last saved in the *Pavement Section Master List*.

Table 22. Summary of *Subgrade*, *Base/Subbase*, *Drainage* dependency.

Design Feature Category	Controlling Design Feature Selection	Explanation of Resulting Dependencies
Subgrade	**IF** *300-mm* (*12-in) lime treated subgrade* **THEN**	• Base/Subbase = (*No base; placed directly on 300-mm (12-in) stabilized subgrade*) • Drainage = (*No drainage layers; 300-mm (12-in) lime treated subgrade*)
	IF subgrade equals anything but *300-mm* (*12-in) lime treated subgrade* **THEN**	No restrictions on base/subbase or drainage
Base/Subbase	**IF** *150-mm (6-in) dense-graded aggregate base (STD)* **THEN**	• Subgrade cannot be *300-mm (12-in) lime stabilized subgrade*. • There are no limitations on the drainage selection.
	IF base type equals anything but *150-mm (6-in) dense-graded aggregate base (STD)* **THEN**	• Subgrade cannot be *300-mm (12-in) lime stabilized subgrade*. • Drainage is automatically set to the standard *No drainage layers, no underdrains*.
Drainage	**IF** *No drainage layers, no underdrains (STD)* **THEN**	No restrictions on subgrade or base/subbase.
	IF any drainage option (including custom drainage) is selected other than *No drainage layers, no underdrains (STD)* **THEN**	• Subgrade can be *Untreated prepared subgrade (STD)* or any of the *Custom* subgrade values. • Base/Subbase must be set to the standard section of *150-mm (6-in) dense-graded aggregate base (STD).*

Key: STD = Standard.

Table 23. Summary of *Thickness/Slab Size*, *Cross Section*, *Joints/Load Transfer*, *Joint Sealing* dependency.

<table>
<tr><th>Design Feature Category</th><th>Controlling Design Feature Selection</th><th>Explanation of Resulting Dependencies</th></tr>
<tr><td rowspan="5">Thickness/Slab Size</td><td>IF 250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing (STD) THEN</td><td>• Joints/load transfer is limited to those 250-mm (10-in) JPCP-related choices and the custom values.
• No limitations on cross section or joint sealing.</td></tr>
<tr><td>IF 200-mm (8-in) JPCP with 3.7-m (12-ft) joint spacing THEN</td><td>Cross section, joints/load transfer, and joint sealing are limited to the Custom values due to a lack of 200-mm- (8-in-) related inputs.</td></tr>
<tr><td>IF 300-mm (12-in) JPCP with 5.5-m (18-ft) joint spacing (38-mm (1.5-in) epoxy-coated dowels) THEN</td><td>• Joints/load transfer becomes inactive and (Defined under ‘Thickness/Slab Size’) is displayed.
• Cross section and joint sealing are limited to the Custom values due to a lack of 300-mm- (12-in-) related inputs.</td></tr>
<tr><td>IF 250-mm (10-in) JRCP with 9.1-m (30-ft) joint spacing (32-mm (1.25-in) epoxy-coated dowels, 150-mm (6-in) by 300-mm (12-in) mesh) THEN</td><td>• Joints/load transfer becomes inactive and (Defined under ‘Thickness/Slab Size’) is displayed.
• Cross section and joint sealing are limited to the Custom values due to a lack of JRCP-related inputs.</td></tr>
<tr><td>IF 240-mm (9.5-in) CRCP (19-mm (0.75-in) epoxy-coated deformed bars, 200-mm (8-in) o.c. (longitudinal), 900-mm (36-in) o.c. (transverse))

OR 240-mm (9.5-in) CRCP (19-mm (0.75-in) noncoated deformed bars, 200-mm (8-in) o.c. (longitudinal), 900-mm (36-in) o.c. (transverse)) THEN</td><td>• Joints/load transfer and joint sealing become inactive and display (Not applicable with CRCP).
• Cross section is limited to the Custom values due to a lack of CRCP-related inputs.</td></tr>
<tr><td rowspan="2">Cross Section</td><td>IF any of the non-custom pavement cross section choices are selected THEN</td><td>• Thickness/slab size is set to the standard 250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing (STD).
• No limitations on joints/load transfer or joint sealing.</td></tr>
<tr><td>IF a Custom pavement cross section choice is selected THEN</td><td>No limitations on thickness/slab size, joints/load transfer, or joint sealing.</td></tr>
<tr><td rowspan="2">Joints/Load Transfer</td><td>IF any of the non-custom joints/load transfer choices are selected THEN</td><td>• Thickness/slab size is limited to the standard 250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing (STD).
• No limitations on cross section or joint sealing</td></tr>
<tr><td>IF a Custom joints/load transfer choice is selected THEN</td><td>No limitations on thickness/slab size, cross section, or joint sealing.</td></tr>
<tr><td rowspan="2">Joint Sealing</td><td>IF any of the non-custom joint sealing options are selected THEN</td><td>• Thickness/slab size is limited to the standard 250-mm (10-in) JPCP with 4.6-m (15-ft) joint spacing (STD).
• No limitations on cross section.
• Joints/load transfer is limited to one of the 250-mm (10-in) JPCP choices or a Custom value.</td></tr>
<tr><td>IF any of the Custom joint sealing options are selected THEN</td><td>No limitations on thickness/slab size, cross section, or joints/load transfer.</td></tr>
</table>

Key: STD = Standard; o.c. = on center.

Life-Cycle Costs Secondary Tab

Because design feature changes alter the expected performance (estimated service life) of a given pavement section, the associated LCC stream is also affected. To compute a LCC stream for a given pavement section, you need to know expected pavement service life as well as detailed expected cost information (i.e., cost types and amounts). Whereas the selected category ranking set is used to determine expected overall performance for a pavement section, the cost details needed for the simplistic LCCA are defined under the *Life-Cycle Costs* secondary tab (shown in figure 6).

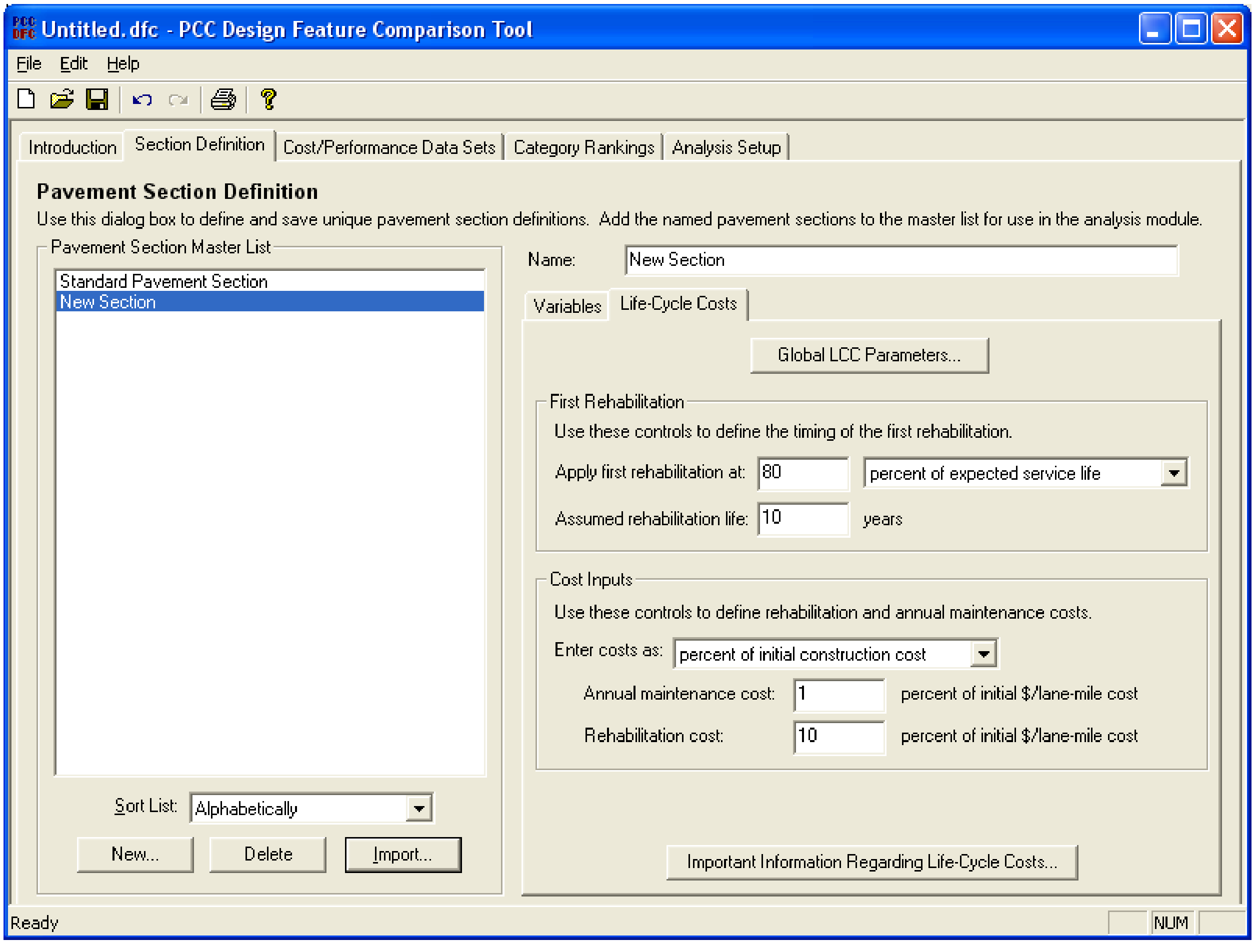


Figure 6. *Section Definition* tab with the *Life-Cycle Costs* secondary tab displayed.

The LCCA conducted within this software is described as simplistic in that the cost stream values (annual maintenance, rehabilitation, and salvage value costs) can all be determined using simplified methods. Because of its simplistic nature, the user of the software tool is warned that the results of the LCCA should be viewed with caution. While the cost trends may be realistic, the actual computed dollar values may or may not be accurate. If more accurate LCCA results are desired, it is recommended that a more rigorous LCCA be conducted using established methods.

Within the software's user interface, cost-related parameters are referred to as either global or pavement section-specific. The details of each of these LCCA parameter types are described separately in the following sections.

Global LCC Parameters

Those cost-related inputs that do not change between defined analysis sessions are referred to as global LCC parameters. Within the user interface, access to these global inputs is obtained by either clicking on the [Global LCC Parameters...] button or by selecting *Global LCC Parameters* from the *Edit* menu. Either action will cause the *Global Life-Cycle Cost Parameters* pop-up dialog box to appear (see figure 7).

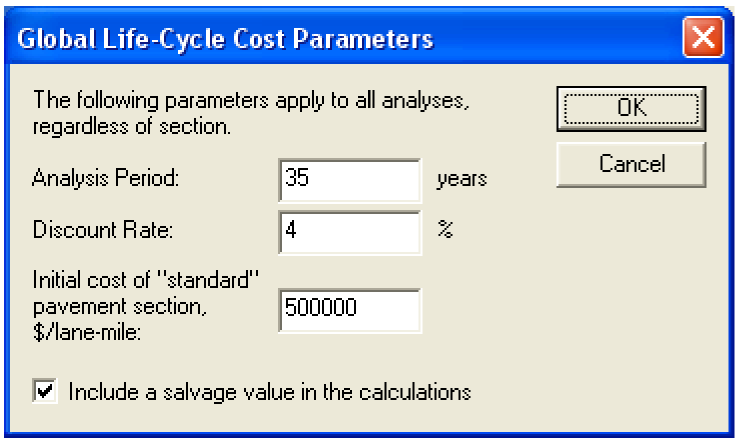


Figure 7. *Global Life-Cycle Cost Parameters* pop-up dialog box.

The remainder of this section contains detailed descriptions of each of the global cost-related inputs included on the *Global Life-Cycle Cost Parameters* pop-up dialog box.

- Analysis Period—The period of time over which future maintenance and rehabilitation costs are to be considered in the LCCA. Based on FHWA recommendations, the analysis period must be a minimum of 35 years.
- Discount Rate—The discount rate, entered as a percentage, is the estimated difference between the interest and inflation rates over a long time period. It is used to translate actual LCCs into equivalent present-worth costs. Historically, the discount rate has been in the range of 3 to 5 percent. A default value of 4 percent is used in the software.
- Initial Construction Cost for the *Standard* Pavement Section—This is a fence-line-to-fence-line pavement-related cost expressed in dollars per lane-mile. Based on the specific design features defined for the *Standard* pavement section, a default value of $311,000 per lane-km ($500,000 per lane-mile) is used as the default value for this input.
- Inclusion of Salvage Value—The user has the choice whether or not to include a salvage value in the LCCA. If it is included, a linear depreciation of the rehabilitation cost is used to determine the salvage value at the end of the analysis period. For example, if a rehabilitation with an expected life of 10 years and a cost of $50,000 is applied 3 years before the end of the analysis period, the salvage value would be computed as $50,000 * (10 – 3)/10 = $35,000. The salvage value is essentially a negative cost applied at the end of the analysis period.

Although it is not a user-definable global input, it is important to mention that the design life for the *Standard* pavement section is hard coded in the software as 20 years. This value is important because other pavement section expected lives are computed by multiplying this 20-year design life by the expected performance ratio computed by using the analysis methodology. For example, if a custom section were found to have an overall modified expected performance of +7.0 percent, the expected design life of the custom section is computed as 1.07 * 20 years = 21.4 years.

