Structural Factors of Jointed Plain Concrete Pavements: SPS-2—Initial Evaluation and Analysis

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LTPP

FOREWORD

The Long-Term Pavement Performance (LTPP) program Specific Pavement Studies 2 (SPS-2) experiment, *Strategic Study of Structural Factors of Jointed Plain Concrete Pavements (JPCP)*, is one of the key components of the LTPP program. The main objective of this experiment is to determine the relative influence and long-term effectiveness of JPCP design features and site conditions on performance. This report documents the first comprehensive review and evaluation of the SPS-2 experiment as it exists today. The evaluation concludes that many important and useful findings and results can be obtained from the SPS-2 sites despite several limitations resulting from not constructing a few of the test sites and a few construction deviations that occurred. In addition, some materials and traffic data are missing from some sites or sections. These data are important to achieving the objectives of the experiment, and are now being sought from the SPS-2 sites.

Some interesting and important early trends have been identified that will be useful to the design and construction of JPCP, even though the oldest sections were no more than 7.5 years old at the time of this study. As time and traffic loadings accumulate at the SPS-2 sites, additional valuable performance data will be obtained. For example, the direct comparison of performance of designs with and without a permeable subdrainage layer is of intense interest to the State highway agencies. Future analyses of the performance data from the SPS-2 experiment will lead to new and important findings on the value of subdrainage, base type (treated and unbound), widened lanes, strength of concrete, subgrade soil, traffic level, and climate. These findings will lead to more reliable and cost-effective designs of JPCP.

This report will be of interest to highway agency engineers involved in design, construction, and management of the pavements as well as future researchers who will analyze the performance of the SPS-2 sections.

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16. Abstract

The SPS-2 experiment, *Strategic Study of Structural Factors for Jointed Plain Concrete Pavements (JPCP)*, is one of the key components of the LTPP program. The main objective of this experiment is to determine the relative influence and long-term effectiveness of JPCP design features (including slab thickness, PCC flexural strength, base type and drainage, and slab width) and site conditions (traffic, subgrade type, climate) on performance. This report documents the first comprehensive review and evaluation of the SPS-2 experiment. Thirteen SPS-2 projects have been constructed with one additional site under construction. At each site, there are 12 core sections plus various numbers of supplemental sections.

The data availability and completeness for the SPS-2 experiment are good overall. A high percentage of the SPS-2 data are at level E—greater than 82 percent for all data types, and greater than 99 percent for many. However, a significant amount of data are still missing, especially traffic, distress and faulting surveys, and key materials testing data. These deficiencies need to be addressed before a comprehensive analysis of the SPS-2 experiment is conducted. Required experimental pavement design factors and site conditions were compared with the actual constructed values. Most SPS-2 sections follow the experiment design for the large majority of the design factors. When comparing designed versus constructed, eight SPS-2 projects can be characterized as good to excellent, four projects are considered poor to fair, and one new SPS-2 project does not yet have enough data in the IMS database to be evaluated.

The evaluation has shown that several problems may limit the results that can be obtained from the SPS-2 experiments if not rectified. Specifically, no SPS-2 projects were built on certain subgrade types and in some climates. Some SPS-2 sites had construction deviations, and significant materials data and traffic data are missing from other sites or sections. One site has excessive early cracking that will limit its usefulness. However, even though the SPS-2 sections are relatively young (oldest project is 7.5 years) and a large majority show no or little distress, some interesting and important early trends have already been identified that will be very useful to the design and construction of JPCP. As time and traffic loadings accumulate, much more valuable performance data will be obtained. The Federal Highway Administration is conducting a concerted effort to obtain missing data. Recommendations for future analyses are provided in the last chapter of this report. Valuable information will be obtained from this experiment if these studies are carried out.

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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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1. INTRODUCTION

The Specific Pavement Studies 2 (SPS-2) project, titled *Strategic Study of Structural Factors for Jointed Plain Concrete Pavements,* was designed as a controlled field experiment that focuses on the study of specific design features (structural factors) for doweled jointed plain concrete pavements (JPCP). It is expected that the successful completion of this experiment will lead to improvements in design procedures and standards for construction of rigid pavements. These improvements will contribute to achieving the overall goal of the Long-Term Pavement Performance (LTPP) program—increased pavement life and better utilization of resources.

This goal is expected to be achieved through investigation of the effects of the specific experimental design features and site conditions (subgrade soil, traffic, and climate) and their interactions on pavement performance. That investigation will make possible the evaluation of existing design methods and performance equations, as well as the development of new and improved design equations and calibration of mechanistic models (including the *2002 Design Guide*).

BACKGROUND

The SPS-2 experimental plans were originally designed to incorporate project sites in all four LTPP climatic regions (dry freeze, wet freeze, dry no-freeze, wet no-freeze) and on both fine-grained and coarse-grained subgrades. This requirement makes it possible to cover a large inference space of the continental United States. A major effort was made by the Strategic Highway Research Program (SHRP), State highway agencies (SHAs), and the Federal Highway Administration (FHWA) to identify appropriate SPS-2 sites and to construct all the sections according to their original experimental design. A wide range of specific data was collected during construction. Extensive field monitoring data (traffic, profile, cracking) have been collected from these sections over time.

The original expectations for the LTPP program are summarized in the SHRP-P-395 report.[1] Originally, the following objectives were established:

- Evaluation of existing design methods.
- Development of improved strategies and design procedures for the rehabilitation of existing pavements.
- Development of improved design equations for new and reconstructed pavements.
- Determination of the effects on pavement distress and performance of loading, environment, materials properties and variability, construction quality, and maintenance levels.
- Determination of specific design procedures to improve pavement performance.
- Establishment of a database to support these objectives and future needs.

The designs for various LTPP experiments were developed with a clear relationship to these objectives. The following products were identified for the LTPP program:

1. General Products: Evaluation of existing design methods and performance equations, new and improved design equations, and calibration of mechanistic models.

2. Specific Products: The effects of the specific experimental design features (i.e., permeable drainage layers, widened slabs, asphalt concrete (AC) overlay thickness, pre-overlay repair, and many others) and site conditions (e.g., subgrade soil, traffic, climate, and their interactions).

3. Other Products: Test methods developed specifically for SPS test sections, correlations between material properties determined by different methods, study of other features and materials, and technology transfer.

The following objectives of the SPS-1 (new flexible pavement) and SPS-2 (new rigid pavement) experiments are stated in the same report:

- "The SPS will develop a comprehensive database with information on construction, materials, traffic, environment, performance, and other features pertaining to the test sections."
- "The primary objective of the experiments on structural factors for flexible and rigid pavements is to more precisely determine the relative influence and long-term effectiveness of the strategic factors that influence the performance of pavements."

As the SPS experiments have been constructed and monitored over time, many concerns have been expressed regarding the ability of those efforts to satisfactorily meet the stated expectations. These concerns include the following:

- Lack of more detailed expectations and objectives from each of these SPS experiments.
- The quality and completeness of available data now and in the future.
- Deviations in the design and construction features of in-place test sections (e.g., layers built to a different thickness or lack of compaction of the subgrade).
- Deficiencies in construction, materials, climate, traffic, and performance data in relation to current and future analysis needs.

The availability of reliable traffic and materials data is perhaps the major concern for the SPS experiments, and efforts are underway to resolve these concerns.

It is known that some of SPS project sites were not constructed in some climatic areas because of lack of interest by the SHAs or lack of suitable sites, leaving a portion of the desired inference space with no performance data. It is also known that some of the SPS project sites were not constructed in complete conformity with the original experimental plans. Despite best efforts, the inventory and monitoring data collected from these sections during construction and for several years afterward may be deficient in some areas.

The full extent of deviation, and the potential impact of that deviation, have not yet been fully evaluated for most of the SPS experiments. Thus, this study was initiated to conduct a comprehensive review of all SPS-2 experimental sites. This review compares the experiment sites as they exist today with the original expectations and, in addition, compares these projects as they exist today with any new expectations for the 21st century. For example, there is now a greater emphasis on mechanistic-based design. This review provides a sound basis for the following:

- Planning remedial actions that may be warranted due to various deficiencies in construction or data collection.
- Decisions regarding future monitoring and data collection.
- Planning future analysis of the collected data.

Issues of experimental design (e.g., existence of planned SPS projects), construction quality, data quality, and data completeness (with respect to both current data collection guidelines and anticipated pavement engineering needs) need to be addressed.

The SPS-2 projects were constructed between 1992 and 1997 (with one site completed in 2000), indicating that they are fairly young and may not yet directly support analysis activities to improve our knowledge in many of the above-listed areas. However, a few of the weaker SPS-2 sections have exhibited distress; thus, it may now be possible to make some preliminary evaluations. However, no in-depth assessment has been undertaken to date to determine the extent to which these two experiments will provide the necessary data to ensure that the broader expectations are attained.

This evaluation of SPS-2 is being conducted at the same time, and in coordination with, evaluation of the SPS-1 (new flexible pavement), SPS-5 (rehabilitated flexible pavement), and SPS-6 (rehabilitated rigid pavement) projects.

STUDY OBJECTIVES

This review concentrates on the core experimental sections that were included in the experimental design for SPS-2 projects. In addition, the SHAs often added supplementary sections to each SPS project that do not fit any formal controlled experimental plan. The value of these sections was also evaluated.

The objectives of this study are as follows:

1. Identify specific objectives and expectations that should be pursued for the SPS-2 experiment, given the original expectations and future needs. Consider the expectations at the local SHA level, the regional level, and the national level as appropriate.

2. Evaluate the set of core and supplemental test sections constructed in the SPS-2 experiment in relation to their ability to support the objectives and characterize the overall health and analytical potential of each SPS experiment. Identify areas of strength and weakness, and recommend corrective measures, as appropriate, to strengthen the SPS-2 experiment to accomplish its objectives. Develop analysis plans for both the short term and the long term.

3. Identify confounding factors introduced into each SPS experiment by virtue of construction deviations or other factors not accounted for in the original experimental design.

4. Evaluate the quality and completeness (in relation to current data collection requirements) of the SPS construction data. Provide recommendations for the resolution or correction of data that are anomalous or of inadequate quality.

5. Evaluate the adequacy of existing data and current data collection requirements in relation to anticipated analytical needs. Identify areas where current requirements are excessive or deficient, and provide recommendations where adjustments (in quantity, quality, frequency, or data type) are warranted.

6. Consider both short-term and long-term timeframes in the evaluation and preparation of data analysis recommendations.

7. Evaluate the opportunities for local, regional, or national analysis of the core and supplemental sections.

REPORT ORGANIZATION

Chapter 2 focuses on the original SPS-2 experimental design and compares this with the SPS-2 projects actually constructed. Chapter 3 reviews the SPS-2 experiment data availability and completeness. This includes a detailed discussion of the quantity and percentage of level E data available in the Information Management System (IMS) database. Chapter 4 presents a comparison of the designed versus constructed section parameters. A comprehensive status assessment of each of the SPS-2 experimental projects is provided in chapter 5. Initial evaluations of the key performance trends are discussed in chapter 6. Finally, chapter 7 provides a summary, conclusions, and recommendations.

Appendix A presents a summary of the SPS-2 project nomination and construction guidelines. SPS-2 project construction and deviation reports are summarized in appendix B.

2. EVALUATION OF THE SPS-2 EXPERIMENT

The first step in the evaluation of the SPS-2 experiment is to assess how much of the original experiment was actually constructed and what effect missing sites will have on obtaining expected findings. The original SPS-2 experiment design, the SPS-2 experimental sites actually constructed, the effects of blank experimental design cells, and potential information available from the SPS-2 supplemental sites are discussed in this chapter.

ORIGINAL SPS-2 EXPERIMENT DESIGN

The complete experiment design factorial, the site classification category, and individual section identification numbers are given in table 1. This design factorial requires a total of 16 SPS-2 sites and 192 core sections be evenly distributed within the factorial matrix of climate and subgrade to satisfy the requirement of the SPS-2 experiment design.

The experiment design for SPS-2 included eight main factors. Three are project site factors, and the remaining five are related to the pavement structure.

Project Site Factors

Three factors describe site conditions at the test site:

1. Climatic—Temperature - Freeze and no-freeze
2. Climatic—Precipitation - Wet and dry
3. Subgrade - Fine-grained and coarse-grained

Traffic loading was incorporated as a covariant. Each of the SPS-2 sites required a relatively high rate of traffic of at least 200,000 rigid equivalent single-axle loads (ESALs) per year.

The experiment design factorial at the project site level is provided in table 2. The design factorial calls for a total of 16 SPS-2 sites to be constructed throughout the United States.

Pavement Structure Design Factors

In addition to the three site factors, five experiment design factors were allocated to the pavement structure. Two factors were allocated to the surface layer: one for thickness and one for strength. Two other structural factors were allocated to the base/subbase: one for strength/stability and the other for drainage. The last factor was assigned to the width of the pavement slab. In summary, the SPS-2 experiment incorporates the following pavement structural factors:

1. Slab thickness - 203 and 279 millimeters (mm)
2. Concrete flexural strength - 3.8 and 6.2 megapascals (MPa) at 14-days
3. Base type and drainage - Dense-graded aggregate base (DGAB)
- Lean concrete base (LCB)
- Permeable asphalt-treated base (PATB) with edge drains
4. Slab width - 3.66 and 4.27 meters (m)

Table 1. Original SPS-2 experiment design.