Pavement Section-Specific LCC Parameters

All annual maintenance and rehabilitation-related LCCA inputs are section-specific within the analysis approach. That is, all of these LCC inputs may be customized for each unique pavement section that is defined. The primary purpose of linking these cost inputs to a section is to accommodate the many cases where the inclusion of a design feature directly influences the future maintenance and rehabilitation costs associated with that section (e.g., including edge drains will result in the additional cost of cleaning the edge drains).

The remainder of this section contains detailed descriptions of each of section-specific LCC parameter inputs included on the *Life-Cycle Costs* secondary tab.

- Time Until First Rehabilitation—The time until first rehabilitation may either be defined in terms of percent of the expected design life or years after initial construction. Use the associated list box to select your input method of choice. The entered value is used to determine the time at which the first rehabilitation cost is to be applied within the LCCA. For example, if the design life is 20 years, and the time until first rehabilitation is entered as 80 percent, then the rehabilitation cost included in the LCC stream would be applied at a time of 20 years * 0.8 = 16 years.
- Assumed Rehabilitation Life—The assumed life of the chosen rehabilitation activity is defined in terms of years. As subsequent rehabilitation activities are assumed to be applied until the end of the analysis period, this LCC input determines the regular interval at which subsequent rehabilitation activities will be applied. For example, assume the user defines a first year of rehabilitation as year 17, a rehabilitation life of 7 years, and an analysis period of 35 years. These inputs would result in rehabilitation costs being applied at years 17, 24, and 31.
- Method of Entering Annual Maintenance and Rehabilitation Costs—To build LCC streams associated with pavement sections, the software needs to determine costs of annual maintenance and rehabilitation. These costs may be defined either in terms of percent of the initial construction cost or as specific cost values. The selected method defined in this list box is then used to set the units of the associated input fields for both the annual maintenance and rehabilitation costs.
- Annual Maintenance Cost—This input field is used to determine the maintenance cost applied annually within the LCCA. For example, if the initial construction cost is \$186,000 per lane-km (\$300,000 per lane-mile), and the user chooses to compute the annual maintenance cost as 5 percent of the initial construction cost, then the annual maintenance costs included in the LCC stream are computed as 0.05 * \$186,000 = \$9,300/lane-km per year (\$15,000/lane-mile per year). Note that this annual maintenance cost is set equal to zero in years where rehabilitation is applied.

- Rehabilitation Cost—This input field is used to determine the rehabilitation cost that is applied at every scheduled rehabilitation application within the LCCA. As with the annual maintenance cost, this input is either entered as a percentage of the initial construction cost or as a specific cost value.

The primary outputs of this simplified LCCA are the individual computed costs defining the associated LCC stream, their computed total present worth value, and the associated equivalent uniform annual cost (EUAC).

SECTION 4. COST AND PERFORMANCE DATA SETS

The analysis approach allows the user to estimate the cost effectiveness of a given pavement section by computing total expected changes in cost and performance associated with the selection of different design features. These overall expected changes are computed as functions of user-defined cost and performance data sets that summarize the expected cost and performance impacts, respectively, associated with changing individual design features. A data set is defined as a summary of the relative percent changes in cost or performance associated with all possible design features available in each of the 10 design categories. The *Cost/Performance Data Sets* tab (shown in figure 8) is provided as a means of defining both types of data sets. Although each is similar in structure, the details of working with each data set type are discussed separately below.

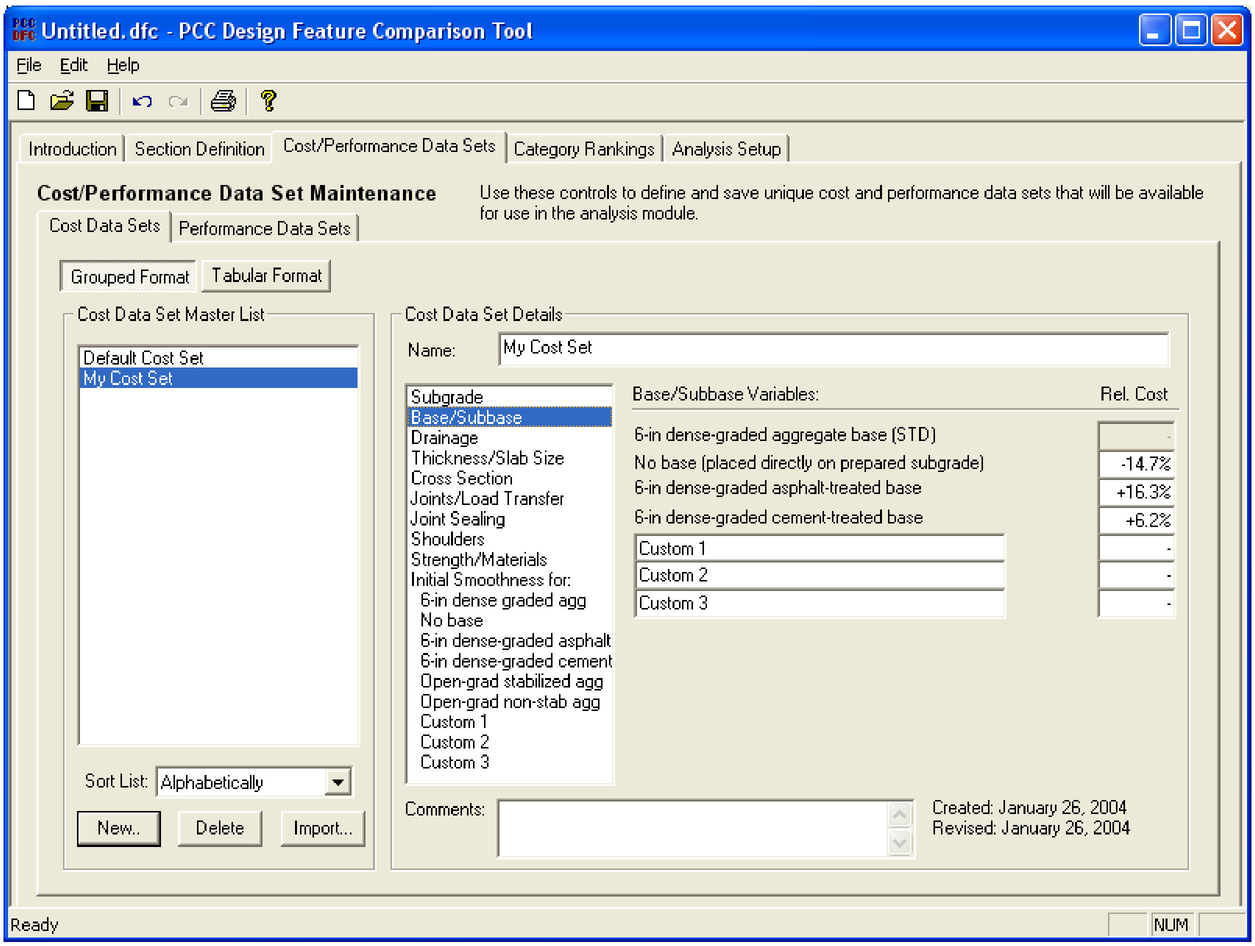


Figure 8. *Cost/Performance Data Sets* tab with the *Grouped Format* controls for the *Cost Data Sets* secondary tab visible.

Grouped Versus Tabular Format

The definition of cost or performance data sets requires a large number of input values to be defined for each. To facilitate the definition of these many data, the software allows the user to enter relative cost or performance data within one of two data-entry interface types termed *Grouped* or *Tabular* formats. Each of these interface types is introduced below.

Grouped Format

The *Grouped* format (the default data-entry format) is divided into a *Master List* area and a *Details* area. The *Cost Data Sets* and *Performance Data Sets* master lists respectively store complete lists of the unique cost and performance data sets defined within the current database. You may sort these lists alphabetically, by section creation date, or by section revision date by selecting the appropriate choice from the associated *Sort List* list box. As with the *Section Definition Master List*, this area also contains *New*, *Delete*, and *Import* buttons that allow you to manage the contents of the master lists.

The *Details* area is used to specify the specific expected relative cost or performance values associated with choosing different design features. To edit the details of a given data set, start by selecting the data set from the corresponding master list. Next, select a design feature category from that provided list (note that the smoothness inputs are specific to different base types). When this selection is made, the specific design feature choices and associated input values for the given design feature category are displayed. Finally, use the provided input values to define the current data set. (Note: guidance on defining specific relative cost or performance values is provided below in the *Defining Cost Data Sets* and *Defining Performance Data Sets* sections, respectively).

Tabular Format

In the *Tabular* format interface (displayed in figure 9), each data set in the master list is presented in a separate column in the table. Use the provided scroll bars to access the specific data set you wish to alter and make appropriate changes within the provided table. This area contains three buttons (*Add Set*, *Delete Set*, and *Export Table*) that perform the following functions:

Add Set... In this *Tabular* view, new data sets can be added to the master list using this button.

Delete Set Existing data sets may be removed from the master list by clicking on a cell in the column associated with the data set of interest, and then clicking this button. Note that the default cost and performance data sets can not be deleted.

Export Table... The current summary table of cost or performance data sets may be exported to an external electronic file by clicking this button. Upon clicking the Export Table... button, use the controls provided in the *Save Table As* dialog box to select a file name, type, and storage location.

Defining Cost Data Sets

Cost data sets are collections of expected changes in total cost resulting from the change of one design feature at a time. The changes in cost are expressed as expected percent changes in relation to the default *Standard* pavement section (see table 21). This is an extremely important point as the impact of all feature changes must be referenced back to the *Standard* pavement section. Therefore, you must complete the following procedure to define a complete cost data set:

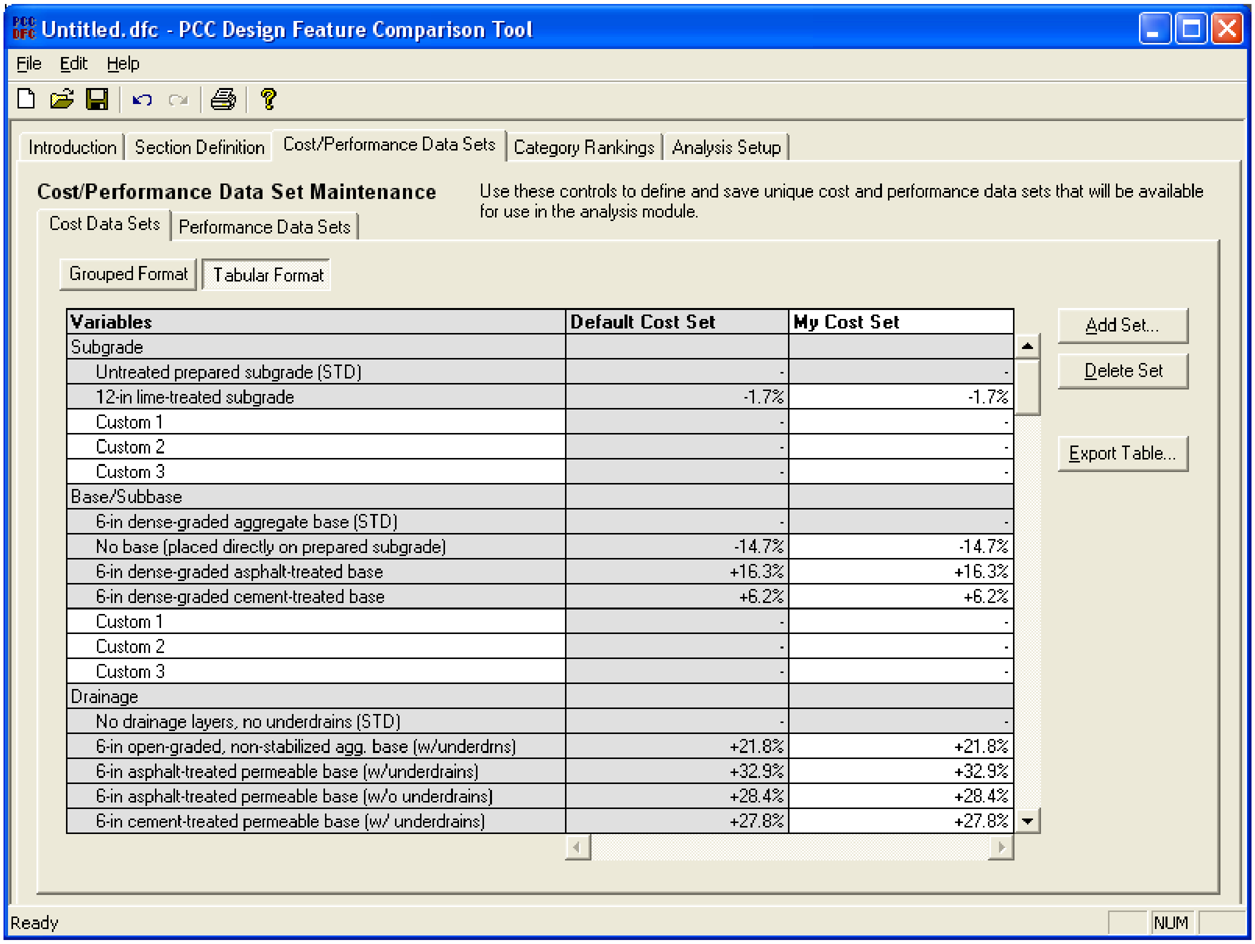


Figure 9. *Cost/Performance Data Sets* tab with the *Tabular Format* controls for the *Cost Data Sets* secondary tab visible.