Pavement Structure					SPS-2 Site Designation															
					J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y
Edge Drain	Base Type	PCC		Lane Width, m	Climate Zones, Subgrade															
		Thick mm	Strength MPa		Wet								Dry							
					Freeze				No-Freeze				Freeze				No-Freeze			
					Fine		Coarse		Fine		Coarse		Fine		Coarse		Fine		Coarse	
No	AGG	203	3.8	3.66	**01**		**01**		**01**		**01**		**01**		**01**		**01**		**01**	
				4.27		**13**		**13**		**13**		**13**		**13**		**13**		**13**		**13**
			6.2	3.66		**14**		**14**		**14**		**14**		**14**		**14**		**14**		**14**
				4.27	**02**		**02**		**02**		**02**		**02**		**02**		**02**		**02**	
		279	3.8	3.66		**15**		**15**		**15**		**15**		**15**		**15**		**15**		**15**
				4.27	**03**		**03**		**03**		**03**		**03**		**03**		**03**		**03**	
			6.2	3.66	**04**		**04**		**04**		**04**		**04**		**04**		**04**		**04**	
				4.27		**16**		**16**		**16**		**16**		**16**		**16**		**16**		**16**
No	LCB	203	3.8	3.66	**05**		**05**		**05**		**05**		**05**		**05**		**05**		**05**	
				4.27		**17**		**17**		**17**		**17**		**17**		**17**		**17**		**17**
			6.2	3.66		**18**		**18**		**18**		**18**		**18**		**18**		**18**		**18**
				4.27	**06**		**06**		**06**		**06**		**06**		**06**		**06**		**06**	
		279	3.8	3.66		**19**		**19**		**19**		**19**		**19**		**19**		**19**		**19**
				4.27	**07**		**07**		**07**		**07**		**07**		**07**		**07**		**07**	
			6.2	3.66	**08**		**08**		**08**		**08**		**08**		**08**		**08**		**08**	
				4.27		**20**		**20**		**20**		**20**		**20**		**20**		**20**		**20**
Yes	PATB	203	3.8	3.66	**09**		**09**		**09**		**09**		**09**		**09**		**09**		**09**	
				4.27		**21**		**21**		**21**		**21**		**21**		**21**		**21**		**21**
			6.2	3.66		**22**		**22**		**22**		**22**		**22**		**22**		**22**		**22**
				4.27	**10**		**10**		**10**		**10**		**10**		**10**		**10**		**10**	
		279	3.8	3.66		**23**		**23**		**23**		**23**		**23**		**23**		**23**		**23**
				4.27	**11**		**11**		**11**		**11**		**11**		**11**		**11**		**11**	
			6.2	3.66	**12**		**12**		**12**		**12**		**12**		**12**		**12**		**12**	
				4.27		**24**		**24**		**24**		**24**		**24**		**24**		**24**		**24**

Notes: Test section numbers included in cells from 01 to 24 represent a full factorial experiment.
AGG = Dense-graded untreated aggregate base.
LCB = Lean concrete base.
PATB = Permeable asphalt-treated base.

Table 2. SPS-2 experiment design—project site factorial.

Subgrade	Climatic Condition			
	Wet		Dry	
	Freeze	No-Freeze	Freeze	No-Freeze
Fine	J	N	R	V
	K	O	S	W
Coarse	L	P	T	X
	M	Q	U	Y

Note: Two related sites (e.g., J and K, T and U) make up a full factorial of design features.

The total full factorial designs for combination of the study factors result in 24 different experimental pavement structures (2 thickness x 2 strength x 3 base type x 2 lane width = 24). The experiment design was implemented using a one-half fractional factorial approach that permitted construction of 12 test sections at one site and construction of the 12 complementary sections at another site within the same climatic region and with a similar subgrade type. Both of these complementary sites must be available to evaluate the main effects and all interactions of the design factors within a given climate and subgrade type.

Each SPS-2 test section length is 152 m long. As a result, an SPS-2 project may be constructed over a length greater than 3.2 kilometers (km). Other key design features common to all SPS-2 sections include the following:

- Joint spacing—4.6 m uniform spacing.
- Joint load transfer—Doweled perpendicular transverse joints, with 32-mm dowel bars for the 203-mm-thick pavements and 38-mm dowel bars for 279-mm-thick pavements. Dowels are to be epoxy coated, 457 mm long, and spaced at 305 mm.
- Longitudinal joints—Between lanes the joints should be sawcut, preferably using up to an 8-mm-wide blade, to a depth of D/3 (where D equals slab thickness). The sealant reservoir may be formed later using a second sawcut to provide for an 8-mm-wide by 25-mm-deep cut.
- Joint sealing—All joints shall be sealed before opening to traffic. Joint sealing shall be accomplished using only silicone sealant.
- Shoulder—Either AC or portland cement concrete (PCC) shoulders. PCC shoulders shall not be tied to the mainline pavement. If the concrete shoulder is placed monolithically with the traffic lanes, then the shoulder joint shall be sawed full depth.

At each SPS-2 site, test sections are numbered sequentially either from 01 to 12 or from 13 to 24, depending on climatic and soil type combinations. The numbering within each site depends on the levels of each design factor. Figure 1 graphically presents the section structure and numbering schematic for SPS-2 sections.

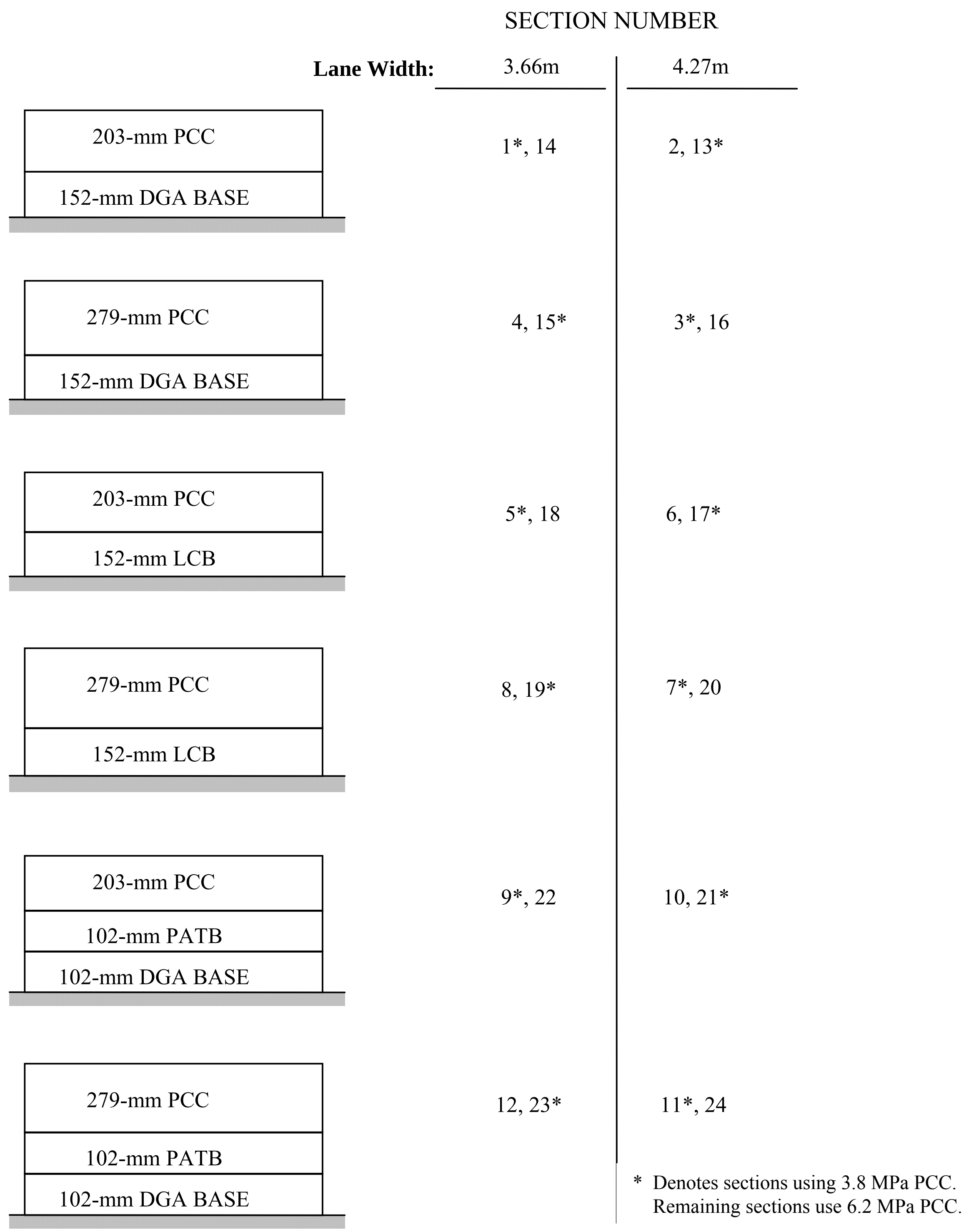


Figure 1. Test section details for a full factorial SPS-2 experiment located at two sites (01 to 12 and 13 to 24).

CURRENT STATUS OF DESIGN FACTORIAL

As of August 1999, 13 SPS-2 sites had been constructed throughout the United States An additional site in California had been nominated and may be constructed in the near future. The distribution of the SPS-2 sites is shown on a map in figure 2. The current status of the site design factorial is provided in table 3, showing which State is filling which design cell. As shown in the table, there are five study sites missing from the experiment factorial, indicating a loss of 5 of the 16 (or 31 percent) of the design cells.

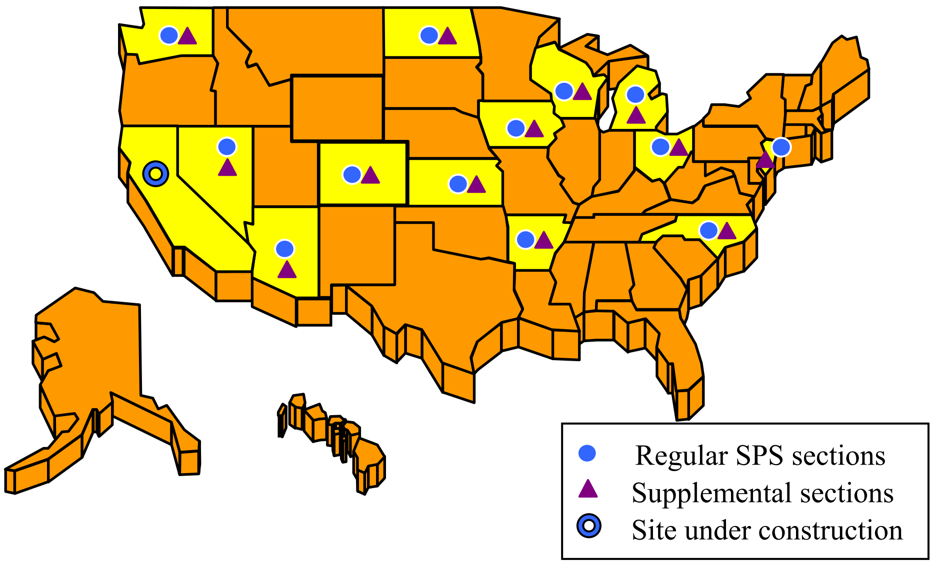


Figure 2. Geographic distribution of the constructed SPS-2 projects.

Table 3. SPS-2 projects constructed in relation to the project site factorial (note missing cells).

Subgrade	Climatic Condition			
	Wet		Dry	
	Freeze	No-Freeze	Freeze	No-Freeze
Fine	Ohio, Kansas[1] (J)	North Carolina (N)	**?** (R)	**?** (V)
	Michigan, Iowa (K)	Arkansas (O)	North Dakota[2] (S)	**?** (W)
Coarse	Delaware (L)	**?** (P)	Nevada, Washington (T)	California[3] (X)
	Wisconsin (M)	**?** (Q)	Colorado (U)	Arizona (Y)

Notes: 1 – Kansas site was planned to fill the dry-freeze zone cell. However, the actual precipitation is quite a bit higher than the specified annual precipitation for this study.
2 – North Dakota site is slightly wetter than the specified annual precipitation.
3 – California site is currently under construction.
? – Represents a missing SPS-2 project.

A list of all the SPS-2 sections, including both core and supplemental sections, is given in table 4. As shown, many SHAs have constructed additional sections at the SPS-2 sites. These additional sections are not included in the original SPS-2 experiment factorial and are referred to as State supplemental sections. A total of 155 core sections and 40 supplemental sections have been constructed to date. Seven SPS-2 sections are designated as seasonal monitoring program (SMP) sections, where additional sets of climatic and monitoring data are measured.

Table 4. List of constructed SPS-2 core and supplemental sections.

State	State Code	Core Sections		Supplemental Sections	Seasonal Sections
		ID	Record Status		
AZ	04	0213-0224	E	0260-0268 (9)	0215
AR	05	0213-0224	E	–	–
CO	08	0213-0224	E	0259	–
DE	10	0201-0212	E	0259-0260 (2)	–
IA	19	0213-0224	E	0259	–
KS	20	0201-0212	E	0259	–
MI	26	0213-0224	E	0259	–
NV	32	0201-0211 (0212 was removed)	E	0259	0204
NC	37	0201-0212	E	0259-0260 (2)	0201, 0205, 0208, 0212
ND	38	0213-0224	E	0259-0264 (6)	–
OH	39	0201-0212	E	0259-0265 (7)	0204
WA	53	0201-0212	E	0259	–
WI	55	0213-0224	A	0259-0266 (8)	–
Total number of sections		**155**		**40**	**7**

POTENTIAL EFFECTS OF MISSING EXPERIMENTAL SITES

The current status of the SPS-2 experiment design, showing constructed sections in relation to the original experiment design, is given in table 5. The following five sites (columns) are missing from the original experiment design:

- Two sites in the wet no-freeze climate (southeast U.S.) with a coarse-grained subgrade. Each project fills half of the design factorial.
- One site in the dry freeze climate (northwest U.S.) with a fine-grained subgrade.
- Two sites in the dry no-freeze climate (southwest U.S.), one with a fine-grained subgrade and the other with a coarse-grained subgrade. Each site fills half of the design factorial.