1. For one design feature category, select one of the other design feature choices (i.e., one that is different from the value defined for the *Standard* pavement section) in that design feature category (see table 20). Note: this should be the only design feature that is different from the defined *Standard* pavement section.
2. Estimate the expected change in overall pavement cost and express this value in terms of a percent difference (increase or decrease) from the expected cost of the *Standard* pavement section. Note: this percent change in cost is then entered into the appropriate cell of the cost data set.
3. Repeat steps 1 and 2 for all available design feature choices (see table 20) for the selected design feature category.
4. Repeat steps 1, 2, and 3 for each of the 10 design categories.

This four-step process systematically defines a complete cost data set that covers all possible individual design feature changes from the standard pavement section. The following two examples illustrate the completion of steps 1, 2, and 3 for base/subbase and initial/smoothness, respectively.

Example 1: Expected Relative Costs Associated with Different Base/Subbase Types

Assume you are currently defining relative costs associated with the available base types in the base/subbase category. Recall that the *Standard* pavement section is defined with a base type of *150-mm* (*6-in) dense-graded aggregate base*. The following paragraphs discuss the thinking a user should follow when trying to estimate cost changes associated with making base type changes.

The first base type that needs a relative cost value defined (i.e., the first base type after that used for the *Standard* section) is the choice of *No base (placed directly on prepared subgrade)*. For this example, you would ask yourself, "If I remove the *150-mm (6-in) dense-graded aggregate base* and build the *Standard* pavement section directly on the prepared subgrade, what percent change in original total construction cost (of the *Standard* section) would I expect to observe?" If you estimate that this base type substitution might save 14.7 percent in the total expected construction cost, then you would enter "–14.7" in the associated relative cost field. Note: This is the case for the example presented in figure 8.

Continuing the methodical investigation of base/subbase type, you would then define the case for the next base type in the list (i.e., *150-mm (6-inch) dense-graded asphalt-treated base (ATB)*). For this case, you would ask yourself, "What percent change in original total construction cost would I expect to observe if I substitute a *150-mm (6-in) dense-graded asphalt-treated base* in place of the *Standard* section's *150-mm (6-in) dense-graded aggregate base*?" If, for example, you estimate that the *Standard* section would cost 16.3 percent more as a result of using an ATB, then you would enter "16.3" in the associated relative cost field (as shown in figure 8).

To complete this procedure, you must define an appropriate relative cost value for every base/subbase type (design feature value) included in the base/subbase list. Note: leaving a relative cost field blank is the same as setting it equal to "0." A relative cost value of "0" is appropriate for those cases where you can safely say that making that specific design feature change will not impact the overall cost of the *Standard* pavement section.

You will notice that three *Custom* fields (initially titled *Custom 1*, *Custom 2*, and *Custom 3*) are included in each design category. These are user-definable fields that are included to provide flexibility in the software by allowing the user to customize the software to match typically used design features. Figure 10 illustrates a case in which a user has defined a *200-mm (8-in) dense-graded aggregate base* in the first custom field. For this case, the user has assumed that constructing the *200-mm (8-in) dense-graded aggregate base* will cost 2.5 percent more than constructing the *150-mm (6-in) dense-graded aggregate base* defined as part of the *Standard* pavement section. Note that the custom field names may only be changed within the *Cost/Performance Data Sets* tab (i.e., you cannot define custom field names under the *Section Definition* tab).

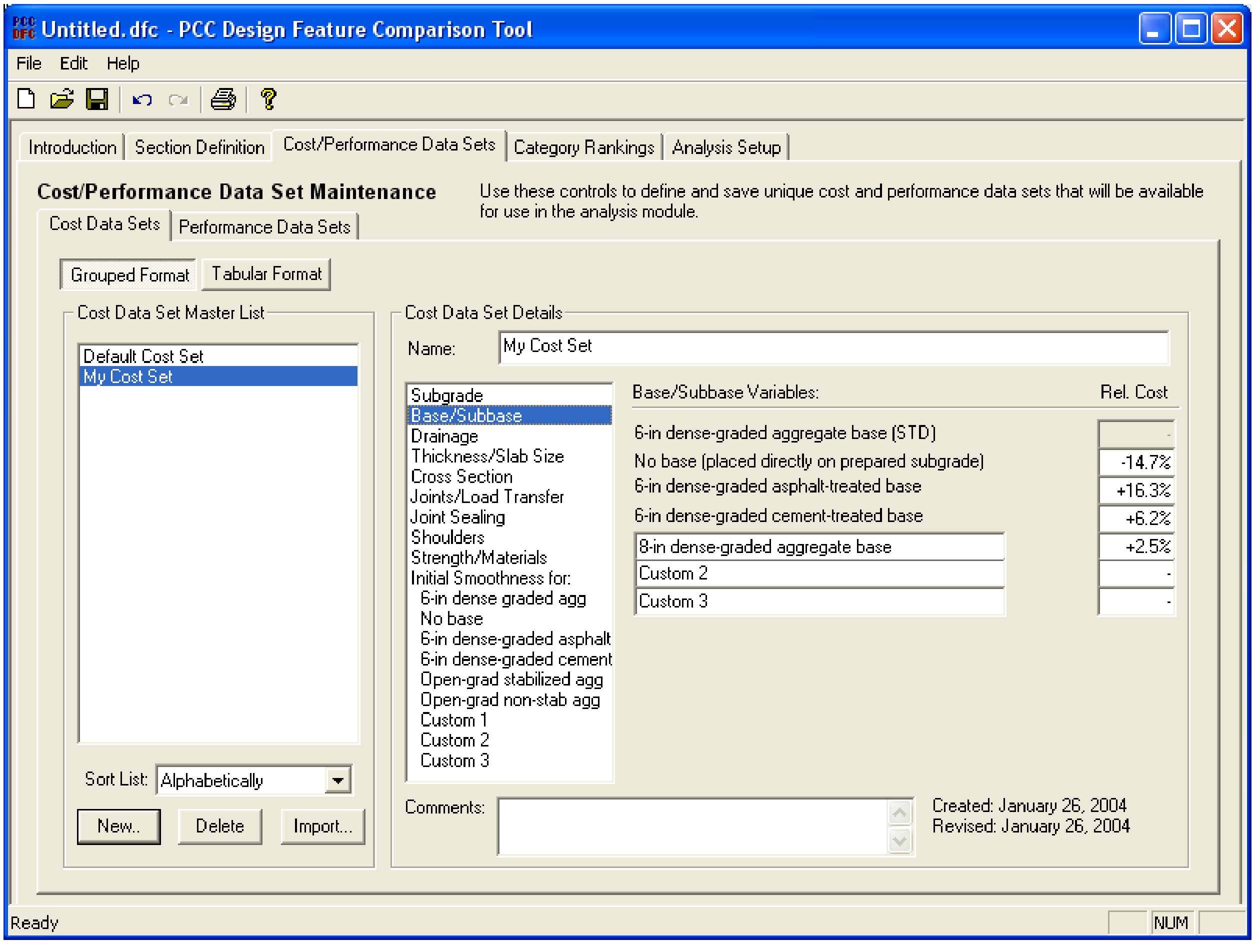


Figure 10. Example of using the provided custom design feature fields to reflect an agency's custom design features.

<u>Example 2: Expected Relative Costs Associated with Different Initial/Smoothness Levels</u>

The definition of relative costs associated with different initial smoothness ranges is different from other design feature categories in that the relative costs are also dependent on different base types. Therefore, the first step in defining smoothness-related relative cost values is to select one of the base types listed under *Initial Smoothness for:* in the design feature category list. Figure 11 illustrates an example in which relative cost values were defined for the scenario in which a *150-mm (6-in) dense-graded asphalt-treated base* was used in place of the *Standard* section's *150-mm* (*6-in) dense-graded aggregate base*.

To accurately interpret the meaning of these smoothness-related relative cost values, you must keep their specific meaning in mind. The +3.3 percent value associated with the 110 to 142 mm/km (7 to 9 inches/mi) smoothness range (associated with a 5-mm (0.2-inch) blanking band) may seem confusing at first as you might expect this value to be 0 percent since this 110 to 142 mm/km (7 to 9 inches/mi) smoothness range is the smoothness range used to define the *Standard* pavement section. However, you must remember that for the different base types, the relative cost values associated with this *Standard* smoothness range (110 to 142 mm/km (7 to 9 inches/mi))

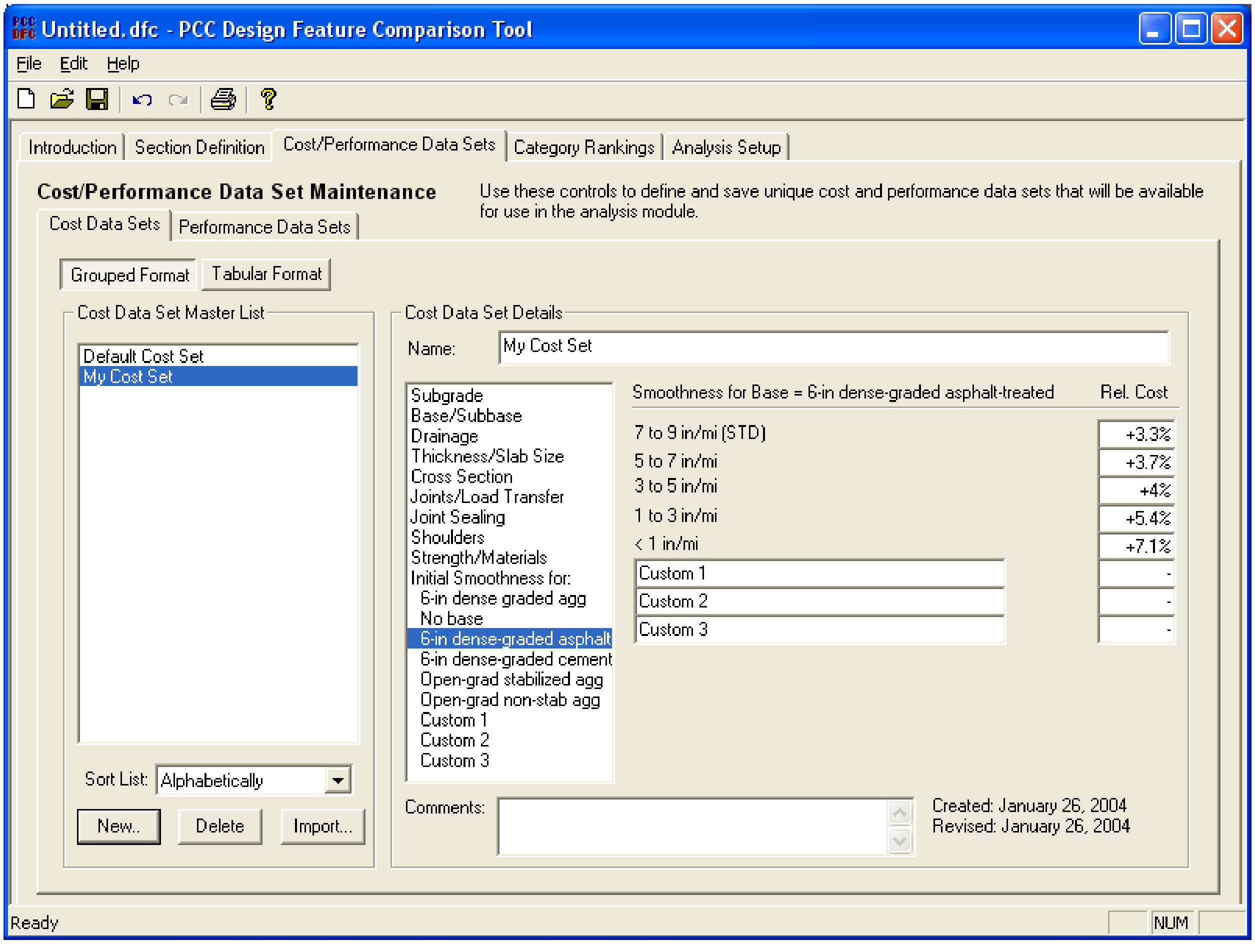


Figure 11. Example showing chosen relative cost values associated with a *150-mm* (*6-inch) dense-graded asphalt-treated base*.

indicates of how much more or less smoothness-related preparation time is associated with the given base type. Specifically, for our example, the user indicated that achieving a target 110 to 142 mm/km (7 to 9 inches/mi) smoothness range is expected to cost 3.3 percent more if constructing on a *150-mm (6-inch) dense-graded asphalt-treated base* than it would if constructing on a *150-mm (6-inch) dense-graded aggregate base*.

For the remainder of the initial smoothness range inputs, the user must remain cognizant of the fact that the entered costs must only reflect the additional or reduced costs associated with achieving the target smoothness value. That is, although smoothness-related costs are entered for a series of different base types, the changes in the cost of the base layer are not to be included in these smoothness-related cost numbers. Costs associated with changing base types are those values entered under the base type category.

In the example displayed in figure 11, the user has decided that achieving 79 to 110 mm/km (5 to 7 inches/mi) on a *150-mm (6-inch) dense-graded asphalt-treated base* would cost 3.7 percent more than achieving 110 to 142 mm/km (7 to 9 inches/mi) on a *150-mm (6-inch) dense-graded aggregate base* (i.e., the *Standard* base type and initial smoothness values). Another way to look at this +3.7 percent number is to only look at the numbers within a given base type. Because we already have an indication of the smoothness-related cost strictly associated with the base type difference (i.e., +3.3 percent), it is the difference between the ATB-related relative cost values that give an indication of the costs associated with additional work required to achieve a smoother pavement for a given base type. In our example, the user has indicated that (assuming an ATB) it will cost +0.4 percent more to increase smoothness from 110 to 142 mm/km (7 to 9 inches/mi) to 79 to 110 mm/km (5 to 7 inches/mi) (i.e., +3.7 percent – 3.3 percent = +0.4 percent); +0.7 percent more to increase smoothness from 110 to 142 mm/km (7 to 9 inches/mi) to 47 to 79 mm/km (3 to 5 inches/mi) (i.e., +4.0 percent – 3.7 percent = +0.7 percent); +2.1 percent more to increase smoothness from 110 to 142 mm/km (7 to 9 inches/mi) to 16 to 47 mm/km (1 to 3 inches/mi) (i.e., +5.4 percent – 3.3 percent = +2.1 percent); and +3.8 percent more to increase smoothness from 110 to 142 mm/km (7 to 9 inches/mi) to < 16 mm/km (< 1 inches/mi) (i.e., +7.1 percent – 3.3 percent = +3.8 percent). To complete the cost data set, values must be defined for all combinations of base type and initial smoothness range.