Table 5. Current status of SPS-2 experiment.

Pavement Structure				Climate Zones, Subgrade, Site															
				Wet								Dry							
Base Type/ Edge Drain	PCC		Lane Width m	Freeze				No-Freeze				Freeze				No-Freeze			
	Thick mm	Flexural Strength MPa		Fine		Coarse		Fine		Coarse		Fine		Coarse		Fine		Coarse	
				J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y
AGG	203	3.8	3.66	OH, KS		DE		NC						NV, WA				CA[1]	
			4.27		MI, IA		WI		AR				ND		CO				AZ
		6.2	3.66		MI, IA		WI		AR				ND		CO				AZ
			4.27	OH, KS		DE		NC						NV, WA				CA[1]	
	279	3.8	3.66		MI, IA		WI		AR				ND		CO				AZ
			4.27	OH, KS		DE		NC						NV, WA				CA[1]	
		6.2	3.66	OH, KS		DE		NC						NV, WA				CA[1]	
			4.27		MI, IA		WI		AR				ND		CO				AZ
LCB	203	3.8	3.66	OH, KS		DE		NC						NV, WA				CA[1]	
			4.27		MI, IA		WI		AR				ND		CO				AZ
		6.2	3.66		MI, IA		WI		AR				ND		CO				AZ
			4.27	OH, KS		DE		NC						NV, WA				CA[1]	
	279	3.8	3.66		MI, IA		WI		AR				ND		CO				AZ
			4.27	OH, KS		DE		NC						NV, WA				CA[1]	
		6.2	3.66	OH, KS		DE		NC						NV, WA				CA[1]	
			4.27		MI, IA		WI		AR				ND		CO				AZ
PATB w/ Drain	203	3.8	3.66	OH, KS		DE		NC						NV, WA				CA[1]	
			4.27		MI, IA		WI		AR				ND		CO				AZ
		6.2	3.66		MI, IA		WI		AR				ND		CO				AZ
			4.27	OH, KS		DE		NC						NV, WA				CA[1]	
	279	3.8	3.66		MI, IA		WI		AR				ND		CO				AZ
			4.27	OH, KS		DE		NC						NV, WA				CA[1]	
		6.2	3.66	OH, KS		DE		NC						WA				CA[1]	
			4.27		MI, IA		WI		AR				ND		CO				AZ

***Notes*:** 1 – California site is under construction.

Five additional projects (60 sections total) are needed to complete the design factorial. These missing projects will definitely limit the results obtainable from the SPS-2 experiment, although it is impossible to determine the exact effects at this time. These limitations are summarized as follows:

- There will be no performance data, and thus, no performance findings from the missing sites.
- The missing section at the Nevada site will reduce the findings for that site and corresponding cell, although there appears to be a replication at the Washington site.
- SPS-2 sites exist in wet-freeze climates, making a full inference space of performance data available. All main effects and interactions should be ascertainable in this climate.
- SPS-2 sites are deficient in wet and dry no-freeze climates. There will be difficulties in determining the main effects and interactions in these climates.

Some of these deficiencies can be overcome through use of mechanistic analysis of the data. However, there is no mechanistic analysis that considers all factors involved, and the missing cells will always present limitations in the verification and calibration of any performance models.

STATE SUPPLEMENTAL SECTIONS

In addition to the 12 core sections required by the SPS-2 experiment, SHAs can include additional experimental sections, referred to as State supplemental sections. Although there were provisions in the experimental design to study undoweled JPCP and jointed reinforced concrete pavements (JRCP), no such controlled sections have been constructed other than some uncontrolled supplemental sections. Table 6 lists the design variables selected by SHAs for supplemental sections.

The main value of the supplemental sections will be as a direct comparison to the core sections. For example, one supplemental section in Washington did not have dowels; this will provide a direct comparison to a similar design with dowel bars. This comparison is also possible in Arizona and North Dakota. Various other comparisons are also possible, including skewed joints, base types, subdrainage, slab thickness, AC pavement, jointed reinforced concrete, special dowels, and variable slab thickness.

Table 6. List of the constructed SPS-2 State supplemental sections and designs.

State	SHRP ID	Pavement Design Description
AZ	0260	216 mm dense-graded AC on 102 mm DGAB.
	0261	216 mm dense-graded AC on 102 mm DGAB.
	0262	203 mm undoweled JPC (3.8 MPa Resilient Modulus (MR)) on DGAB and 4.27 m lane.
	0263	203 mm undoweled JPC (3.8 MPa MR) on PATB and DGAB and 4.27 m lane.
	0264	279 mm undoweled JPC (3.8 MPa MR) on PATB and DGAB and 3.66 m lane.
	0265	279 mm undoweled JPC (3.8 MPa MR) on DGAB and 3.66 m lane.
	0266	318 mm doweled JPC (3.8 MPa MR) on bituminous treated base (BTB) and 4.27 m lane.
	0267	279 mm doweled JPC (3.8 MPa MR) on BTB and 4.27 m lane.
	0268	203 mm doweled JPC (3.8 MPa MR) on BTB and 4.27 m lane.
CO	0259	279 mm JPC (4.5 MPa) on subgrade and 3.66 m lanes.
DE	0259	254 mm JPC (20.7 MPa f'c) on 203 mm DGAB; 3.66 m lane; steel dowels.
	0260	254 mm JPC (20.7 MPa f'c) on 203 mm DGAB; 3.66 m lane; plastic dowels.
IA	0259	279 mm JPC; 4.27 m wide lane.
KS	0259	305 mm doweled JPC (4.1-MPa mix) on 152 mm stabilized subbase on 152 mm modified flyash subgrade and 3.66 m lane.
MI	0259	267 mm JRC on 102 mm open-graded base course (OGBC) on 76 mm aggregate base.
NV	0259	267 mm JPC on 38 mm leveling course, 27.6 MPa +- 20% 14-day compressive strength.
NC	0259	254 mm JPC on 102 mm PATB on 25.4 mm AC on 203-mm lime-stabilized subgrade.
	0260	279 mm JPC on 25.4 mm AC on 127 mm BTB on 203-mm cement-treated subgrade.
ND	0259	254 mm doweled JPC (ND mix) on 203 mm salve with skewed joints and 3.66 m lanes.
	0260	279 mm doweled JPC (ND mix) on DGAB with skewed joints and 4.27 m lanes.
	0261	279 mm undoweled JPC (3.8 MPa MR) on DGAB with skewed joints and 3.66 m lanes.
	0262	279 mm undoweled JPC on LCB with skewed joints (various lengths) and 4.27 m lanes.
	0263	279 mm undoweled JPC on PATB with random skewed joints and 3.66 m lanes.
	0264	279 mm undoweled JPC on PATB with skewed joints and 4.27 m lanes.
OH	0259	279 mm JPC (3.8 MPa MR) on 152 mm DGAB.
	0260	279 mm JPC (3.8 MPa MR) on 102 mm PATB on 102 mm DGAB.
	0261	279 mm JPC (3.8 MPa MR) on 102 mm CTPB on 102 mm DGAB.
	0262	279 mm JPC on 102 mm CTPB on 102 mm DGAB.
	0263	279 mm JPC on 152 mm DGAB.
	0264	279 mm JPC on 152 mm DGAB.
	0265	279 mm JPC (3.8 MPa MR) on 102 mm PATB on 102 mm DGAB.
WA	0259	Undoweled 254 mm JPC (4.5 MPa MR) on 76 mm ATB on 51 mm crushed surfacing base course; 4.27 m lane.
WI	0259	279 mm JPC (3.8 MPa MR) on 152 mm DGAB.
	0260	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with alternate dowel bar placement.
	0261	203 mm JPC (3.8 MPa MR) on 102 mm OGBC on 102 mm DGAB.
	0262	203 mm JPC (6.3 MPa MR) on 152 mm DGAB, with tied concrete shoulder.
	0263	203-279 mm JPC (3.8 MPa MR) on 152 mm DGAB, variable pavement thickness.
	0264	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with composite dowels.
	0265	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with stainless steel dowels.
	0266	Unknown

Note: The Arkansas SPS-2 project site does not contain any supplemental sections.

3. ASSESSMENT OF DATA AVAILABILITY AND COMPLETENESS

The second step in the SPS-2 review and evaluation study is to assess the key data availability and completeness. LTPP data availability and quality control (QC) checks are discussed first. Then, key data elements are assessed for their quality level and completeness. The data reviews are divided into the following categories:

- General site information.
- Pavement structure data.
- Construction data.
- Material testing data.
- Traffic data.
- Climate data.
- Monitoring data.
- Dynamic load-response data.

IMS data release 9.8, obtained on August 10, 1999, was used for the majority of the study; however, the distress, profile, and materials testing data are from IMS release 10.1, obtained on February 1, 2000.

LTPP DATA AVAILABILITY AND QUALITY CONTROL CHECKS

The quality of the data is the most important factor in any type of analysis. From the outset of the LTPP program, data quality has been considered of paramount importance. Procedures for collecting and processing data were defined (and are modified as necessary) to ensure consistency across various reporting contractors, laboratories, equipment operators, and so forth. Although these procedures formed the foundation of quality control/quality assurance (QC/QA) and data integrity, many more components of a QC/QA plan were necessary to ensure that the data sent to researchers were as error-free as practical.

LTPP has developed and implemented an extensive QC program that classifies each of the data elements into categories depending upon the location of the data in this QC process. Several components or steps comprise the overall QC/QA plan used on LTPP data, as are discussed in the following paragraphs.

1. Collect Data: Procedures for collecting data are documented for each module in the IMS. These procedures are intended to ensure that data are collected in similar format, amounts, conditions, etc. Documentation references include the *Data Collection Guide for Long-Term Pavement Performance Studies*(2) and various module-specific guides.

2. Review Data: Regional engineers review essentially all data input into regional IMS (RIMS) to check for possible errors related to keystroke input, field operations, procedures, equipment operations, and other variables. The regional review is intended to catch obvious data collection errors. In addition, some data are preprocessed before they are entered into the IMS. For example, PROFCAL™ software is used on SHRP profilometers to provide a system check by comparing measurements taken at different speeds. PROFSCAN™ is a

field QA tool that allows an operator to identify invalid data while still in the field, thus avoiding costly revisits to the site.

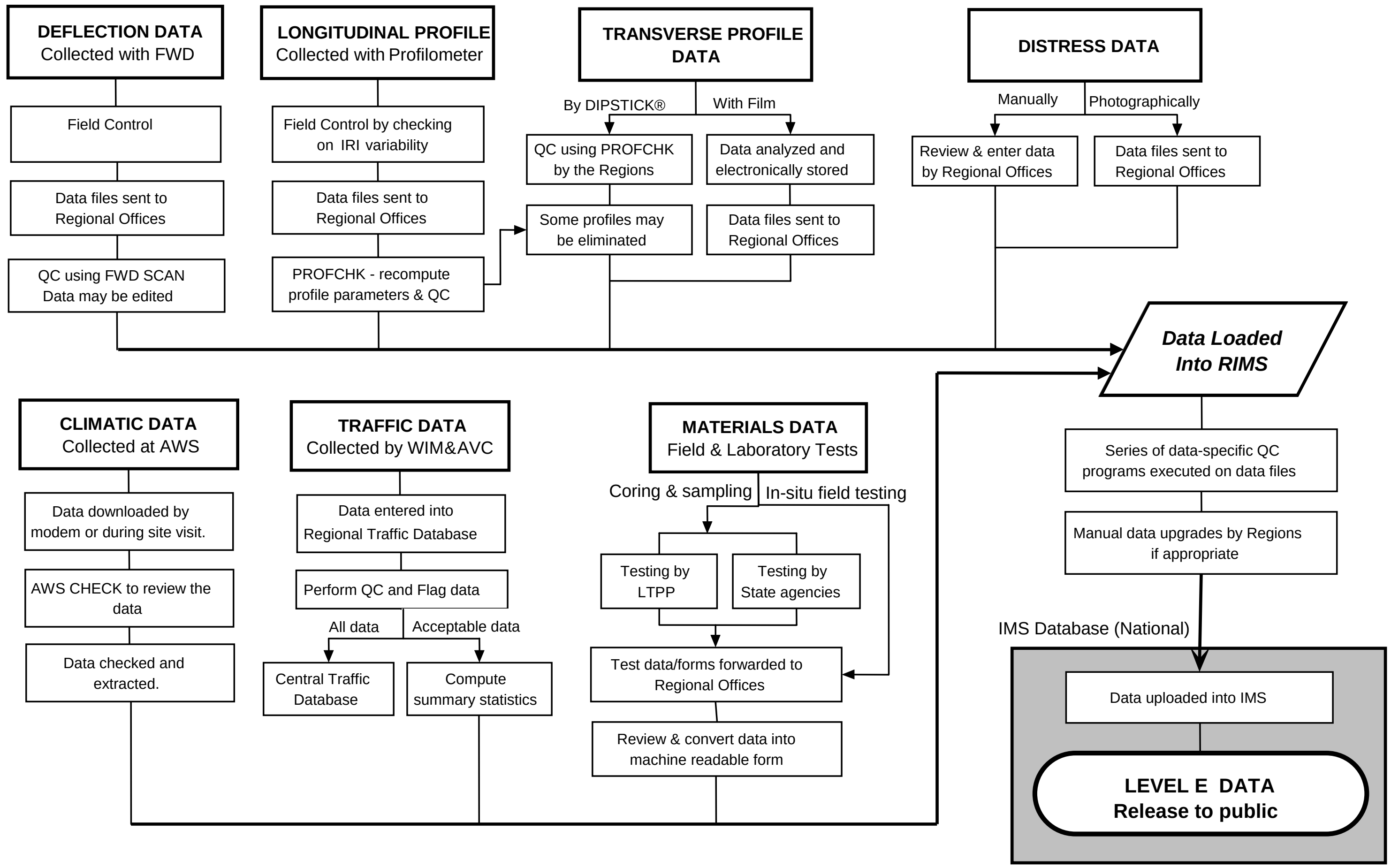
3. Load Data in IMS: Some checks are programmed into the IMS to identify errors as data are entered. The IMS contains mandatory logic, range, data verification, and other miscellaneous checks that are invoked during input.