Defining Performance Data Sets

Performance data sets are presented in exactly the same structure as cost data sets. However, instead of storing a collection of relative cost data, performance data sets are collections of expected changes in total performance resulting from the change of one design feature at a time. As with the cost data sets, the changes in performance are expressed as expected percent changes in relation to the expected performance of the *Standard* pavement section. This is an extremely important point as the impact of all feature changes must be referenced back to the *Standard* pavement section. This concept is best explained by the following two examples. (Note: although only two examples are presented, the complete definition of a performance data set involves defining relative performance values for each design feature value within each of the 10 design feature categories.)

<u>Example 1: Expected Relative Performance Associated with Different Base/Subbase Types</u>

Using the same examples provided in the cost data sets discussion, let's assume you are currently defining relative performance values associated with the available base types in the base/subbase category. Recall that the standard section is defined with a base type of *150-mm (6-in) dense-graded aggregate base*. The following paragraphs discuss the thinking a user should follow when trying to estimate performance changes associated with making base/subbase type changes.

The first base/subbase type that needs a relative performance value defined (i.e., the first base type after that used for the *Standard* section) is the choice of *No base (placed directly on prepared subgrade)* (see table 20). For this example, you would ask yourself, "If I remove the *150-mm (6-in) dense-graded aggregate base* and build the *Standard* pavement section directly on the prepared subgrade, what percent change in pavement performance (pavement life) would I expect to observe?" Again, you are reminded that the baseline performance that should be used for comparison is the expected performance of the default *Standard* section. If you estimate that this base type substitution might result in a decrease in performance of 20.0 percent (i.e., the new section will carry 20.0 percent fewer ESALs than the *Standard* section), then you would enter "–20.0" in the associated relative performance field. Note: This specific example is the case for the example presented in figure 12.

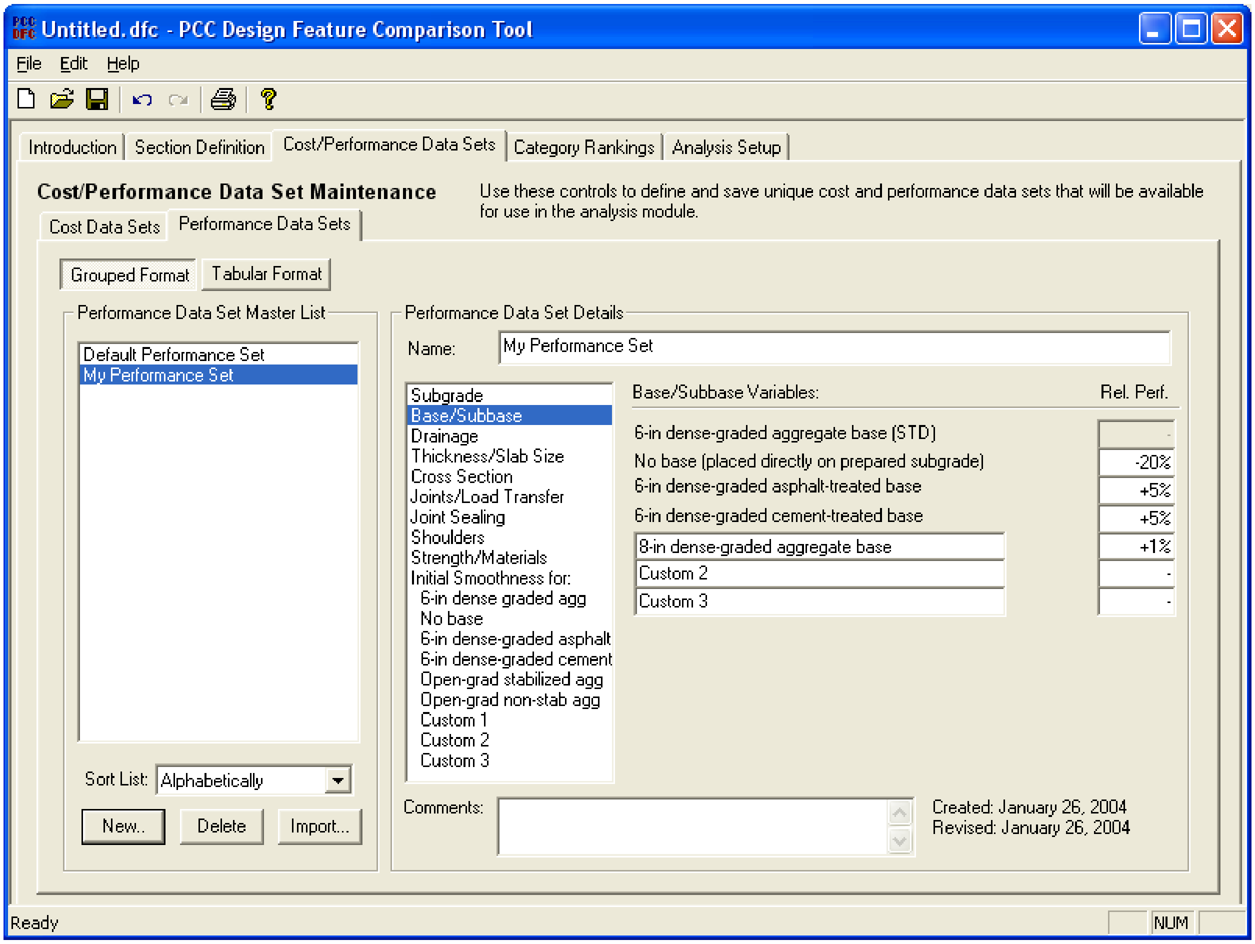


Figure 12. Example showing defined relative performance values associated with different base type choices.

The next base/subbase type in the list to be considered in the investigation is the *150-mm (6-inch) dense-graded asphalt-treated base*. For this case, you would ask yourself, “What percent change in expected performance would I expect if I substituted the *Standard* section’s *150-mm (6-inch) dense-graded aggregate base* with a *150-mm (6-inch) dense-graded asphalt-treated base*?” The user-defined numbers presented in figure 12 indicate that the user expected to increase the *Standard* section’s allowable ESALs by 5.0 percent as a result of substituting base types.

As with the cost data sets, you must define an appropriate relative performance value for every base/subbase type (design feature value) included in the base/subbase list (including the custom fields). For completeness, this definition procedure must be completed for all 10 of the design feature categories. Note: Leaving a relative performance field blank is the same as setting it equal to “0.” A relative performance value of “0” is appropriate for those cases where you can safely say that making that specific design feature change will not change (increase or decrease) the overall expected performance of the *Standard* pavement section.

Note: The user has the same access to the custom field names in both the *Cost Data Sets* and *Performance Data Sets* tabs. Therefore, a custom name defined when entering a cost data set will be reflected when defining performance data sets. Also note that custom field definitions are global in the sense that they are the same for all cost and performance data sets in the database (i.e., you cannot enter custom names that apply to specific cost or performance data sets).

<u>Example 2: Expected Relative Performance Values Associated with Different Initial/Smoothness Levels</u>

The relative performance changes (increase or decrease in pavement life) associated with different pavement initial smoothness ranges can be viewed as an estimate of the effect of dynamic loading effects on the pavement. That is, many believe that rougher pavements cause dynamic movements in vehicles, resulting in dynamic loadings that cause an increased deterioration of the pavement, therefore decreasing service life. These changes in expected pavement life should be quantified in this portion of the user interface.

The definition of relative performance associated with different initial smoothness ranges is different from other design feature categories in that the relative performance is associated with combinations of base/subbase type and initial smoothness range. Although the expected performance changes associated with different initial smoothness ranges are not expected to be greatly influenced by the underlying base type, performance is still entered for different combinations of smoothness range and base/subbase type to maintain consistency with the cost data sets. You will note that in the default performance data set, the expected performance differences associated with different initial smoothness ranges are the same for all of the included base types.

The first step in defining smoothness-related relative cost values is to select one of the base types listed under *Initial Smoothness for:* in the *Design Feature Category* list. Figure 13 illustrates an example (in tabular format) in which relative performance values were defined for different initial smoothness ranges. To accurately interpret the meaning of these smoothness-related relative performance values, you must keep their specific meaning in mind. An investigation of the *Default Performance Set* finds a value of +2.0 percent associated with the 79 to 110 mm/km (5 to 7 inches/mi) smoothness range (measured with a 5-mm (0.2-inch) blanking band). This value indicates that a pavement constructed with an initial smoothness value in the 79 to 110 mm/km (5 to 7 inches/mi) range (based on an 5-mm (0.2-inch) blanking band) is expected to carry 2.0 percent more ESALs than the same pavement constructed with an initial smoothness value in the 110 to 142 mm/km (7 to 9 inches/mi) range. That is, these expected changes in performance are solely due to the reduced initial smoothness value (i.e., most likely interpreted as the influence of initial smoothness on dynamic traffic loading-related damage). Therefore, when defining values associated with different combinations of initial smoothness and base/subbase type, be careful that you do not include the performance impact of changing the base/subbase type. The influence of base/subbase type on performance is represented under the base/subbase type design feature category inputs.

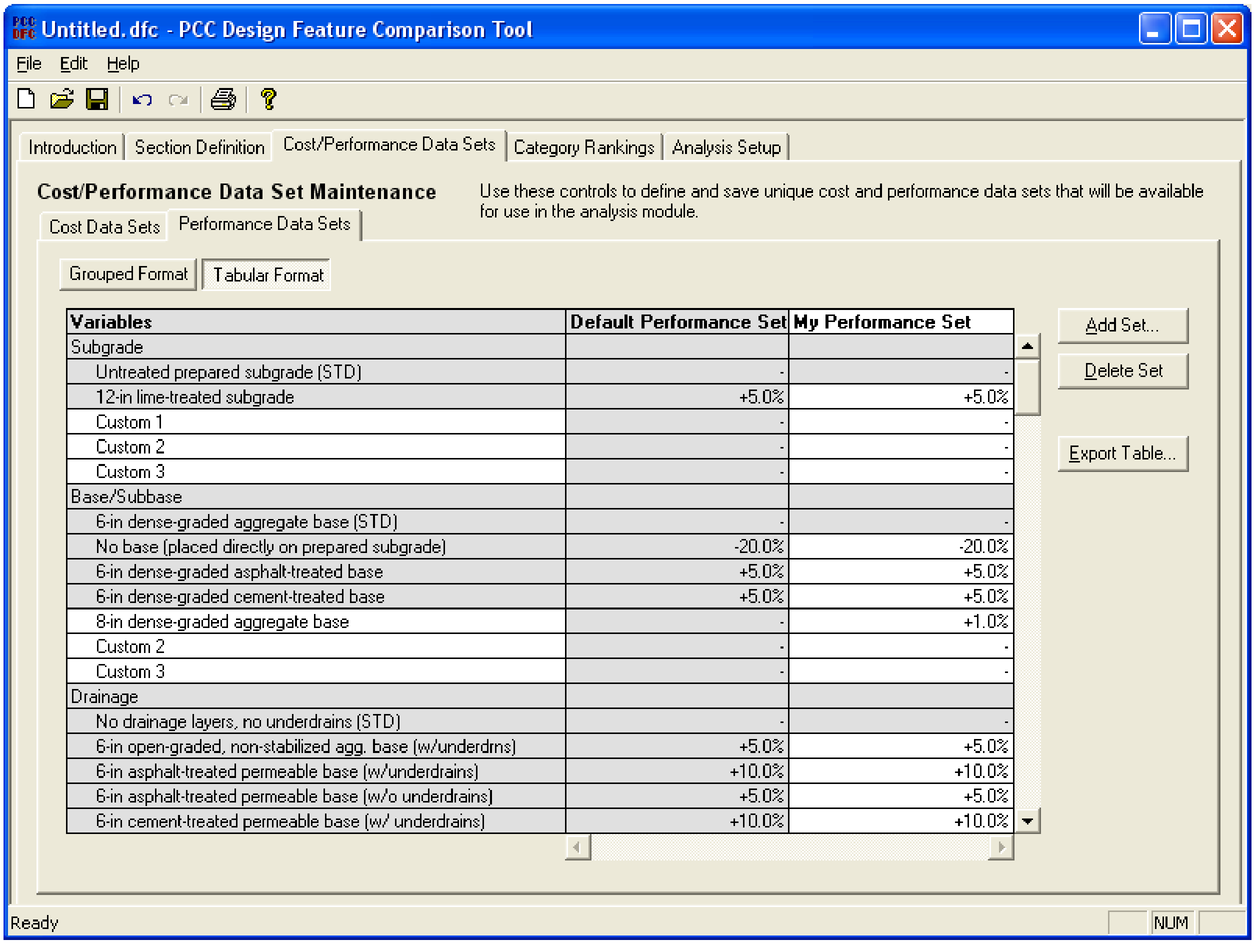


Figure 13. Example showing defined relative performance values associated with different initial smoothness choices.

SECTION 5. DEFINING CATEGORY RANKING FACTOR SETS

As noted previously, determining the overall expected change in performance is not as simple as summing the expected performance changes associated with changing different design features individually. For example, if we assume that the removal of dowels will decrease the expected performance by 20.0 percent, but the addition of the CTB will increase performance by 5.0 percent, this does not necessarily mean that overall performance will be reduced by 15.0 percent. Therefore, as explained in chapter 3 of the report, the combined performance is estimated by making use of user-defined design feature category ranking factors.

As with the cost and performance data sets, the software allows you to define and save different category ranking sets. A category ranking set is defined as a unique combination of ranking factors (integers from 1 to 10) chosen for each of the 10 different design feature categories. No two design features were allowed to share the same ranking, so the result is essentially a "forced ranking" of the importance of each design feature category. The remainder of this section includes an introduction to the *Category Rankings* tab and guidance on defining your own ranking factor sets.

Introduction to the *Category Rankings* Tab

The *Category Rankings* tab (shown in figure 14) provides controls that are used to define and edit user defined category ranking sets. The tab is divided into two areas titled *Category Rankings Master List* and *Category Ranking Set Details,* each of which is discussed separately below.

Category Ranking Set Master List Area

The *Category Ranking Set Master List* stores a complete list of the all category ranking sets defined within the current database. In addition to the user-defined category ranking sets, this master list also contains the *Default Rankings* set (shown in table 24) that is based on a combination of average survey results and expert opinion. Note that the default ranking factor list in table 24 is presented in descending order or importance (i.e., the most important category is assigned a value of 10 while the least important category is assigned a 1).

As with the other master lists in the software, you may use the supplied *Sort List* list box to sort the *Category Rankings Set Master List* alphabetically, by ranking set creation date, or by ranking set revision date. This area also contains *New*, *Delete*, and *Import* buttons that allow you to manage the contents of the master lists by creating new, deleting existing, or importing existing defined ranking factor sets, respectively. Note, however, the *Default Rankings* set cannot be deleted or altered by the user.

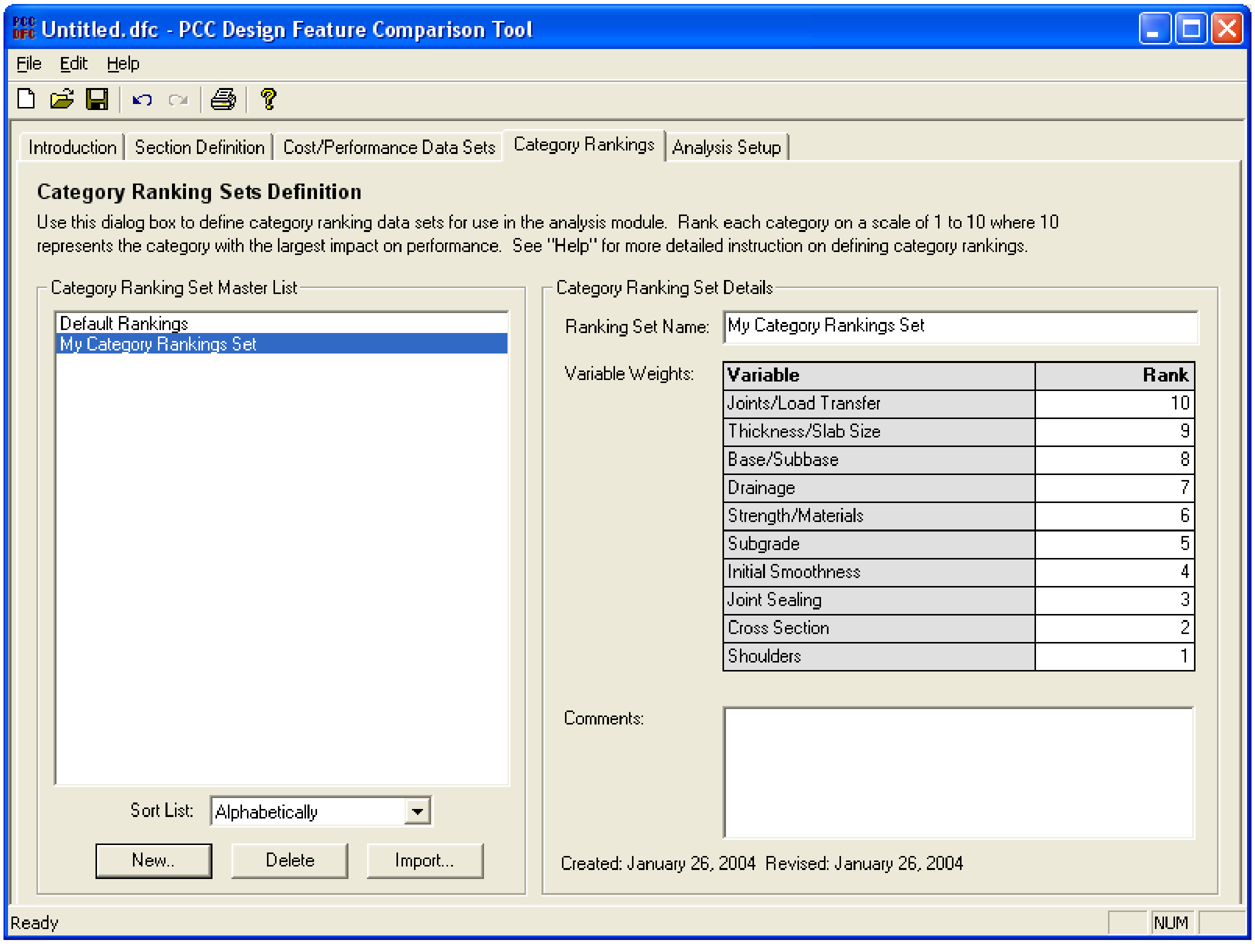


Figure 14. *Category Rankings* tab.

Table 24. Details of the default ranking factor set.

Design Category	Ranking Factor
Joints/Load Transfer	10
Thickness/Slab Size	9
Base/Subbase	8
Drainage	7
Strength/Materials	6
Subgrade	5
Initial Smoothness	4
Joint Sealing	3
Cross Section	2
Shoulders	1

Most Important

Least Important

<u>Category Ranking Set Details</u> <u>Area</u>

The *Category Ranking Set Details* area is used to define and edit user-defined category ranking sets. To edit the relative weights assigned to each design feature category, click on the ranking factor set of interest in the *Category Ranking Set Master List* and then enter appropriate values in the provided *Ranking Factors* input boxes. Note that whenever a ranking factor value changed, the displayed ranking factor list is immediately resorted in descending order to reflect the change. When defining a ranking factor set, you must abide by the following three value-associated rules:

1. Entered ranking factor values must be integers from 1 to 10.
2. The highest weighted design category must be assigned a value of 10. This provides some consistency to all defined ranking sets.
3. No two design categories are allowed to be assigned the same ranking factor.

To assign meaningful design feature category ranking factors, it is first important to understand specifically how these values are used. The following section contains a discussion that provides the reader with a better understanding of how ranking factors are used in the performance computations, thereby helping you to select ranking factors that correspond with your assumptions of each design category's impact on performance.

Defining Category Ranking Sets

One of the most difficult steps in the analysis approach is the assignment of category ranking factors that accurately reflect an agency's assessment of which design feature categories have the largest impact on overall performance. The use of defined ranking factors by the software is best explained with an illustrative example. Let's assume that we have an example (summarized in table 25) in which the subgrade, base/subbase, and drainage design features are changed simultaneously.

Table 25. Example computations using entered category ranking factors.

Design Feature Category	Standard Section Feature	Custom Section Feature	Expected Relative Performance (%)	Ranking Factor	Normalized Impact Multiplier	Modified Performance (%)
Subgrade	Untreated prepared subgrade	300-mm (12-in) lime treated subgrade	+5.0	5	(5/8) = 0.625	+3.1
Base/ Subbase	150-mm (6-in) dense-graded aggregate base	150-mm (6-in) ATB	+5.0	8	(8/8) = 1.00	+5.0
Drainage	No drainage layers, no underdrains	150-mm (6-in) ATPB with underdrains	+10.0	7	(7/8) = 0.875	+8.8
TOTAL						**+16.9**

In this example, the relative performance values of +5.0 percent, +5.0 percent, and +10.0 percent, respectively, were retrieved from the default performance data set. Next, we assume that the selected category ranking set contained factors of 5, 8, and 7 for the design categories of *Subgrade*, *Base/Subbase*, and *Drainage*, respectively. The associated ranking factors are then converted into normalized impact multipliers based on the largest observed ranking factor for only those design features changing from the *Standard* pavement section. For our example, the largest impact factor associated with the three changing feature categories is the "8" associated with *Base/Subbase*. Therefore, all three of the included impact factors are divided by "8" to compute normalized impact multipliers. These normalized impact multipliers are then multiplied by the associated expected relative performance values to give a modified performance value for each design category. The overall section performance is then determined as the sum of all modified performance values. For this example, the expected increase in performance is estimated to be 16.9 percent.

It is important to note that within this methodology, it is the relative differences between ranking factors rather than the actual ranking factor values that are important when determining overall modified performance. For example, one might think that the design feature assigned an impact factor of 10 is always going to be important when determining the overall pavement section performance. The previous example shows that this is not the case, as none of the three changing design feature categories had an impact factor of 10. Normalizing all individual ranking factors to the largest of the included factors ensures that the performance of the most important included design feature becomes the starting point of the modified performance computation. In the example, it is noted that if *Base/Subbase* were the only design feature category that was changing, then the total modified performance would be +5.0 percent. Therefore, the other design features deemed less important are, in a sense, used to adjust the +5.0 percent value associated with *Base/Subbase*. The normalized ranking multipliers give an indication of the relative impact of the adjustments.

When selecting category ranking factors, it is very important to assign factors that do not contradict the performance values observed within different design categories. That is, those design categories where the largest percent increases or decreases in performance are observed should most likely be the design categories with the largest category ranking factors. For example, assume that investigated *Thickness/Slab Size* choices result in individual performance changes from –40 to +50 percent, while different *Joint Sealing* choices result in a range of individual performance between –5 and + 5 percent. For this case, the category ranking factor assigned to *Thickness/Slab Size* should be significantly larger than that assigned to the *Joint Sealing*.

It is equally important to remember that the modified performance values resulting from the application of ranking factors are additive with the individual performance associated with the largest ranking factor is used as the starting point (e.g., the +5.0 percent was used as the starting point of the overall performance computation in the example illustrated table 25). This is because the category with the largest associated ranking factor is assumed to have the largest influence on overall performance. That is, in many cases, one should expect the overall performance to be close to the one individual performance value associated with the largest ranking factor as, by definition, it is the governing performance value. As mentioned previously, the ranking factors associated with other included design features are used to diminish those associated individual performance changes before adding them to the overall performance calculation. That is, all design feature categories that are not deemed to be the most important category (i.e., their ranking factors are less than the largest included ranking factor) are simply used to adjust the individual performance change associated with the largest ranking factor. It is the defined ranking factors that are used to determine the ranking factor ratios (normalized ranking multipliers) that determine the how much of each individual performance change is added to the overall performance value.

SECTION 6. ANALYSIS SESSION SETUP

The final tab of the user interface is the *Analysis Setup* tab. Using the controls of this tab, the user may setup, define, and conduct two different types of analysis sessions (i.e., *Direct Comparison* and *Sensitivity Analysis* sessions). Also provided are controls that allow the user to customize the contents of an output report using an on-screen preview window. The remainder of this chapter includes an introduction to the *Analysis Setup* tab, brief explanations of the purpose of the *Direct Comparison* and *Sensitivity Analysis* session types, and detailed information on the methods and controls used to define both analysis session types.

Introduction to the *Analysis Setup* Tab

The *Analysis Setup* tab (shown in figure 15) provides controls that are used to define and conduct two different types of analysis sessions: *Direct Comparison* sessions and *Sensitivity Analysis* sessions. Upon activating the tab, the user is first presented with two master lists representing these respective analysis types. (Note: When you are working with a new database, these master lists initially will be empty.)