4. QC/QA: Once data are input into the IMS and reviewed by regional engineers, formal QC/QA software programs are run on the data.

    - Level A—Random checks of data are performed to ensure correct RIMS to IMS data transfer.
    - Level B—A set of dependency checks is performed to ensure that essential section information has been recorded in the IMS. In addition, experiment types are verified based on inventory data. These checks are currently being incorporated into the level E checks for all modules.
    - Level C—A minimum data search is performed for critical elements. For example, inventory data should contain the coordinates of the section, friction data should contain the skid number, and rehabilitation data should have a code entered to identify each work type activity.
    - Level D—Expanded range checks are applied to certain fields to identify data element values that fall outside an expected range. These checks are more stringent than the input range checks reviewed by the regional engineers.
    - Level E—Intramodular checks are employed to verify the consistency of data within a data module. For example, if an overlay is identified in the inventory layer structure, the data of the overlay should be recorded in the inventory table that includes major improvements to the pavement structure.

Once the QC/QA programs are completed, the regional engineers review the output and resolve any data errors whenever possible. Often the data entered are accurate and legitimate, but do not pass a QC/QA check. When this occurs, the regional engineer can document that the data have been confirmed using a comments table in the IMS and manually upgrade the record to Level E.

Figure 3 is a flowchart that shows the movement of data elements and quality checks completed on the data prior to release to the public. Only a fraction of the data fields are checked. A value of "A" is automatically assigned to a record on entry in the database. A value of "B" indicates that the QC process was executed and a level C check was failed. Any record for which correct section information is stored in the database is available after the QC is completed. A record of the QC processing is included with the record. Since the checks are run in sequence from A to E, the last successful check is identified on the record as the record status variable. A value of B or C does not necessarily indicate that higher level QC was unsuccessful, merely that a necessary data element was not available when the QC was done.



FWD = falling-weight deflectometer
IRI = International Roughness Index
AWS = automated weather station
WIM = weigh-in-motion
AVC = automated vehicle classification

Figure 3. LTPP data collection and data movement flowchart.

There are numerous reasons why some important data may not be available from publicly released IMS database at the time of analysis. The following are some possible examples:

- Data are yet to be collected or the laboratory tests have not been performed yet.
- Data are under regional review.
- Data have failed one of the quality checks and are to be reviewed.
- Data have failed one of the quality checks and were identified as anomalies.
- Data are yet to be quality checked.

As such, the unavailable data identified in this report do not necessarily mean that the data were not collected or submitted by the States. There are several places where data may get held up and not reach level E. Note that the results reported in this report are based upon level E data only.

The LTPP program is embarking on a systemwide effort to resolve all unavailable data so that information will be available to future researchers. Some data have already been located during the course of this study.

GENERAL SITE INFORMATION

General site-related information availability for SPS-2 projects is discussed in this section. This includes site identification and location, key equipment installed, report availability, and important dates associated with each SPS-2 site. The information was obtained from the site construction reports and deviation reports, or from the following IMS tables:

- EXPERIMENT_SECTION
- SPS_ID

The EXPERIMENT_SECTION table contains records for all the SPS-2 sites and sections. All the site-level records (0200) for the 13 constructed SPS-2 projects are at level E. The section-level records are at level E except for the 12 sections at the newly constructed Wisconsin SPS-2 site. The SPS_ID tables contain records for all 13 SPS-2 sites, and the site data are all at level E.

Since this site-level information is fundamental to the SPS-2 sites and is very important for an overall understanding of the sites, actual key data are presented, in addition to the data availability assessment. General State identification, equipment installation, and report availability information about the SPS-2 sites are provided in table 7. The construction reports were prepared and submitted by LTPP regional coordination office contractors (RCOCs) for all SPS-2 projects.

The site location and functional class information are provided in table 8. Table 9 presents the significant dates such as the approximate construction complete date, traffic opening date, and the LTPP assign and deassign dates. The oldest SPS-2 site is 7.5 years old. As indicated in both tables, all the important site-level information is available for the 13 SPS-2 sites. The only exception is that the approximate construction completion date for the North Dakota site was not available at the time of analysis in the SPS_ID table.

Table 7. SPS-2 site general information and report availability.

State Information				Equipment Installed			Report Availability	
Abbr.	Code	Name	SHRP Region	AWS	WIM	AVC	Construction	Deviation
AZ	04	Arizona	W	✓	✓	✓	✓	–
AR	05	Arkansas	S	✓	✓	–	✓	✓
CO	08	Colorado	W	✓	✓	✓	✓	–
DE	10	Delaware	NA	✓	✓	✓	✓	✓
IA	19	Iowa	NC	✓	✓	✓	✓	✓
KS	20	Kansas	NC	✓	✓	✓	✓	✓
MI	26	Michigan	NC	✓	✓	✓	✓	✓
NV	32	Nevada	W	✓	✓	✓	✓	–
NC	37	North Carolina	NA	✓	✓	✓	✓	–
ND	38	North Dakota	NC	✓	✓	✓	✓	✓
OH	39	Ohio	NC	✓	✓	✓	✓	✓
WA	53	Washington	W	✓	✓	✓	✓	–
WI	55	Wisconsin	NC	✓	✓	✓	✓	✓

Notes:

NA = North Atlantic Region
NC = North Central Region
S = Southern Region
W = Western Region

Table 8. SPS-2 site location information.

State	County	Route No.	Functional Class	Lanes
AZ	Maricopa	Interstate 10	Rural principal arterial—interstate	2
AR	Hot Springs	Interstate 30	Rural principal arterial	2
CO	Adams	Interstate 76	Rural principal arterial	2
DE	Sussex	US 113	Rural principal arterial—other	2
IA	Polk	US 65	Urban principal arterial—other freeways or expressways	2
KS	Dickinson	Interstate 70	Rural principal arterial—interstate	2
MI	Monroe	US 23	Rural principal arterial—other	2
NV	Lander	Interstate 80	Rural principal arterial	2
NC	Davidson	US 52	Rural principal arterial—other	2
ND	Cass	Interstate 94	Rural principal arterial—interstate	2
OH	Delaware	US 23	Rural principal arterial—other	2
WA	Adams	State 395	Urban principal arterial—other freeways or expressways	2
WI	Marathon	State 29	Rural other principal arterial	2

Table 9. SPS-2 sites significant dates and age as of August 1999.

State Abbr.	Age as of August 1999 (years)	Significant Dates			
		Date Completed	Data Open to Traffic	Assign Date	Deassign Date
AZ	5.8	10/01/93	10/01/93	01/01/93	–
AR	3.8	10/01/95	11/01/95	09/01/93	–
CO	5.8	10/01/93	11/01/93	01/01/93	–
DE	3.3	05/01/96	05/01/96	01/01/92	–
IA	5.0	08/01/94	12/01/94	01/01/92	–
KS	7.1	07/01/92	08/01/92	01/01/92	–
MI	5.8	11/01/93	11/01/93	01/01/93	–
NV	4.0	08/01/95	09/01/95	01/01/93	09/18/97 (0202, 0206)
NC	5.1	07/01/94	07/01/94	07/15/92	–
ND	4.8	11/01/94	11/01/94	01/01/94	–
OH	2.9	09/01/96	10/01/96	01/01/94	–
WA	3.8	11/01/95	11/01/95	01/01/93	–
WI	1.8	10/01/97	11/01/97	01/01/97	–

PAVEMENT STRUCTURE DATA

Pavement structure data are further divided into two categories: pavement layer data and pavement design features.

Pavement Layer Data

Pavement layer data for SPS-2 sections are available from two sources: rod and level measurements (IMS table SPS2_Layer) and core measurements (IMS table TST_L05B). Both tables were examined for the following pavement structure layers:

- PCC slab thickness.
- Base type and thickness.
- Subgrade type.

The data availability and QC levels for these data elements are summarized in table 10.

The TST_L05B table contains records with all layer data for 143 core sections at 12 SPS-2 sites. Layer information from the Wisconsin SPS-2 site is not available from the database at the time of analysis.

The SPS2_LAYER table contains all layer data for all 155 sections from all 13 SPS-2 sites, and 143 sections are at QC level E. The remaining 12 records, all from the same site in Wisconsin, are at level A.

Table 10. Data availability and QC levels for key pavement layer data.

Data Availability	TST_L05B			SPS2_LAYER		
	Slab Thickness	Base Layer	Subgrade	Slab Thickness	Base Layer	Subgrade
Core Sections (Total 155 sections)						
At all levels (A to E)	143	143	143	155	155	155
At level E only	127	108	126	143	143	143
Percent data at level E	89	82	88	92	92	92
Core sections missing data at all levels	12	12	12	0	0	0
Sites with missing data at all levels	WI	WI	WI	–	–	–
Supplemental Sections (Total 40 Sections)						
Supplemental sections with data	30	26	30	34	33	28
Supplemental sections missing data	10	14	10	6	7	12

Key Design Feature Data

Important general design features, such as drainage, lane width, and shoulder type data, are included in table SPS_GENERAL. The data availability assessment for these data elements is provided in table 11.

Table 11. Data availability for lane width, drainage, and shoulder data for SPS-2 sections.

Data Availability Number	Lane Width Data	Drainage and Shoulder Type Data
Core Sections (Total 155 sections)		
At all levels (A to E)	131	131
At level E only	131	131
Percent data at level E	100	100
Core sections missing data at all levels	24	24
Sites with missing data at all levels	KS, WI	KS, WI
Supplemental Sections (Total 40 Sections)		
Supplemental sections with data	29	27
Supplemental sections missing data	11	13

As indicated in table 11, information is available for 133 SPS-2 sections, and the data are all at level E. The key design feature data for 24 SPS-2 core sections in Kansas and Wisconsin were not available at the time of analysis.

CONSTRUCTION DATA

SPS-2 construction data include information pertaining to the pavement layers constructed according to the requirements stipulated for the experiment. The following key SPS-2 construction tables were evaluated for the data completeness and QC levels:

- SPS2_PCC_JOINT_DATA—PCC layers: Joint data (sheets 15, 16).
- SPS2_PCC_MIXTURE_DATA—PCC layers: Mixture data (sheets 18, 19).
- SPS2_PCC_PLACEMENT_DATA—PCC layers: Placement data. (sheets 20, 21).
- SPS2_PROJECT_STATIONS—Test section information (sheet 3).
- SPS2_SUBGRADE_PREP—Subgrade preparation (sheet 6).
- SPS2_UNBOUND_AGG_BASE—Unbound aggregate base material placement data (Sheet 9).

Data availability assessment and QC levels summary for these tables are provided in tables 12 and 13.

Table 12. Data availability assessment and QC levels for SPS-2 key construction data.

Data Availability Number	SPS2_PCC_ JOINT_DATA	SPS2_PCC_ MIXTURE_DATA	SPS2_PCC_ PLACEMENT_DATA
	Core Sections (Total 155 sections)		
At all levels (A to E)	151 sections (157 records)	139 sections (175 records)	155 sections (194 records)
At level E only	136 sections (142 records)	133 sections (169 records)	142 sections (181 records)
Percent data at level E	90	97	93
Core sections missing data at all levels	4	16	0
Sites with missing data at all levels	WA	AZ, OH	-
Supplemental Sections (Total 40 Sections)			
Supplemental sections with data	35	21	35
Supplemental sections missing data	5	19	5

Table 13. Data availability assessment and QC levels for other SPS-2 construction data.

Data Availability Number	SPS2_PROJECT_STATIONS	SPS2_SUBGRADE_PREP	SPS2_UNBOUND_AGG_BASE
Core Sections (Total 155 sections)			
At all levels (A to E)	143 sections	153 sections	92 sections (194 records)
At level E only	143 sections	141 sections	92 sections (181 records)
Percent data at level E	100	92	100
Core sections missing data at all levels	12	12	12 (only 8 sections at each site)
Sites with missing data at all levels	WI	WI	WI
Supplemental Sections (Total 40 Sections)			
Supplemental sections with data	32	34	20
Supplemental sections missing data	8	6	20

Over 90 percent of the existing pavement structure data are at level E, but some data for several SPS-2 core sections were not available at the time of analysis. However, the supplemental sections were missing a lot of data at the time of analysis, as shown in these tables. The Wisconsin SPS-2 site represents most of the missing data, as it was a new site at the time of analysis.

MATERIAL TESTING DATA

Field and laboratory tests are conducted to establish material properties and characteristics for LTPP sections. Characterization of material properties and the variations in these properties between and within the test sections is required to evaluate causes of performance differences between test sections. The materials characterization includes parameters used in current pavement design and mechanical analysis models.