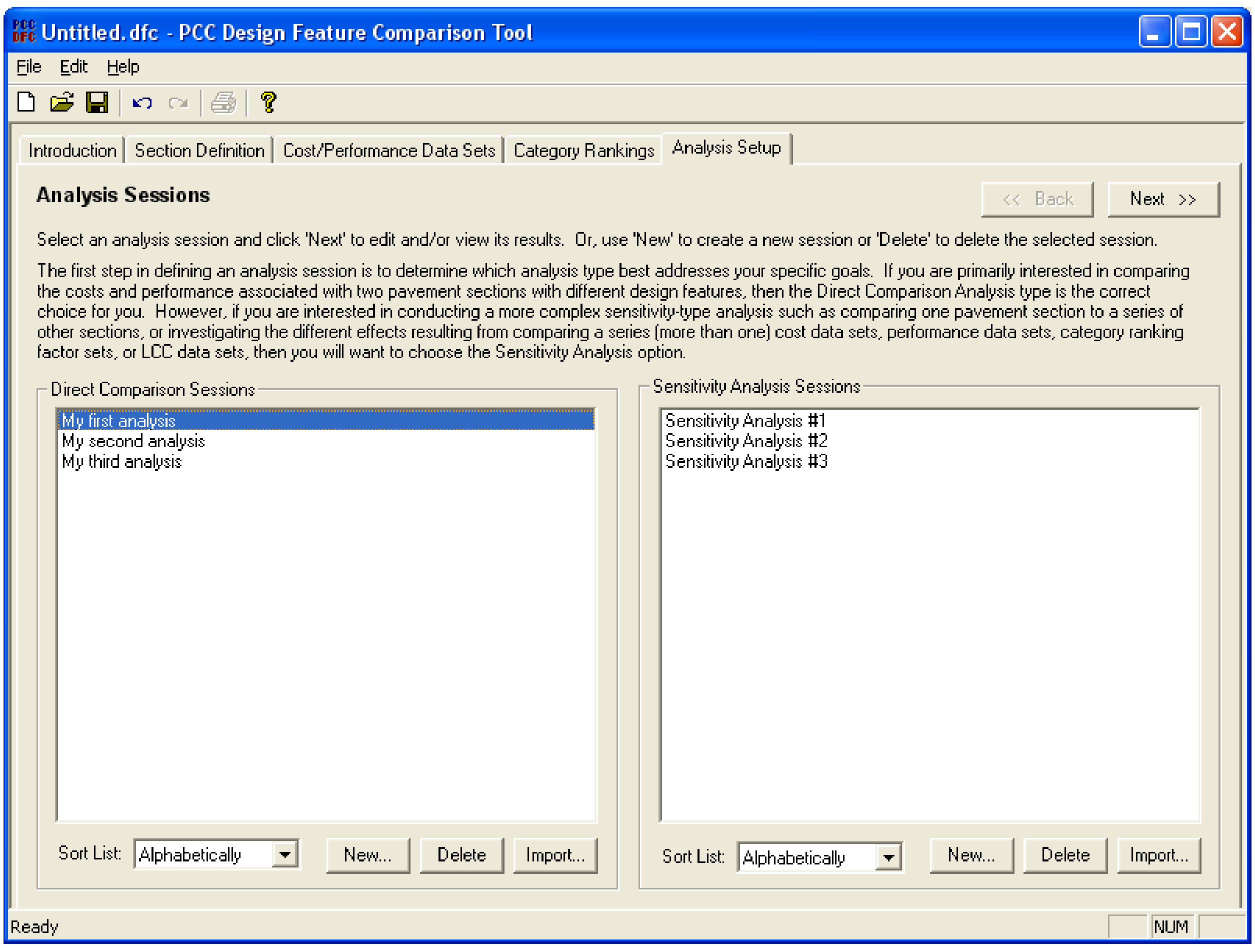


Figure 15. Example of the *Analysis Setup* tab.

Each master list stores a complete list of the all defined analysis sessions (by analysis type) defined within the current database. As with the other master lists in the software, you may use the respective *Sort List* boxes to sort each master list alphabetically, by creation date, or by revision date. This area also contains *New* and *Delete* buttons that allow you to manage the contents of the master lists by creating new or deleting existing analysis sessions, respectively.

Choosing an Analysis Type

The first step in defining an analysis session is to determine which analysis type best addresses your specific goals. If you are primarily interested in comparing the costs and performance associated with two pavement sections with different design features, then the *Direct Comparison* analysis type is the choice for you. However, if you are interested in conducting a more complex sensitivity-type analysis such as comparing one pavement section to a series of other sections, or investigating the different effects resulting from comparing a series (more than one) cost data sets, performance data sets, or category ranking factor sets, then you will want to choose the *Sensitivity Analysis* option. More specific discussions of both of these two analysis types are defined below.

<u>*Direct Comparison* Analyses</u>

The *Direct Comparison* analysis type is used to compare two defined pavement sections (labeled *Section A* and *Section B* in the analytical tool) to assess the expected differences in cost and performance between the sections. By carefully defining different pavement sections, this analysis type is used to directly assess the cost and performance impact of changing one or more design features. Note that in a *Direct Comparison* analysis, the other data collection modules (the cost data set, performance data set, and category ranking set) are all held constant in the analysis.

<u>*Sensitivity Analysis* Sessions</u>

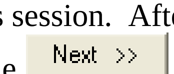
The *Sensitivity Analysis* type is included for users who want to conduct more complex one- or two-dimensional sensitivity analysis investigations. These analysis sessions are defined by: 1) selecting one primary pavement section to be used as the basis for the investigation; 2) selecting one of five different analysis session types from a provided list (these are described below); and 3) selecting the specific parameters that define the scope of the analysis.

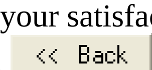
The following are the five specific types of sensitivity analyses from which the user may choose:

- *Comparing multiple sections*—A series of direct comparisons in which one selected pavement section is systematically compared to each in a series of other selected pavement sections. Expected cost and performance changes associated with each comparison are summarized in the output report.
- *Comparing cost data sets*—An investigation of cost sensitivity resulting from analyzing the use of different cost data sets (i.e., the only data collection variable changing in a series of analysis scenarios is the cost data set). Note: The primary pavement section definition, performance data set, and category ranking factor set are held constant during the analysis session.

- *Comparing performance data sets*—An investigation of performance sensitivity resulting from analyzing the use of different performance data sets (i.e., the only data collection variable changing in a series of analysis scenarios is the performance data set). Note: The primary pavement section definition, cost data set, and category ranking factor set are held constant during the analysis session.
- *Comparing category ranking sets*—A sensitivity analysis that investigates the use of different category ranking sets (i.e., the only data collection variable changing in a series of analysis scenarios is the category ranking set). Note: The primary pavement section definition, cost data set, and performance data set are held constant during the analysis session.
- *Comparing both cost and performance data sets*—A two-dimensional sensitivity analysis that simultaneously investigates different combinations of cost and performance data sets. This is a factorial-type analysis that defines a series of individual analysis scenarios by investigating all possible combinations of the different included cost and performance data sets. Note: The primary pavement section definition and category ranking set are held constant during the analysis.

Defining *Direct Comparison* Analysis Sessions

When you create a new (or edit an existing) *Direct Comparison* analysis session, the dialog box presented in figure 16 is displayed. The controls on this dialog box lead the user through a series of steps that will complete the setup of a *Direct Comparison* analysis session. After the analysis session setup steps have been completed to your satisfaction, click the 

button to view the analysis results. Alternatively, click the 

button to return to the analysis session master lists. Each specific control on this page is described in detail in this section.

<u>Changing the Name of the *Direct Comparison* Analysis Session</u>

To change the name of the current *Direct Comparison* analysis session, enter the new name in the *Name* input box. Note: The name displayed in the input box is the name that will be displayed in the *Direct Comparison Sessions* master list.

<u>Defining the First Pavement Section (Section A)</u>

The first step in defining the *Direct Comparison* analysis session is the selection of the first pavement section to be compared (Section A). To define *Section A*, select a named pavement section from the list box within the *Step 1: Select Section A* area (note: The named pavement sections contained in this list box are those pavement sections stored in the *Pavement Section Master List*.) When you select a section from the list, the design category values associated with

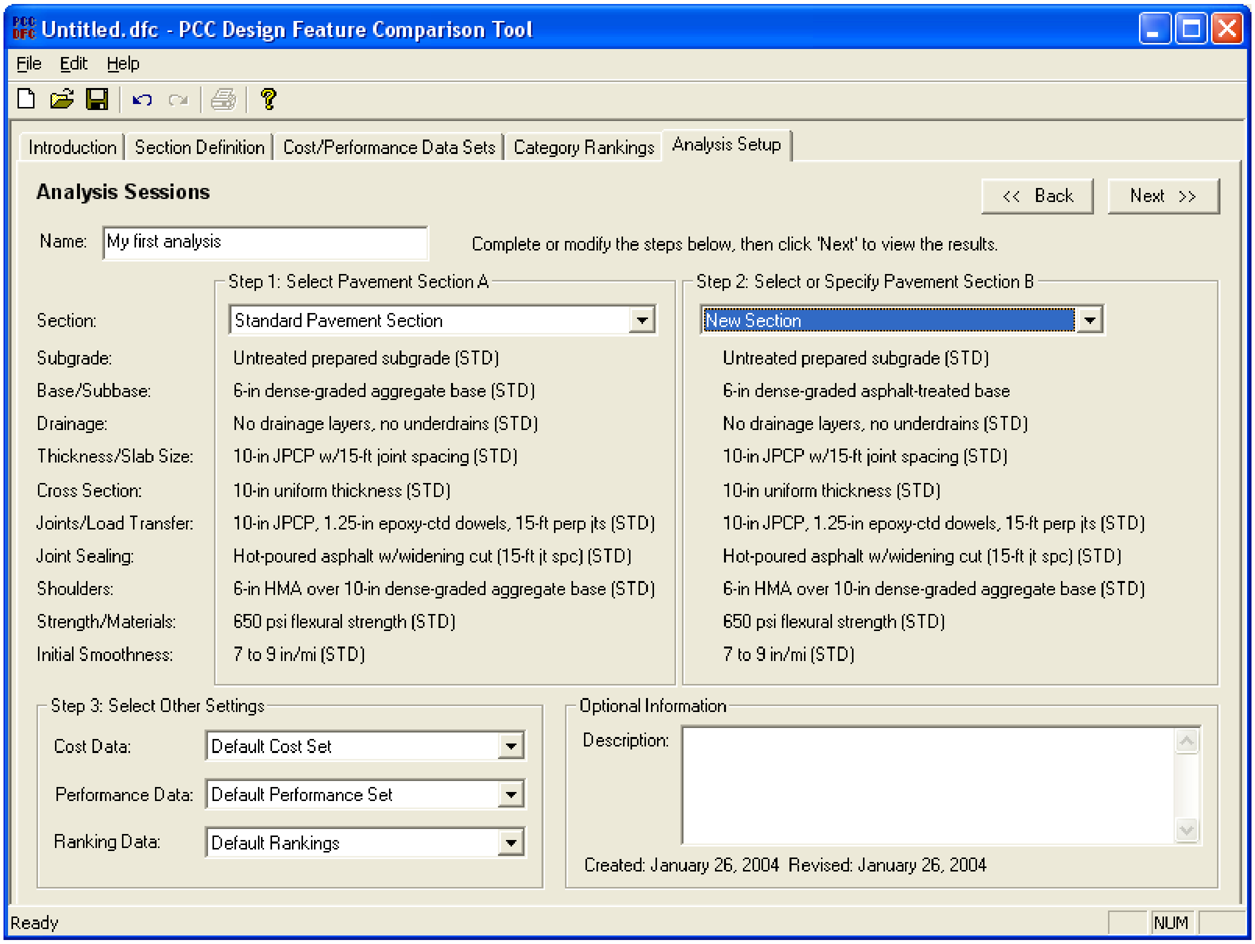


Figure 16. Example of the *Direct Comparison* analysis session setup dialog box.

Section A will be reflected below the list box. Those design feature values that match those that define the *Standard* pavement section (see table 21) are indicated by the "(STD)" that follows the feature description.

<u>Defining the Second Pavement Section (Section B)</u>

The second step in defining the *Direct Comparison* analysis session is the selection of the second pavement section (*Section B*) that will be compared to *Section A*. *Section B* may be defined by one of two methods:

1. *Select an existing pavement section*. As with the selection of *Section A*, you may select a previously defined pavement section from the list box within the *Step 2: Select or Specify Pavement Section B* area. As with the *Section A* list, the named pavement sections contained in this list box are those named and stored in the *Pavement Section Master List*. When you select a section from the *Section B* list box, the associated design category values will be reflected below the list box (note that you do not have the option to change these design feature values as they are associated with a previously defined and named pavement section).

2. *Manually define Section B.* The software also gives you the option of manually selecting the design features that define *Section B.* When this option is used, *Section B* is not linked to an existing saved pavement section. This option is provided for those who wish to investigate many what if-type scenarios without having to define and keep track of many different saved pavement sections. Therefore, the user may quickly view output results associated with tweaking the design features that define the *Section B* design.

Selection of Other Settings

The last required step of the *Direct Comparison* analysis involves defining the nonvarying data collection modules contained in the *Step 3: Select Other Settings* area. This involves selecting a cost data set, performance data set, and category ranking set from their respective list boxes. The items in each of these list boxes are those that are available in their respective master lists. Therefore, to see the detailed make up of a particular named data collection module, visit the appropriate tab and select that item from the corresponding master list.

Additional Information Area

The final section of this direct analysis dialog is an area labeled *Additional Information*. The *Description* input box is provided for users who would like to enter text that describes the focus of a given analysis session (note: entering text in this input box is completely optional). The second part of this additional information area is the displayed analysis session *Creation Date*.

Defining *Sensitivity Analysis* Sessions

In a *Sensitivity Analysis* session, various analysis scenarios are considered, and the resulting cost, performance, and LCC results are compared for all scenarios. When you create a new (or edit an existing) *Sensitivity Analysis* session, the dialog box presented in figure 17 is displayed. The controls on this dialog box lead the user through a series of steps that will complete the setup of a *Sensitivity Analysis* session. Each of the specific controls on this page is described in detail in this section.

Changing the Name of the *Sensitivity Analysis* Session

To change the name of the current *Sensitivity Analysis* session, enter the new name in the *Name* input box. Note: The name displayed in the input box is the name that will be displayed in the *Sensitivity Analysis* session's master list.

Selecting a Principal Pavement Section To Be Used as the Basis of the *Sensitivity Analysis* Session

The first step in defining a *Sensitivity Analysis* session is the selection of a principal pavement section that is used as the basis of the analysis. If multiple pavement sections are being compared, it is to this principal section that all pavement sections in the series are compared. For the other sensitivity analysis types that investigate the use of different cost data sets, performance data sets or category ranking sets, the principal pavement section is held constant throughout all defined analysis scenarios.