Material sampling and testing requirements are documented in the *SPS-2 Material Sampling and Testing Requirements* report.[(3)] This report includes the development of SPS-2 sampling and testing plans, field material sampling and testing requirements, and laboratory material testing requirements for each SPS-2 site. The SPS-2 material sampling and testing plans for subgrade and bases are provided in table 14, while the material sampling and testing plans for PCC surface are presented in table 15.

The sampling and testing plan specified methods for material sampling and testing at the site level for similar material and structure layers. Therefore, the evaluation of the material testing data should also be conducted at the SPS-2 site level.

Since there is a comprehensive LTPP material data review study underway, only the key data elements for the PCC surface from the SPS-2 sampling and testing plan were evaluated in this study.

Table 14. SPS-2 materials sampling and testing plan for subgrade and bases.

Material Type and Properties	LTPP Designation	LTPP Protocol	Minimum No. of Tests per Layer
SUBGRADE OR EMBANKMENT			
Sieve analysis	SS01	P51	6
Hydrometer to 0.001 mm	SS02	P42	6
Atterberg limits	SS03	P43	6
Classification	SS04	P52	6
(visual manual only on thin-wall tubes)			18
Moisture-density relations	SS05	P55	6
Resilient modulus	SS07	P46	6
(if thin-wall tube is not available)			6
Unit weight (if thin-wall tube is not available, test is not conducted)	SS08	P56	6
Natural moisture content	SS09	P49	6
Unconfined comp. strength (if thin-wall tube is not available, test is not conducted)	SS10	P54	6
Permeability	SS11	P57	3
Permeability	UG09	P48	6
UNBOUND GRANULAR BASE			
Particle size analysis	UG01	P41	3
Sieve analysis (washed)	UG02	P41	3
Atterberg limits	UG04	P43	3
Moisture-density relations	UG05	P44	3
Resilient modulus	UG07	P46	3
Classification	UG08	P47	3
Permeability	UG09	P48	3
Natural moisture content	UG10	P49	3
PERMEABLE-TREATED ASPHALT BASE			
Asphalt content (extraction)	AC04	P04	3
Extracted aggregate: Gradation of aggregate	AC04	P14	3
LEAN CONCRETE BASE			
Compressive strength	PC01	P61 (beams, cores)	
7 day			14 (6, 8)
28 day			14 (6, 8)
1 year			14 (6, 8)
Core examination and thickness	PC06	P66	24 (all cores)

Table 15. SPS-2 materials sampling and testing plan for the PCC surface.

PCC Properties	LTPP Designation	LTPP Protocol	Minimum No. of Tests per Layer
Compressive strength			
14 day 3.8 MPa	PC01	P61	9 (3, 6)
14 day 6.2 MPa		(beam, cores)	9 (3, 6)
28 day 3.8 MPa			9 (3, 6)
28 day 6.2 MPa			9 (3, 6)
1 year 3.8 MPa			9 (3, 6)
1 year 6.2 MPa			9 (3, 6)
Splitting tensile strength			
14 day 3.8 MPa	PC02	P62	9 (3, 6)
14 day 6.2 MPa		(beam, cores)	9 (3, 6)
28 day 3.8 MPa			9 (3, 6)
28 day 6.2 MPa			9 (3, 6)
1 year 3.8 MPa			9 (3, 6)
1 year 6.2 MPa			9 (3, 6)
Coefficient of thermal expansion	PC03	P63	2
Static modulus of elasticity			
28 day 3.8 MPa	PC04	P64	6
28 day 6.2 MPa			6
1 year 3.8 MPa			6
1 year 6.2 MPa			6
PCC unit weight	PC05	P65	12
Core examination thickness	PC06	P66	98 (all cores)
Air content, 28 day	PC08	P68	2
Flexural strength			
14 day 3.8 MPa	PC09	P69	3
14 day 6.2 MPa			3
28 day 3.8 MPa			3
28 day 6.2 MPa			3
1 year 3.8 MPa			3
1 year 6.2 MPa			3

For the SPS-2 experiment, the following materials testing tables were evaluated for data availability and completeness:

- TST_PC01—Compressive strength of in-place concrete test results for PCC layers.
- TST_PC02—Split tensile strength test results for PCC layers.
- TST_PC09—Flexural strength.
- TST_PC06—Core examination and thickness.

There are currently no data available on coefficient of thermal expansion for SPS-2 sections; such data are an essential variable for any mechanistic analysis.

The data availability and completeness assessment results for these key PCC materials testing tables are presented in table 16. As shown, 9 of 13 projects have good to excellent data availability for these tables, ranging from 81 to 100 percent tests completed. Three sites—Arkansas, Kansas, and Wisconsin—have a fair amount of PCC testing data available, ranging from 66 to 71 percent. The North Carolina SPS-2 project is missing much PCC testing data at the time of analysis, with only 33 percent available.

TRAFFIC DATA

Traffic data provide estimates of annual vehicle counts by vehicle classification, and distributions of axle weights by axle type. Annual traffic summary statistics are stored in the IMS traffic module. Data are provided for each year since the road was opened to traffic. With few exceptions (such as annual average daily traffic (AADT)-based values), the information applies only to the lane being studied. Traffic data are collected by the individual States/Provinces using a combination of permanent and portable equipment.

For the SPS-2 experiment, traffic data are generally obtained at the site level. In places where an intersection is located within the test site (thus resulting in different traffic levels on the test sections), measurements of the traffic level on the different groups of sections on each side of the intersection should be obtained. For simplicity and consistency, a traffic data availability assessment is conducted on a section-by-section basis.

The SPS-2 experiment design calls for continuous weigh-in-motion (WIM) monitoring, as permitted by WIM scale operating conditions. Table TRF_MONITOR_BASIC_INFO was examined to identify SPS-2 records containing WIM records, automatic vehicle classifier (AVC) data, and annual ESAL estimates. The WIM and AVC data were further classified into “at least 1 day” and “continuous” monitoring frequency categories. Continuous AVC monitoring was defined as over 300 AVC monitoring days in a given year. Continuous WIM monitoring was defined as over 210 WIM monitoring days in a given year.

Table 17 summarizes the data availability and completeness for SPS-2 traffic data. As shown, very few sections have continuous WIM or AVC monitoring data stored in the IMS database. Non-zero computed annual ESALs were found for 84 core SPS-2 sections at 8 sites with 83 records at level E status. A total of 71 core sections (nearly half of the core sections) have neither WIM monitoring data nor annual ESAL estimates data in the table TRF_MONITOR_BASIC_INFO. Additional annual ESAL estimates were available for 15 supplemental sections located in 6 different States.

Table 16. Data availability assessment for key PCC material testing tables.

Tables	**TST_PC01 Comp. Strength**		**TST_PC02 Split Tensile**		**TST_PC09 Flex. Strength**		**TST_PC06 Core Exam.**	**All**
Min. Req'd*	**Total 27**		**27**		**9**		**98**	**224**
Core Section Summary	**702 records (96% at level E)**		**613 records (87% at level E)**		**282 records (88% at level E)**		**1121 (100% at E)**	**Avg. % Tests Conducted (% range)**
States /Cells	**3.8 MPa**	**6.2 MPa**	**3.8 MPa**	**6.2 MPa**	**3.8 MPa**	**6.2 MPa**	**All**	
AZ	28 (>100%)	26 (96%)	26 (96%)	28 (>100%)	9 (100%)	18 (>100%)	114 (>100%)	**99% (96 - >100%)**
AR	19 (70%)	18 (67%)	10 (37%)	10 (37%)	15 (>100%)	6 (67%)	120 (>100%)	**68% (37 - >100%)**
CO	72 (>100%)	86 (>100%)	53 (>100%)	53 (>100%)	25 (>100%)	27 (>100%)	57 (58%)	**94% (58 - >100%)**
DE	21 (78%)	31 (>100%)	21 (78%)	26 (96%)	9 (100%)	9 (100%)	82 (84%)	**91% (78 - >100%)**
IA	26 (96%)	25 (93%)	27 (100%)	27 (100%)	9 (100%)	9 (100%)	121 (>100%)	**98% (93 - >100%)**
KS	19 (70%)	17 (63%)	17 (63%)	17 (63%)	21 (>100%)	17 (>100%)	0	**66% (0 - >100%)**
MI	24 (89%)	18 (67%)	23 (85%)	17 (63%)	9 (100%)	7 (78%)	88 (90%)	**82% (63 – 100%)**
NV	27 (100%)	24 (89%)	27 (100%)	24 (89%)	9 (100%)	9 (100%)	116 (>100%)	**97% (89 - >100%)**
NC	7 (26%)	24 (89%)	12 (44%)	12 (44%)	2 (22%)	3 (33%)	0	**37% (0 – 89 %)**
ND	27 (100%)	24 (89%)	14 (52%)	13 (48%)	8 (89%)	8 (89%)	98 (100%)	**81% (48 – 100%)**
OH	26 (96%)	25 (93%)	28 (>100%)	23 (85%)	9 (100%)	9 (100%)	125 (>100%)	**96% (93 - >100%)**
WA	29 (>100%)	27 (100%)	30 (>100%)	27 (100%)	12 (>100%)	9 (100%)	122 (>100%)	**100%**
WI	18 (67%)	20 (74%)	18 (67%)	20 (74%)	6 (67%)	8 (89%)	62 (63%)	**71% (67 – 89%)**

****Note:*** Min. req'd refers to the minimum number of tests required for the PCC layer.

Table 17. Traffic monitoring data availability assessment for SPS-2 experiment.

Data Availability	AVC		WIM		With at Least 1-Year Annual ESAL Computed
	At least 1 day	Continuous	At least 1 day	Continuous	
Core Sections (Total 155 Sections)					
At all levels (A to E)	96	37	84	23	84
At level E only	83	12	83	10	83
Percent data at level E	86	32	99	43	99
Core sections missing data at all levels	59	118	71	132	71
Sites with NO data at all levels	4	9	5	11	5
Supplemental Sections (Total 40 Sections)					
Supplemental sections with data	15	4	15	2	15
Supplemental sections missing data	25	36	25	38	25

CLIMATE DATA

There are three types of climatic information: general environmental, automated weather station (AWS), and seasonal monitoring. General environmental and AWS data for the SPS-2 project are obtained at a project or site level.

The general environmental information includes actual measurements from at least one nearby weather station for each LTPP site. In addition, a site-specific statistical estimate based on as many as five nearby weather stations is available. The estimates are called *virtual weather stations*. The IMS contains monthly and annual summary statistics. Daily data for both the virtual weather stations and actual weather stations are kept offline. General environmental data available in the IMS are derived from weather data originally collected from the National Climatic Data Center and the Canadian Climatic Center.

AWSs have been installed at nearly all of the SPS-2 project sites. The equipment provides site-specific information for the same parameters as the general environmental tables. AWS tables are available with monthly, daily, or hourly statistics.

The availability of both types of climatic data is shown in table 18. As noted, historic climatic data are available for all SPS-2 sites except Wisconsin. AWS data are available for 10 of 13 SPS-2 sites; Arkansas, Delaware, and Wisconsin have no data. The time periods covered by the AWS data at these sites range from 3 to 6 years.

Table 18. SPS-2 climate information availability.

State	Age as of August 1999	General Environment Information, Number of Years with Data				AWS, Number of Years with Data	
		Temperature		Precipitation		Temperature	Precipitation
		At All Levels	At Level E	At All Levels	At Level E	At All Levels	At Level E
AZ	5.8	17	15	17	17	6	6
AR	3.8	17	17	17	17	0	-
CO	5.8	17	17	17	17	5	5
DE	3.3	17	17	17	17	0	-
IA	5.0	17	17	17	17	3	3
KS	7.1	17	17	17	17	4	4
MI	5.8	17	17	17	17	3	3
NV	4.0	17	17	17	17	5	5
NC	5.1	17	17	17	17	5	5
ND	4.8	17	17	17	17	5	5
OH	2.9	17	17	17	17	6	6
WA	3.8	17	17	17	17	5	5
WI	1.8	0	0	0	0	0	0

MONITORING DATA

Seven types of monitoring data are included in the LTPP IMS: automated distress, manual distress, friction, longitudinal profile, cross profile, deflection, and dynamic load response. The monitoring data reviewed for the SPS-2 project are broken into the following categories for discussion:

- Longitudinal profile data.
- Deflection data.
- Faulting data.
- Manual and photographic distress data.
- Friction data.

In this section, the monitoring frequency requirement is discussed first, followed by the data availability and completeness assessment of all the categories.

Monitoring Frequency

During the life of these pavement sections, multiple directives have been issued regarding the testing frequency for each type of monitoring data collected. Some of these directives have slightly adjusted the testing intervals during the life of the program. The following is a list of the key documents and directives that affect the monitoring frequency requirement for the SPS-2 project:

1. *Data Collection Guide for Long-Term Pavement Performance Studies*.[2]
2. LTPP Directive D-02: *Quality Assurance of PASCO Products*.[4]
3. LTPP Directive D-05: *Measurement Frequency and Priorities of Manual Distress Surveys*.[5]
4. LTPP Directive FWD-03: *Deflection Monitoring Frequency Priorities and Use of FWDs Owned by Other Agencies*.[6]
5. LTPP Directive FWD-10: *Deflection Monitoring Frequencies and Priorities*.[7]
6. LTPP Directive P-02: *Profile Monitoring Frequencies and Priorities*.[8]
7. LTPP Directive GO-20: *Revised Friction Measurement Requirements*.[9]
8. LTPP Directive GO-21: *LTPP Test Section Monitoring Adjustments*.[10]

These directives were used to identify all previous testing frequencies for each type of monitoring data collected, and are summarized in table 19. For supplemental sections, the monitoring frequencies are every 3 years for manual distress and joint faulting monitoring, every 2 years and responsive for photographic survey, and every 5 years and responsive for falling-weight deflectometer (FWD) testing.

Table 19. Testing frequencies for SPS-2 monitoring data collection.

Data Collection Type	**Postconstruction Monitoring**	**Long-Term Monitoring Frequency**	
		In Effect Before October 1, 1999	**In Effect After October 1, 1999**
Longitudinal profile	< 6 months is permitted	Biennially, but may be postponed up to 1 year	Annually
Deflection (for nonfractured PCC)	< 6 months is permitted	Biennially and responsive	Biennially and responsive
Manual distress and faulting	< 3 months	Biennially, but may be postponed up to 1 year	Annually
Photographic	Not specified	–	Biennially
Friction	<12 months	Biennially	Not specified

In addition, close-out monitoring consisting of FWD, profile, and manual distress surveys should be conducted on each section. According to LTPP Directive GO-21, this monitoring is performed "when it is determined that the test section will be taken out-of-study (due to a construction event or at the option of the highway agency) or at the end of the field monitoring portion of the LTPP program, whichever comes first." [10]

The testing frequency requirements specified in LTPP Directive GO-21 are also listed in the table. This requirement has been in effect since October 1, 1999.

Monitoring Data Assessment

The following IMS monitoring tables are used in evaluating the data availability and completeness for SPS-2 monitoring:

- MON_PROFILE_MASTER—Monitoring profilometer master record.
- MON_DEFL_DROP_DATA—Peak and other drop-specific data values for Dynatest FWD.
- MON_DIS_JPCP_FAULT_SECT—Section faulting statistics for transverse joints and cracks.
- MON_DIS_JPCP_REV—Distress identification for jointed PCC surfaces.
- MON_DIS_PADIAS_JPCP—Distress identification for JPCP surfaces.
- MON_DIS_PADIAS42_JPCP—Distress identification for jointed PCC surfaces.
- MON_FRICTION—Friction-resistance measurements.

Tables 20 to 24 show summaries of the data availability assessment for longitudinal profile, deflection, faulting, manual, and photographic distress surveys, and friction-monitoring data for SPS-2. Using the minimum monitoring data collection requirement noted in these tables, an assessment of this data availability and completeness follows:

- Longitudinal profile data are acceptable for most sites, but some are seriously deficient (such as Arkansas and North Dakota) with very late initial longitudinal profile measurement.
- Deflection data are very complete. The only exception is the late initial survey at the Arkansas site.
- Faulting data are very deficient at the Kansas site. Data from Arizona, Arkansas, Colorado, and North Carolina are late. The other eight sites have adequate faulting data.
- Combined distress data satisfies the minimum requirement, except for the initial surveys of Arizona, Arkansas, and North Dakota.
- Friction monitoring data are deficient, with very few test visits, too-long survey intervals, and far too long to initial friction measurement.

Table 20. Summary of the number of the surveys for longitudinal profile data collection.

SPS-2 Project in State	Age as of Aug. 1999	No. of Test Visits			Long-Term Interval, year	Initial Survey Age, month	Meet Minimum Requirement? *
		Core Sections		Suppl. Sections			
		Avg (min-max)	% At E		Avg (min-max)	Avg (min-max)	
AZ	6.3	4.9 (4-5)	100	5.0 (5-5)	1.3 (1-2)	3.8 (4-4)	✓
AR	4.3	1.0 (1-1)	100	N/A	N/A	16.2 (16-16)	Not the initial survey
CO	6.3	3.0 (3-3)	100	3.0 (3-3)	2.2 (2-2)	6.4 (6-6)	✓
DE	3.7	6.0 (6-6)	100	6.0 (6-6)	0.3 (0-0)	7.2 (7-7)	✓
IA	5.4	4.9 (4-5)	100	5.0 (5-5)	1.1 (1-1)	6.5 (7-7)	✓
KS	7.5	7.9 (7-8)	100	7.0 (7-7)	0.9 (1-1)	2.0 (1-8)	✓
MI	6.2	8.6 (7-9)	99	9.0 (9-9)	0.6 (1-1)	10.2 (10-10)	✓
NV	4.4	3.6 (2-4)	100	4.0 (4-4)	0.7 (1-1)	10.9 (11-11)	✓
NC	5.5	7.9 (7-8)	100	7.5 (7-8)	0.7 (1-1)	<0 (-3-3)	✓
ND	5.3	2.0 (2-2)	100	2.0 (2-2)	2.0 (2-2)	31.0 (31-31)	Not the initial survey
OH	3.3	4.0 (4-4)	100	3.4 (3-4)	0.7 (1-1)	<0 (-1-1)	✓
WA	4.2	4.0 (4-4)	100	4.0 (4-4)	1.2 (1-1)	0.6 (1-1)	✓
WI	2.3	3.0 (3-3)	100	3.0 (3-3)	0.7 (1-1)	2.0 (2-2)	✓

Note: * Minimum longitudinal profile data collection requirement: Within 12 months for the initial survey, and less than 3 years for all long-term monitoring intervals (i.e., biannually, but may be postponed up to 1 year).

Table 21. Summary of the number of the surveys for deflection data collection.

SPS-2 Project in State	Age as of Aug. 1999	No. of Test Visits			Long-Term Interval, year	Initial Survey Age, month	Meet Minimum Requirement? *
		Core Sections		Suppl. Sections			
		Avg (min-max)	% At E		Avg (min-max)	Avg (min-max)	
AZ	6.3	6.1 (5-7)	100	2.3 (1-5)	1.1 (1-1)	<0 (-2-2)	✓
AR	4.3	1.5 (1-2)	100	N/A	0.0 (0-0)	13.5 (13-14)	Not the initial survey
CO	6.3	4.2 (3-5)	100	3.0 (3-3)	1.6 (1-2)	0.5 (-2-6)	✓
DE	3.7	2.2 (2-3)	100	2.0 (2-2)	1.2 (1-1)	0.2 (0-0)	✓
IA	5.4	1.4 (1-2)	100	1.0 (1-1)	2.6 (3-3)	5.3 (3-35)	OK except for 0219
KS	7.5	5.3 (5-7)	100	2.0 (2-2)	1.1 (1-1)	1.1 (1-1)	✓
MI	6.2	4.2 (3-5)	100	4.0 (4-4)	1.4 (1-2)	0.5 (0-1)	✓
NV	4.4	4.0 (2-6)	100	2.0 (2-2)	0.7 (0-1)	<0 (-2-1)	✓
NC	5.5	2.4 (2-5)	100	2.0 (2-2)	1.9 (1-2)	<0 (-2-2)	✓
ND	5.3	1.3 (1-2)	100	1.3 (1-2)	0.8 (1-1)	1.1 (1-1)	✓
OH	3.3	3.9 (3-5)	100	3.3 (2-4)	0.6 (0-1)	<0 (-14--12)	✓
WA	4.2	4.7 (4-5)	100	4.0 (4-4)	0.9 (1-1)	<0 (-4-4)	✓
WI	2.3	1.0 (1-1)	0	0.9 (0-1)	N/A	6.8 (7-7)	✓

Note: * Minimum deflection data collection requirement: Within 12 months for the initial survey, and less than 3 years for all long-term monitoring intervals (i.e., biannually, but may be postponed up to 1 year).

Table 22. Summary of the number of the surveys for faulting data collection.

SPS-2 Project in State	Age as of Aug. 1999	No. of Test Visits			Long-Term Interval, year	Initial Survey Age, month	Meet Minimum Requirement? *
		Core Sections		Suppl. Sections	Avg (min-max)	Avg (min-max)	
		Avg (min-max)	% At E				
AZ	6.3	2.6 (2-3)	100	2.3 (0-3)	1.7 (1-2)	29 (17-49)	Not the initial survey
AR	4.3	1.0 (1-1)	100	N/A	N/A	14 (13-14)	Not the initial survey
CO	6.3	3.0 (3-3)	100	3.0 (3-3)	1.7 (2-2)	30.8 (31-31)	Not the initial survey
DE	3.7	3.2 (3-4)	100	3.0 (3-3)	1.5 (1-2)	0.2 (0-0)	✓
IA	5.4	2.8 (2-3)	100	2.0 (2-2)	2.9 (2-5)	2.6 (3-3)	✓
KS	7.5	1.9 (1-2)	100	1.0 (1-1)	4.1 (4-4)	13.4 (9-59)	No
MI	6.2	4.8 (3-6)	100	4.0 (4-4)	1.4 (1-1)	0.5 (0-1)	✓
NV	4.4	2.2 (2-3)	100	2.0 (2-2)	2.1 (1-3)	7.9 (8-8)	✓
NC	5.5	2.4 (1-6)	100	2.0 (2-2)	2.6 (1-3)	2.6 (0-33)	Not the initial survey for 0204, 0208, 0212
ND	5.3	2.1 (1-3)	100	1.7 (1-2)	3.9 (2-5)	0.8 (1-1)	✓
OH	3.3	2.0 (2-2)	100	2.0 (2-2)	2.7 (3-3)	3.4 (3-3)	✓
WA	4.2	3.0 (3-3)	100	3.0 (3-3)	1.5 (1-1)	0.0 (0-0)	✓
WI	2.3	1.1 (1-2)	0	0.9 (0-1)	N/A	6.8 (7-7)	✓

Note: * Minimum faulting data collection requirement: Within 12 months for the initial survey, and less than 3 years for all long-term monitoring intervals (i.e., biannually, but may be postponed up to 1 year).

Table 23. Summary of the number of the surveys for manual and photographic distress data collection.

SPS-2 Project in State	Age as of Aug. 1999	No. of Test Visits			Long-Term Interval, year	Initial Survey Age, month	Meet Minimum Requirement? *
		Core Sections		Suppl. Sections	Avg (min-max)	Avg (min-max)	
		Avg (min-max)	% At E				
AZ	6.3	3.6 (3-4)	100	3.1 (0-4)	1.5 (1-2)	17.2 (17-18)	Not the initial survey
AR	4.3	1.0 (1-1)	100	N/A	N/A	13.5 (13-14)	Not the initial survey
CO	6.3	5.1 (5-6)	100	5.0 (5-5)	1.3 (1-1)	9.4 (9-9)	✓
DE	3.7	3.2 (3-4)	100	3.0 (3-3)	1.5 (1-2)	0.2 (0-0)	✓
IA	5.4	3.8 (3-5)	100	3.0 (3-3)	1.7 (1-2)	2.6 (3-3)	✓
KS	7.5	3.8 (3-4)	100	4.0 (4-4)	1.5 (1-2)	9.2 (9-9)	✓
MI	6.2	5.8 (4-7)	100	6.0 (6-6)	1.1 (1-1)	0.5 (0-1)	✓
NV	4.4	3.2 (3-4)	100	3.0 (3-3)	1.1 (1-1)	7.9 (8-8)	✓
NC	5.5	2.5 (2-6)	100	2.0 (2-2)	1.0 (1-1)	19.0 (17-19)	Not the initial survey
ND	5.3	3.3 (3-4)	100	3.0 (3-3)	2.1 (2-2)	0.8 (1-1)	✓
OH	3.3	3.0 (3-3)	100	2.6 (2-3)	1.5 (2-2)	<0 (0-0)	✓
WA	4.2	3.0 (3-3)	100	3.0 (3-3)	1.5 (1-1)	<0 (0-0)	✓
WI	2.3	1.0 (1-1)	0	0.9 (0-1)	N/A	6.8 (7-7)	✓

Note: * Minimum manual and photographic data collection requirement: Within 12 months for the initial survey, and less than 3 years for all long-term monitoring intervals (i.e., biannually, but may be postponed up to 1 year).

Table 24. Summary of the number of the surveys for friction data collection.

SPS-2 Project in State	Age as of Aug. 1999	No. of Test Visits			Long-Term Interval, year	Initial Survey Age, month	Meet Minimum Requirement? *
		Core Sections		Suppl. Sections			
		Avg (min-max)	% At E		Avg (min-max)	Avg (min-max)	
AZ	6.3	0	–	–	–	–	No Data
AR	4.3	0	–	–	–	–	No Data
CO	6.3	1.0 (1-1)	100	1.0 (1-1)	–	7.0 (7-7)	No
DE	3.7	0	–	0.0 (0-0)	–	–	No Data
IA	5.4	3.0 (3-3)	100	3.0 (3-3)	1.0 (1-1)	13.0 (13-13)	No
KS	7.5	2.0 (2-2)	100	2.0 (2-2)	1.1 (1-1)	46.0 (46-46)	No
MI	6.2	0.5 (0-1)	100	1.0 (1-1)	–	46.8 (47-47)	No
NV	4.4	0	–	0.0 (0-0)	–	–	No Data
NC	5.5	0	–	0.0 (0-0)	–	–	No Data
ND	5.3	0	–	0.0 (0-0)	–	–	No Data
OH	3.3	0	–	0.0 (0-0)	–	–	No Data
WA	4.2	2.0 (2-2)	100	2.0 (2-2)	0.5 (0-1)	10.5 (6-11)	✓
WI	2.3	0	–	–	–	–	No Data

Note: * Minimum friction data collection requirement: Within 12 months for the initial survey, and less than 3 years for all long-term monitoring intervals (i.e., biannually, but may be postponed up to 1 year).

DYNAMIC LOAD-RESPONSE DATA

Various PCC pavement sections of the SPS-2 project were selected for measuring pavement response under controlled loading conditions. Both deflections and strains at defined positions within the slab were recorded under loading by known vehicles. Deflections of the PCC surface were measured at six locations (corner, midslab edge, and midslab out wheel path) within two adjacent slabs. The pavement surface strains were measured using surface-mounted strain gauges located at midslab within the wheel path and midslab along the slab edge. Data from a total of 30 traces were obtained from each pass of the loaded vehicle, with multiple repetitions at multiple speeds collected at various times of the day. During the early life of the pavement, dynamic load-response data were collected on a quarterly basis. Data collection was terminated after 2 years.