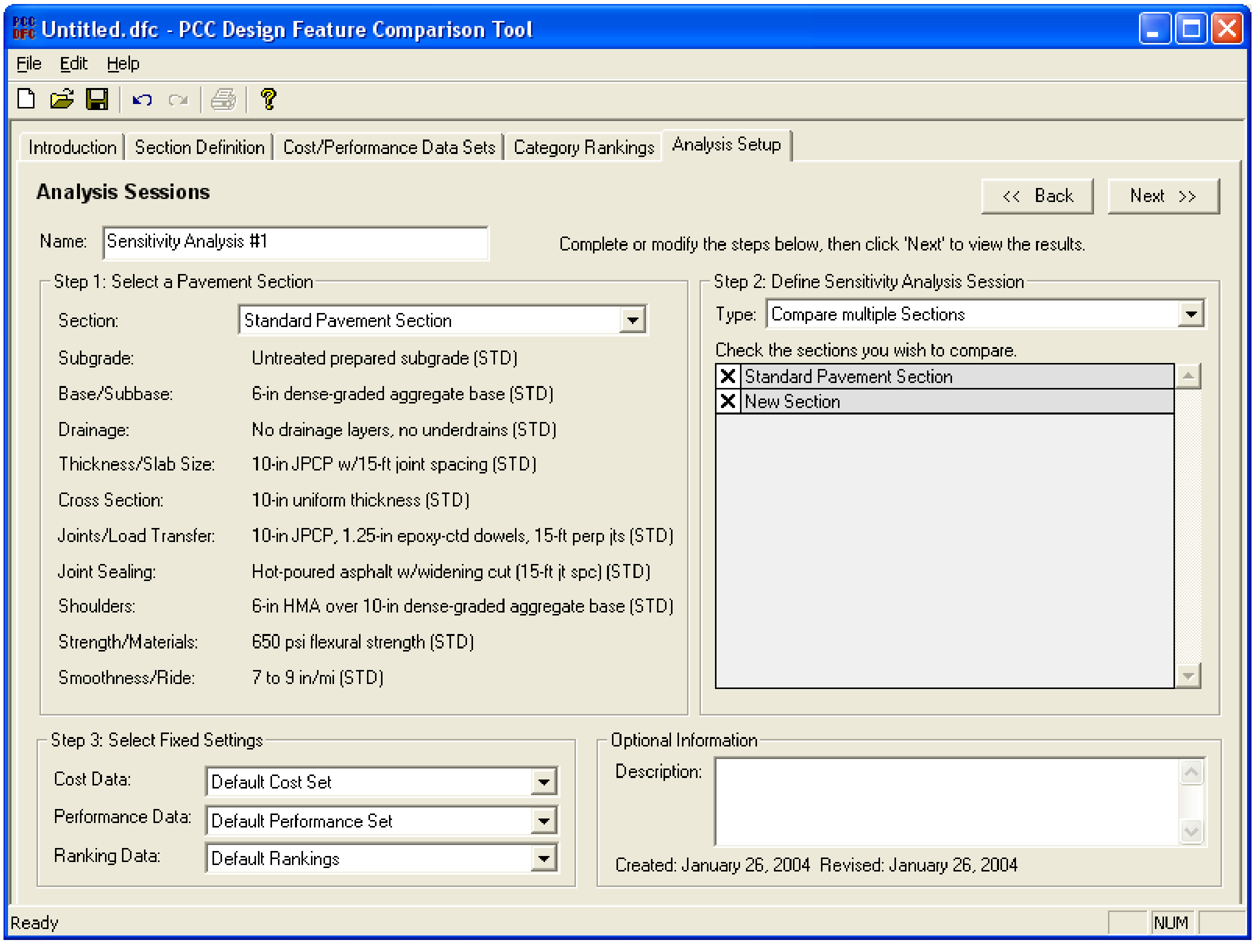


Figure 17. Example of the *Sensitivity Analysis* session setup dialog box.

To define the principal section, select a named pavement section from the list box within the area labeled *Step 1: Select a Pavement Section*. The named pavement sections contained in this list box are those stored in the *Pavement Section Master List*. When you select a section from the list, the design category values associated with chosen section are reflected below the list box.

Define the Type and Parameters of the *Sensitivity Analysis* Session

After selecting a principal pavement section, you must next select the specific type of *Sensitivity Analysis* you wish to conduct, as well as set the parameters that define the analysis. To accomplish these tasks, use the controls provided in the frame titled *Step 2: Define Sensitivity Analysis Session*. Specifically, the five different dimensional analysis choices contained in the *Type* drop-down list box are the following (note: each of these was defined earlier in this chapter):

- Compare multiple pavement sections.
- Compare cost data sets.
- Compare performance data sets.
- Compare category ranking sets.
- Compare both cost and performance data sets (2-dimensional).

Upon selecting one of these analysis types, you will be provided with a list of related available items (i.e., pavement sections or cost, performance, or category ranking data sets) that can be included in the analysis. By default, all available items are initially included in the analysis. Included items are indicated by an "X" placed in the box to the left of the item's name. To exclude an item from the analysis, simply click on the associated box that contains an "X." To include a previously excluded item in the analysis, click on the empty box to again display the "X."

To illustrate this analysis setup process, figure 18 shows an example in which the user has selected a 2-dimensional analysis in which both cost and performance data sets will be compared simultaneously. Specifically, the defined analysis will investigate all of the analysis scenarios representing the different combinations of one cost set (*My Cost Set*) and all three available performance data sets.

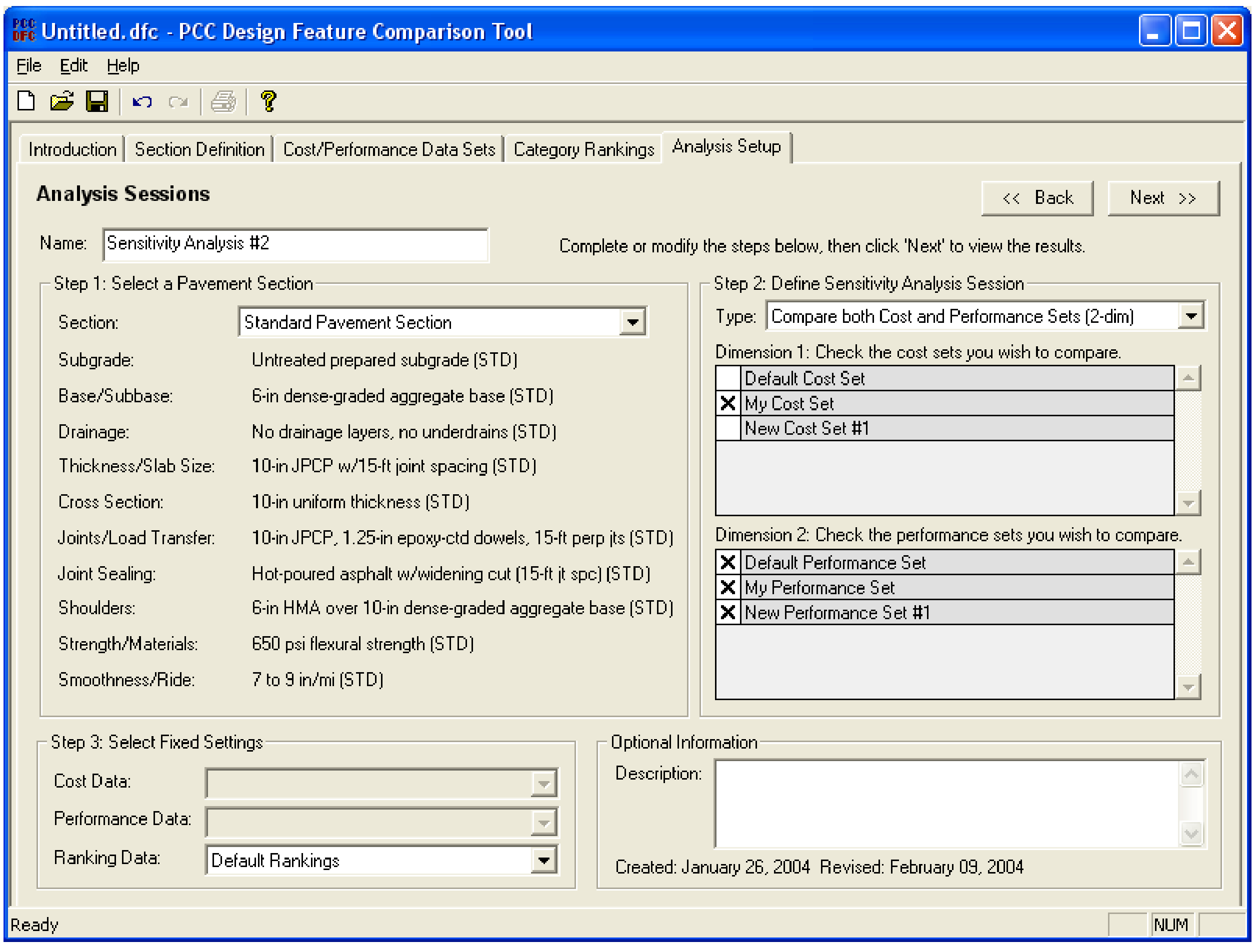


Figure 18. Example of a *Sensitivity Analysis* session comparing both cost and performance data sets.

Selection of Fixed Settings

The last required step of the *Sensitivity Analysis* setup involves the definition of the nonvarying data collection modules contained in the *Step 3: Select Fixed Setting* area. This involves the selection of those data collection modules that remain constant throughout the *Sensitivity Analysis*. To define these items, select the appropriate named item from each provided list box. The items in each of these list boxes are those that are available in their respective master lists. Therefore, to see the detailed makeup of a particular named data collection module, visit the appropriate tab and select that item from the corresponding master list.

Additional Information Area

The final section of this direct analysis dialog is an area labeled *Additional Information*. The *Description* input box is provided for users who would like to enter text that describes the focus of a given analysis session (note: entering text in this input box is completely optional). The second part of this *Additional Information* area is the displayed analysis session *Creation Date*.

SECTION 7. ANALYSIS SESSION RESULTS

The results of the current defined analysis session are summarized into a customizable output report that may be reviewed within the software. To view the results from an analysis session, click the 

button after you have defined the inputs for the three required steps of the analysis session setup process. The remainder of this chapter introduces the specific output reports associated with the *Direct Comparison* and *Sensitivity Analysis* session types.

Viewing *Direct Comparison* Analysis Results

By default, all of the available *Direct Comparison* analysis results are presented in the summary report (i.e., the *Full Report* format). However, if you are only interested in viewing specific summary tables, you may choose the *Basic Tables Only* option. Selecting this option allows you to customize the simplified output report by choosing to view one or more of the following summary tables:

- Cost details.
- Performance details.
- Benefit/cost summary.
- LCC analysis summary.

More details on both the *Full Report* and *Basic Tables Only* reporting options are discussed separately below.

Direct Comparison Analysis—Full Report

To view all of the details associated with the conducted *Direct Comparison* analysis, click the *Full Report* check box when analysis results are displayed (see figure 19). Specifically, the many different sections that are included in the full report include the following:

- *Introduction*—A quick summary of the *Direct Comparison* analysis, including definitions of which sections were chosen for *Section A* and *Section B,* and a summary table of the other data collection module choices (i.e., the chosen cost data set, performance data set, and category ranking set) used in the analysis.
- *Definition of the Default “Standard” Pavement Section*—Because the analysis results often make reference to the defined default *Standard* pavement section, this section of the output report summarizes the unique combination of design features the define the *Standard* pavement section.
- *Percent Changes in Cost and Performance by Design Feature*—The first step in estimating the relative changes in cost and performance between the two sections being compared is to summarize the respective percent changes taken from the chosen cost and performance data sets. By definition, these percent changes are relative to the expected costs and performance of the *Standard* pavement section. This section or the full report summarizes all of these individual changes for both *Section A* and *Section B.*

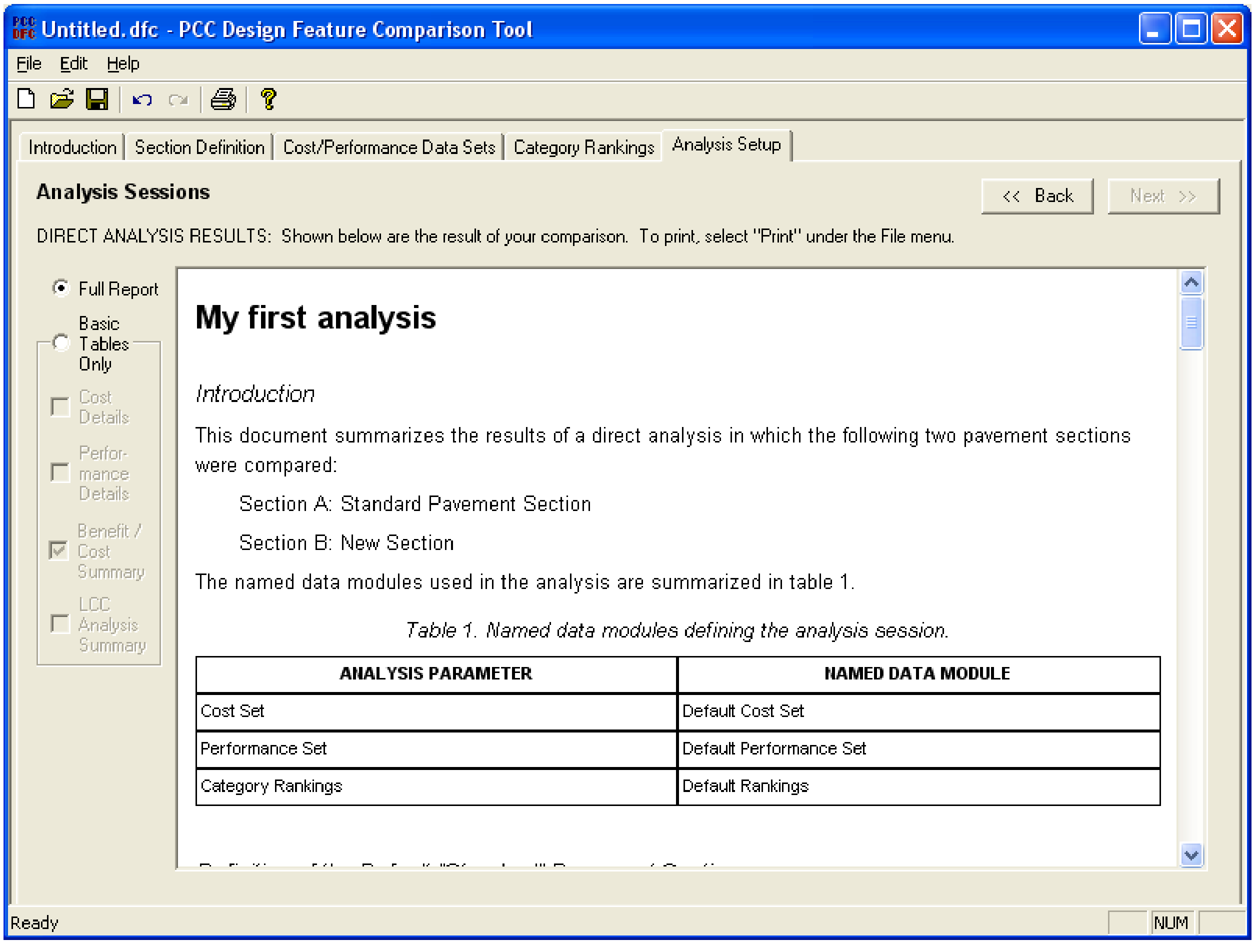


Figure 19. Example of a *Full Report* summary resulting from a *Direct Comparison* analysis.