The dynamic load-response data for PCC sections are stored in the DLR_* module in the following seven IMS tables:

- DLR_LVDT_CONFIG_PCC—LVDT gauge device, settings, and location information.
- DLR_LVDT_TRACE_SUM_PCC—LVDT trace summary information.
- DLR_MASTER—Dynamic load response site and instrumentation summary information.

- DLR_STRAIN_CONFIG_PCC—Sensor gauge device, settings, and location information.
- DLR_STRAIN_TRACE_SUM_PCC—Data load response strain trace summary information.
- DLR_TEST_MATRIX—Data load response test matrix summary information.
- DLR_TRUCK_GEOMETRY—Data load response truck geometry summary information.

The only dynamic load response data in the IMS database are from the North Carolina and Ohio SPS-2 sites. The data availability assessment of these tables is provided in table 25. All records in these tables are at level E. As shown in the table, significant amounts of stress and strain data are available on the instrumented sections. These data should be very useful for the analysis of the pavement dynamic load responses.

Table 25. Data availability assessment for SPS-2 dynamic load response data.

Table Name	**Total Records (All at E)**	**State**	**Records for Each Section**			
DLR_LVDT_PCC	**880**	**37**	112	112	112	112
		39	96	112	112	112
DLR_LVDT_TRACE_SUM_PCC	**39,421**	**37**	9,089	9,106	8,146	9,954
		39	760	809	810	747
DLR_MASTER_PCC	**59**	**37**	8	8	8	8
		39	6	7	7	7
DLR_STRAIN_CONFIG_PCC	**1,051**	**37**	128	128	128	127
		39	120	140	140	140
DLR_STRAIN_TRACE_SUM_PCC	**31,659**	**37**	7,658	8,128	6,348	8,674
		39	199	136	240	276
DLR_TEST_MATRIX	**3,350**	**37**	556	681	804	803
		39	108	133	134	131
DLR_TRUCK_GEOMETRY	**3**	**37**	2 Truck load/types			
		39	1 Truck ID/type			

SUMMARY OF SPS-2 DATA AVAILABILITY AND COMPLETENESS ASSESSMENT

Table 26 summarizes the data availability and completeness by key data types that are not subject to long-term monitoring, while table 27 summarizes the data availability and completeness for the key data types subject to long-term monitoring. Note that any rating of "fair" or "poor" means that these sites would not meet analysis needs and therefore must be improved as soon as possible. The SPS-2 data deficiencies are summarized below:

- Wisconsin—newly constructed, data processing underway.
- Arizona, Arkansas, and North Carolina—late initial survey for most monitoring types.

- Colorado and North Dakota—late initial survey for one monitoring collection activity, either longitudinal profile measurements, deflection testing, faulting, or distress data.
- Kansas SPS-2—very deficient faulting data.
- Traffic data are very deficient for 5 of 13 sites (40 percent).
- Joint faulting data are not being collected at the frequency specified by LTPP data collection guidelines, and this will limit the analyses that can be conducted.
- Friction data are completely deficient for most projects (11 out of 13).
- Arkansas, Kansas, North Carolina, and Wisconsin are missing significant PCC material testing data at the time of analysis.

A very good percentage of the SPS-2 data are at level E. More than 82 percent of the records are at level E for all data types, with many greater than 99 percent. The availability and completeness of data for the SPS-2 experiment is good overall. However, a significant amount of data was not available at the time of analysis, especially traffic, distress and faulting surveys, and key materials testing data. These deficiencies need to be addressed before serious data analysis can be undertaken. There is an active plan in place to address the deficient materials testing and traffic data.

Table 26. Summary of the SPS-2 data availability and completeness for key data types.

Type of Data	**SPS-2 Core Sections—Total 13 Sites, 155 Sections**				**SPS-2 Supplemental Sections—Total 40 Sections**
	No. Sites (Sections)		**% At Level E**	**Comments**	
	W/ Data	**Missing Data**			
Site information (reports, location, and significant dates data)	13 sites (155)	ND—Construction date	100	Excellent	Excellent—Same as the core sections
Key design features (drainage and lane width)	11 sites (131)	KS, WI (all 24 sections)	100	Good	Good—Available for 27 to 29 sections
Pavement structure (subgrade layer, base, surface)	12 sites (143)	WI	82–89	Good	Good—Available for 26 to 30 sections
SPS-2 construction type data	11 to 13 (92–155)	AZ, OH, WA, WI (2 to 16 sections)	90–100	Good	Good—Available for 20 to 34 sections
Key PCC material testing	9 sites	AR, KS, NC, and WI	87–100	Fair to good	Not evaluated

Table 27. Summary of the SPS-2 data availability and completeness assessment for traffic, climate, and monitoring data types.

Monitoring Data Types	SPS-2 Sites and Core Sections—Total 13 Sites, 155 Sections						Comments
	Initial Survey <12 month		Long-Term Max. <3 years		No Data	% at Level E	
	Yes	No	Yes	No			
Longitudinal profile	11	AR, ND	13	0	–	99.9	Good
Deflection	11	AR, IA (0219)	13	0	–	86	Good
Faulting	9	AZ, AR, CO, NC (0204, 0208, 0212)	12	KS	–	99	Fair
Distress—manual and PASCO	10	AZ, AR, NC	13	0	–	100	Fair
Friction	2	11 sites	2	11	8 sites	92	Poor
Traffic and Climatic Data	**Sites with Data**				**No Data**	**% At Level E**	**Comments**
Traffic	Eight sites have at least 1 day of WIM data at 1 year.				5 sites	83	Poor
Climate	Twelve sites have 17 years of general climatic data. Ten sites have 3 to 6 years of AWS data.				WI	>99	Excellent

4. EXPERIMENTAL DESIGN VERSUS ACTUAL CONSTRUCTION

One of the main objectives of this study is to identify confounding factors introduced into the SPS-2 experiment by virtue of construction deviations or other factors not accounted for in the original experimental design. It is important to evaluate the variables that are considered as key design factors in the SPS-2 experiment and to determine if they meet the parameters established in the design factorial. Additionally, two SPS guideline reports established specific site-selection criteria and key variable construction guidelines.[11,12] The guidelines in both reports were developed to control the quality and integrity of the SPS-2 experiment results and findings, and therefore should be included in the construction adequacy evaluation.

This chapter evaluates the design and the actual construction of key variables identified in the experiment design factorial and the above-mentioned guidelines. This includes the following:

- Climate.
- Subgrade.
- Traffic.
- Concrete slab thickness.
- PCC flexural strength.
- Base layer.
- Drainage (edge drains).
- Lane width.

CLIMATE

The experimental design specified that the SPS-2 sites be located in four specific climates:

- Wet freeze.
- Wet no-freeze.
- Dry freeze.
- Dry no-freeze.

The main purpose of this requirement was to obtain representative SPS-2 sections in widely varying climates, with a geographic distribution across the continental United States Table 28 shows a summary of the design requirements and actual precipitation data. All of the sites meet the criteria, except two that were supposed to be in dry areas (Kansas and North Dakota). The Kansas site is much wetter than the design limit, and the North Dakota site is just barely wetter than the limit.

What effect will these deviations have on achieving the objectives relative to climate? Analysis of the data will utilize the actual precipitation, not dry or wet variables. The only limitation is that the performance from the Kansas site will represent an area with greater precipitation than desired (819 mm versus 508 mm maximum or dry area); however, this site is still much drier than the corresponding wet sites. Kansas has 819 mm annual precipitation, and the other wet sites range from 865 to 1,380 mm with an average of 1,068 mm.

Table 28. Summary of the SPS-2 designed versus as-constructed sites, annual precipitation.

SPS-2 Project in State	Designated		Actual Precipitation, mm		Designated?
	Zone	Precipitation, mm	From General Climatic Information	From AWSs	
AZ	Dry	< 508	232.02	198.75	✓
AR	Wet	> 508	1380.55	–	✓
CO	Dry	< 508	369.74	344.00	✓
DE	Wet	> 508	1143.92	–	✓
IA	Wet	> 508	900.46	–	✓
KS	Dry	< 508	819.48	698.00	No
MI	Wet	> 508	865.59	871.00	✓
NV	Dry	< 508	221.51	249.33	✓
NC	Wet	> 508	1,150.78	1,198.50	✓
ND	Dry	< 508	544.61	534.00	No
OH	Wet	> 508	971.56	730.00	✓
WA	Dry	< 508	308.44	355.00	✓
WI	NA	NA	NA	NA	NA

The freezing index data are shown in table 29. As shown, all sites meet the criteria for freeze and non-freeze based on the annual freezing index criteria.

Table 29. Summary of the SPS-2 designed versus constructed sites, annual freezing index.

SPS-2 Project in State	Designated		Freezing Index, ^{o}C-days		Same as Designated?
	Zone	Freezing Index, ^{o}C-days	From General Climatic Information	From AWSs	
AZ	No-freeze	< 83.3	0.0	0.0	✓
AR	No-freeze	< 83.3	38.0	–	✓
CO	Freeze	> 83.3	327.4	394.0	✓
DE	Freeze	> 83.3	102.7	–	✓
IA	Freeze	> 83.3	579.7	–	✓
KS	Freeze	> 83.3	259.1	254.0	✓
MI	Freeze	> 83.3	381.9	140.0	✓
NV	Freeze	> 83.3	275.8	180.7	✓
NC	No-freeze	< 83.3	47.2	67.0	✓
ND	Freeze	> 83.3	1,313.1	1,162.0	✓
OH	Freeze	> 83.3	374.5	121.0	✓
WA	Freeze	> 83.3	264.8	138.0	✓
WI	NA	NA	NA	NA	NA

SUBGRADE

The SPS-2 experimental design called for half of the sites to be constructed on coarse-grained subgrade soils, and the other half to be constructed on fine-grained soils. Furthermore, it was required that all test sections at one site must be constructed on soils classified as same soil type, either fine-grained or coarse-grained.

Table 30 provides a comparison of the designated versus constructed subgrade types for all SPS-2 projects. Information from both cores taken from constructed pavements (TST_L05B table) and construction surveys (SPS2_LAYER table) is provided for comparison purposes. As indicated, for 11 of 13 SPS-2 projects, the subgrade soils are approximately uniform for all the core sections within the project. Furthermore, the soil types are now consistent between the designated and the constructed after correcting the subgrade type of the Washington SPS-2 project from fine-grained to coarse-grained. Further evaluation of the site data is needed to assess the significance of this finding.

For the Colorado site, the project was designed as a coarse-grained subgrade soil. However, four sections within the project were found to be constructed on fine-grained sandy clay soil. For the Nevada site, the project was designed as a coarse-grained soil. However, 9 out of the 11 sections were constructed on fine-grained sandy silt soil. Further evaluation is needed of the site data to assess the significance of this finding.

Table 30. Comparison of the SPS-2 designed versus constructed values for subgrade types.

SPS-2 Project in State	Assigned Soil Type	From TST_L05B Table			From SPS2_LAYER Table
		Soil Type	No. Sections	Ok?	
AZ	Coarse	Coarse-grained: clayey sand with gravel or silty sand with gravel	12	✓	Clayey gravel or poorly graded gravel
AR	Fine	Fine-grained: silty clay	12	✓	Silty clay
CO	Coarse	Coarse-grained: clayey sand, poorly graded sand with silt, or well-graded sand with silt	8	✓	Clayey sand or poorly graded sand
		Fine-grained: sandy clay or sandy lean clay	4	No	Sandy clay
DE	Coarse	Coarse-grained: clayey sand or silty sand	12	✓	Silty sand
IA	Fine	Fine-gained: clay with gravel	12	✓	Silty clay
KS	Fine	Fine-grained: silty clay	12	✓	Silty clay
MI	Fine	Fine-grained: sandy clay or silty clay	12	✓	Sandy clay
NV	Coarse	Coarse-grained: silty sand with gravel	2	✓	Silt
	Coarse	Fine-grained: sandy silt	9	No	Silt
NC	Fine	Fine-grained: clay, clayey slit, sand silt, or sandy silty clay	12	✓	Silty clay
ND	Fine	Fine-grained: clay	12	✓	Silty clay
OH	Fine	Fine-grained: silty clay	12	✓	Silty clay
WA	Coarse	Coarse-grained: poorly graded gravel	12	✓	Poorly graded gravel or sandy silt
WI	Coarse	NA	12	✓	Silty sand

TRAFFIC

In the original SPS-2 experimental design, traffic was incorporated as a covariant. The traffic rate of at least 200,000 ESALs per year was required. The required annual ESAL and actual ESALs per year are compared in table 31. As shown, this requirement was met for most of the sites and years, with exceptions of the annual traffic for Iowa 1997. The annual ESAL data are not completely available at the time of analysis for five SPS-2 sites (38 percent). The wide range of traffic loadings between sites will need to be fully considered in any comparative analysis between sites.

Table 31. Comparison of the designed versus actual values for annual traffic.