- *Ranking Factor Summary*—The section of the output report summarizes the ranking factor set chosen for use in the analysis.
- *Cost Summary*—A summary of the detailed cost calculations. Specifically, this table presents a summary of relative cost changes and their associated cost multipliers, as well as an overall ratio of costs between *Section A* and *Section B* (presented as a *B/A Cost Multiplier*).
- *Performance Summary*—A summary of the detailed performance-related calculations. Specifically, this section of the output report contains two tables showing the detailed performance computations for *Section A* and *Section B*, respectively, as well as an overall summary table showing performance changes for both sections.
- *Benefit/Cost Summary*—The summary table presented in this section of the report summarizes the relative performance multiplier, relative cost multiplier, and benefit/cost ratio associated with *Section A* and *Section B*, respectively. Each of these results is explained below:
  - *Relative Performance Multiplier*—The ratio of the expected *Section A* or *Section B* performance as it relates to the expected performance of the *Standard* pavement section. For example, if the relative performance multiplier associated with *Section B*

was 0.950, this would indicate that *Section B* is expected to carry 95.0 percent of the ESALs expected to be carried by the *Standard* pavement section.

  – *Relative Cost Multiplier*—The ratio of the expected *Section A* or *Section B* cost as it relates to the expected cost of the *Standard* pavement section. For example, if the relative cost multiplier associated with *Section B* was 0.980, this would indicate that *Section B* is expected to cost 98.0 percent as much as the *Standard* pavement section.
  – *Benefit/Cost Ratio*—For a given section, the benefit/cost ratio is the ratio of the relative performance multiplier to the relative cost multiplier. For this example, the benefit/cost ratio associated with *Section B* is computed as 0.950/0.980 = 0.969.

- *Simple Life-Cycle Cost (LCC) Analysis*—The results of this simple LCC analysis are presented in this section of the output report.

To print the defined output report, use the *Print* option included under the *File* menu or the *Print* toolbar button.

<u>*Direct Comparison* Analysis—Basic Tables Only</u>

The *Basic Tables Only* report option allows you to build a simplified version of the output report (see figure 20). Only tables selected by the user (indicated by the check boxes) are displayed in the output report with limited explanatory text.

Viewing *Sensitivity Analysis* Session Results

As with a *Direct Comparison* analysis, *Sensitivity Analysis* results are by default presented in a full report format (i.e., *Detailed Results*). However, if you are only interested in viewing a summary table of the cost and performance results associated with each analysis scenario included in the analysis session, you may deselect the *Detailed Results* check box. Both the detailed results and simplified summary reports are discussed separately below.

<u>*Sensitivity Analysis*—Detailed Results Report</u>

To view all of the details associated with the conducted *Sensitivity Analysis* session, the *Detailed Results* check box needs to be selected (see figure 21). The detailed report provides many of the lower level calculations associated with each analysis scenario. Specifically, the detailed *Sensitivity Analysis* report contains the following three main sections:

- *Introduction*—A quick summary of the *Sensitivity Analysis* session including definitions of the different analysis scenarios, as well as a summary table of the other data collection module choices that remain constant during the analysis.
- *Cost and Performance Summary*—A combined summary of cost, performance, and benefit/cost results summarized for all of the analysis scenarios of the analysis session.
- *Life-Cycle Cost Analysis Summary*—A summary of the results from the LCCA conducted for each of the analysis scenarios included in the analysis session.

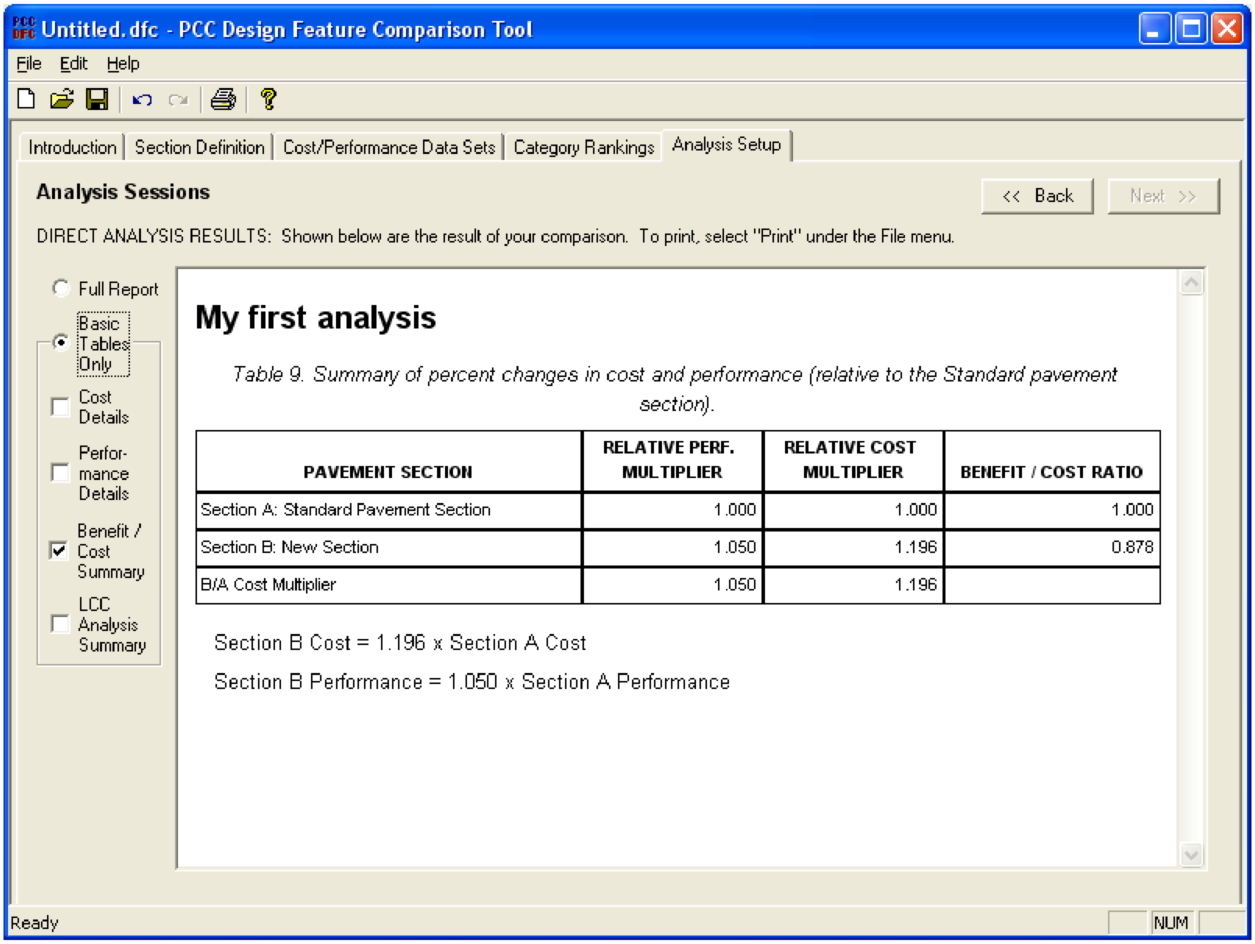


Figure 20. Example of the *Basic Tables Only* output report resulting from a *Direct Comparison* analysis.

To print the defined output report, use the *Print* option included under the *File* menu or the *Print* button on the toolbar.

<u>*Sensitivity Analysis*—Simplified Output Report</u>

If you choose to deselect the *Detailed Results* check box, you will only view an overall summary table of results associated with the conducted *Sensitivity Analysis* session. Each row of the table summarizes the relative performance multiplier, relative cost multiplier, and benefit/cost ratio associated with each included analysis scenario. Note: A detailed explanation of these multipliers and the benefit/cost ratio was provided in the *Direct Comparison Analysis—Full Report* section.

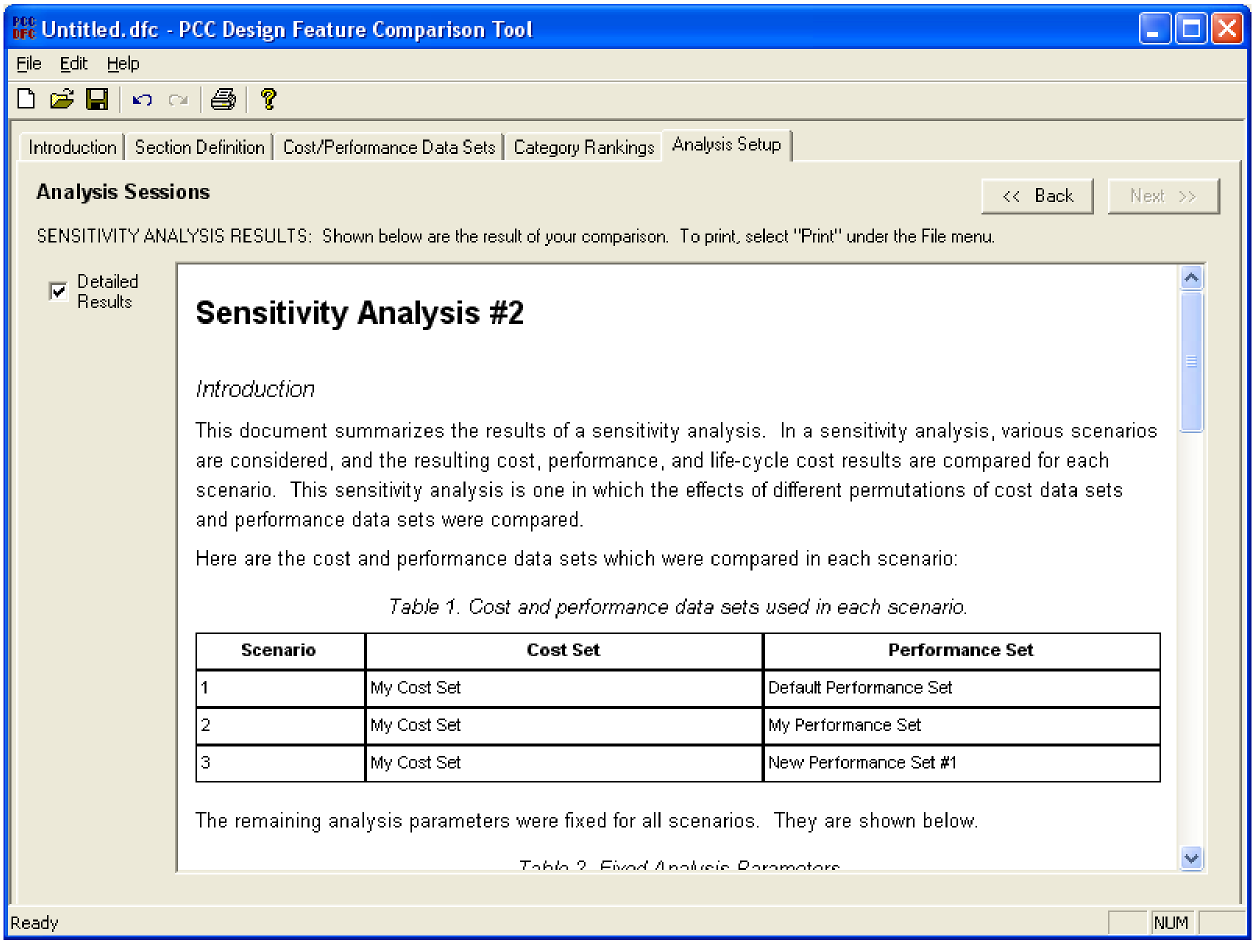


Figure 21. Example of a *Detailed Results* summary table output resulting from a *Sensitivity Analysis* session.

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