SPS-2 Project in State	Required ESALs per Year	Year Recorded	Annual ESALs from IMS Database TRF_MONITOR_BASIC_INFO			No. of Sections
			Avg.	Min	Max	
AR	>200,000	–	–	–	–	-
AZ	>200,000	1994	1,343,854	1,333,149	1,352,180	12
AZ	>200,000	1995	725,978	722,887	731,911	12
AZ	>200,000	1996	1,091,263	1,086,667	1,095,274	11
CO	>200,000	1995	477,870	463,068	487,401	24
CO	>200,000	1996	341,187	334,124	346,082	12
CO	>200,000	1997	223,882	220,773	226,004	12
DE	>200,000	–	–	–	–	–
IA	>200,000	1997	56,406	56,125	57,013	12
KS	>200,000	1993	639,131	639,131	639,131	1
MI	>200,000	1993	596,967	588,953	602,291	12
MI	>200,000	1994	1,778,419	1,710,288	1,816,069	12
MI	>200,000	1996	1,495,685	1,445,548	1,524,539	12
MI	>200,000	1997	2,550,760	2,447,282	2,608,271	12
MI	>200,000	1998	1,661,157	1,620,051	1,684,665	12
NV	>200,000	1997	812,944	799,856	819,517	11
NC	>200,000	1994	779,957	738,986	804,407	12
NC	>200,000	1995	716,309	681,993	737,157	12
NC	>200,000	1996	816,174	774,908	841,857	12
NC	>200,000	1997	727,578	697,168	746,904	13
NC	>200,000	1998	792,086	761,745	809,605	12
ND	>200,000	–	–	–	–	–
OH	>200,000	–	–	–	–	–
WA	>200,000	1998	461,759	452,372	470,407	12
WI	>200,000	–	–	–	–	–

CONCRETE SLAB THICKNESS

The SPS-2 experimental design specifies two levels for concrete slab thickness: 203 mm and 279 mm. The SPS-2 construction guideline requires that the concrete slab thickness should be constructed within ±6.4 mm. Many sections did not meet this guideline. Therefore, for practical reasons, ±12.7 mm was used as the thickness tolerance or the design range. Table 32 compares designed versus constructed or measured mean PCC thicknesses from table TST_L05B. Thirty-

Table 32. Designed versus mean constructed SPS-2 PCC slab thickness, mm.

State	Sections NOT Within Limit?	Section Number											
		0201, 0213	**0202, 0214**	**0205, 0217**	**0206, 0218**	**0209, 0221**	**0210, 0222**	**0203, 0215**	**0204, 0216**	**0207, 0219**	**0208, 0220**	**0211, 0223**	**0212, 0224**
		Design Value: 203 (190 to 216), mm						**279 (267 to 292), mm**					
AZ (0213-0224)	1	201	211	206	211	208	**218**	287	284	274	287	282	272
AR (0213-0224)	2 (all below)	**188**	211	191	**188**	208	213	284	277	282	272	277	277
CO (0213-0224)	6	**221**	213	**218**	196	211	**221**	290	**300**	282	282	**300**	**297**
DE (0201-0212)	7	211	**224**	**234**	**226**	208	211	**297**	279	287	**307**	**300**	**315**
IA (0213-0224)	5	216	213	196	208	**239**	211	**300**	**295**	284	290	**297**	**295**
KS (0201-0212)	1 (below)	196	**188**	198	201	216	211	282	287	287	279	282	277
MI (0213-0224)	3 (1 below)	**218**	**226**	216	**180**	208	213	284	290	277	282	279	284
NV (0201-0212)	5	**234**	208	216	198	**226**	**257**	**302**	**300**	277	279	287	–
NC (0201-0212)	4	**229**	**259**	203	213	**218**	213	284	284	**295**	284	290	277
ND (0213-0224)	0	208	201	201	201	206	208	279	284	277	277	282	274
OH (0201-0212)	0	201	211	203	201	206	203	277	282	282	279	290	269
WA (0201-0212)	4	**221**	211	216	**218**	**229**	211	282	284	282	284	**300**	287
WI (0213-0224)	NA	–	–	–	–	–	–	–	–	–	–	–	–
Summary	**38 out of 143 sections (27%) are outside the design range, with 4 below and 34 above the limits.**												

Note: Bolded numbers are outside the design required range.

Table 33. Designed versus mean constructed SPS-2 PCC slab flexural strength, MPa.

State	**Section Number**											
	0201, 0203, 0205, 0207, 0209, 0211, or 0213, 0215, 0217, 0219, 0221, 0223						**0202, 0204, 0206, 0208, 0210, 0212, or 0214, 0216, 0218, 0220, 0222, 0224**					
	Design Value: 3.8 MPa						**6.2 MPa**					
	No. of Samples	Avg	StD	% Deviation	Average within 10%?	Average within 20%?	**No. of Samples**	Avg	StD	% Deviation	Average within 10%?	Average within 20%?
AZ (0213-0224)	3	3.94	0.07	3.73	✓	✓	6	5.77	0.40	-6.95	✓	✓
AR (0213-0224)	5	3.76	0.20	-1.04	✓	✓	2	4.59	1.56	-25.99	–	–
CO (0213-0224)	9	3.63	0.31	-4.54	✓	✓	9	6.25	0.40	0.77	✓	✓
DE (0201-0212)	3	4.53	0.69	19.15	–	✓	3	5.22	1.05	-15.85	–	✓
IA (0213-0224)	3	3.22	0.21	-15.32	–	✓	3	5.19	0.33	-16.22	–	✓
KS (0201-0212)	7	4.23	0.33	11.25	–	✓	6	5.81	0.34	-6.21	✓	✓
MI (0213-0224)	1	4.27		12.50	–	✓	2	6.71	0.02	8.15	✓	✓
NV (0201-0212)	3	3.60	0.22	-5.34	✓	✓	3	5.41	0.60	-12.70	–	✓
NC (0201-0212)	–	–	–	–	–	–	–	–	–	–	–	–
ND (0213-0224)	–	–	–	–	–	–	–	–	–	–	–	–
OH (0201-0212)	3	4.72	0.39	24.17	–	–	3	4.23	1.05	-31.75		
WA (0201-0212)	4	3.34	0.38	-12.00	–	✓	3	5.73	0.24	-7.55	✓	✓
WI (0213-0224)	3	4.37	0.20	14.92	–	✓	4	6.09	0.36	-1.72	✓	✓
Summary	**7 sites over 10% and 1 site over 20% deviation, with 2 sites below and 5 sites above the design value.**						**5 sites over 10% and 2 sites over 20% deviation, all below the design value.**					

8 of 143 SPS-2 sections (27 percent) fall outside of the design ranges (design value ±12.7 mm), with 4 sections having below-range values and 34 sections having above-range values. Twelve sections, all at the Wisconsin SPS-2 site, do not have thickness information in TST_L05B table at the time of analysis.

The frequency distributions of the tested slab thickness from table TST_L05B are provided in figure 4 for 203-mm design cells, and figure 5 for 279-mm design cells. As shown, 203-mm cell design sections have more scatter slab thickness distribution. Both the 203-mm and 279-mm mean thickness distribution shows a skew toward higher-than-designed thicknesses.

PCC FLEXURAL STRENGTH

The SPS-2 experimental design specifies two levels for concrete flexural strength at 14 days: 3.8 MPa and 6.2 MPa. Table TST_PC09 was examined to compare the designed and constructed flexural strength values. The 14-day concrete flexural strength data were found for 11 SPS-2 sites; North Carolina and North Dakota sites' flexural strength information was not available at the time of analysis.

The design versus constructed SPS-2 PCC slab flexural strength comparison results are given in table 33. For the 3.8 MPa design cells, 7 of the 11 sites (64 percent) have average tested flexural strength values 10 percent outside of the design range, and 1 site's values were 20 percent outside of the design range. For the seven sites that are 10 percent outside of the design range, two sites are below the design range (3.8 MPa) and five are above. For the 6.2 MPa design cells, 5 of the 11 sites (45 percent) have average tested flexural strength 10 percent outside of the design range, and 3 sites' data were 20 percent outside of the design range. All of these five sites fall below the design value of 6.2 MPa.

The frequency distributions of the tested flexural strength values are provided in figure 6 for 3.8 MPa design cells, and in figure 7 for 6.2 MPa design cells. As shown, the distribution of the 3.8 MPa design cells is closer to a normal distribution, while the distribution of the 6.2 MPa design cells is very skewed to the right.

Field studies have shown that PCC continues to gain strength over many years. The 1 year strength data may be more indicative of the actual strength over the 20-year pavement evaluation period than the 20-day data. The differences in strength levels at 1 year are very important. Time-series plots were generated for concrete strength, as shown in figures 8 to 10. For most sites, the time-series plot of the concrete strength remains more or less parallel between 3.8 MPa and 6.2 MPa cells. The frequency distributions of the 1-year modulus of rupture values are shown in figures 11 and 12. They remain two distinct distributions with some overlay.

Statistical *t*-tests were performed on both the 14-day concrete strength and 1-year concrete strength, and the results are presented in table 34. Even though the mean difference of the strength measurements decreases from 1.71 MPa at 14 days to 1.24 MPa at 1 year, the strength differences between the lower and higher strength concrete were still very significant at 1 year specimen age. This finding indicates that overall concrete strength values of the 6.2 MPa cells are still significantly higher than those of the 3.8 MPa cells at 1 year of pavement age.

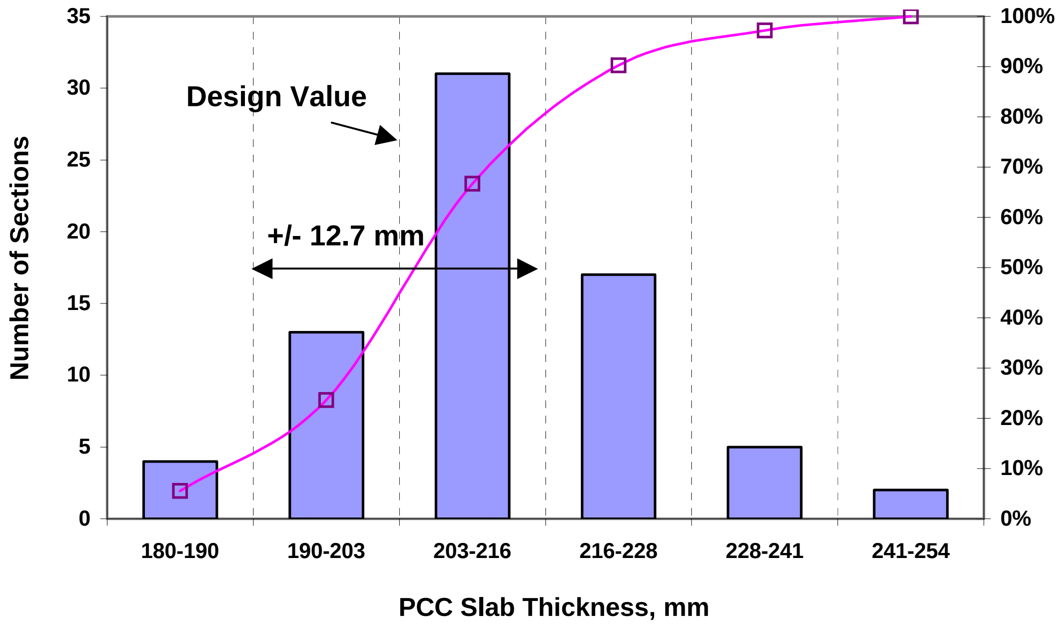


Figure 4. Frequency distribution of the mean PCC slab thickness for SPS-2 203-mm cells.

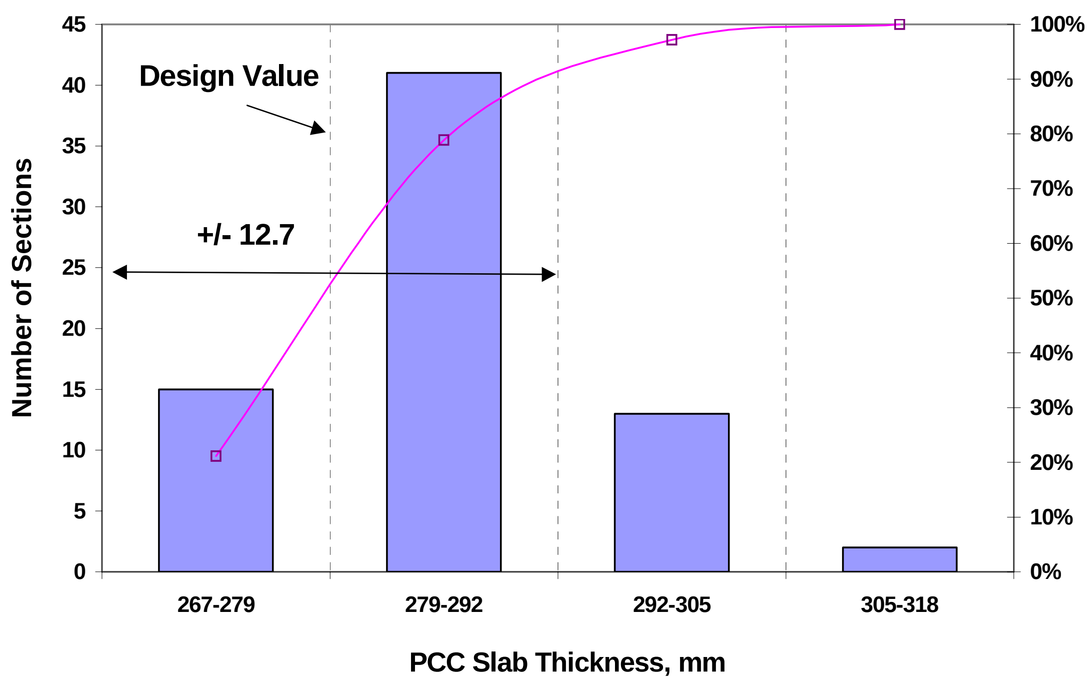


Figure 5. Frequency distribution of the mean PCC slab thickness for SPS-2 279-mm cells.

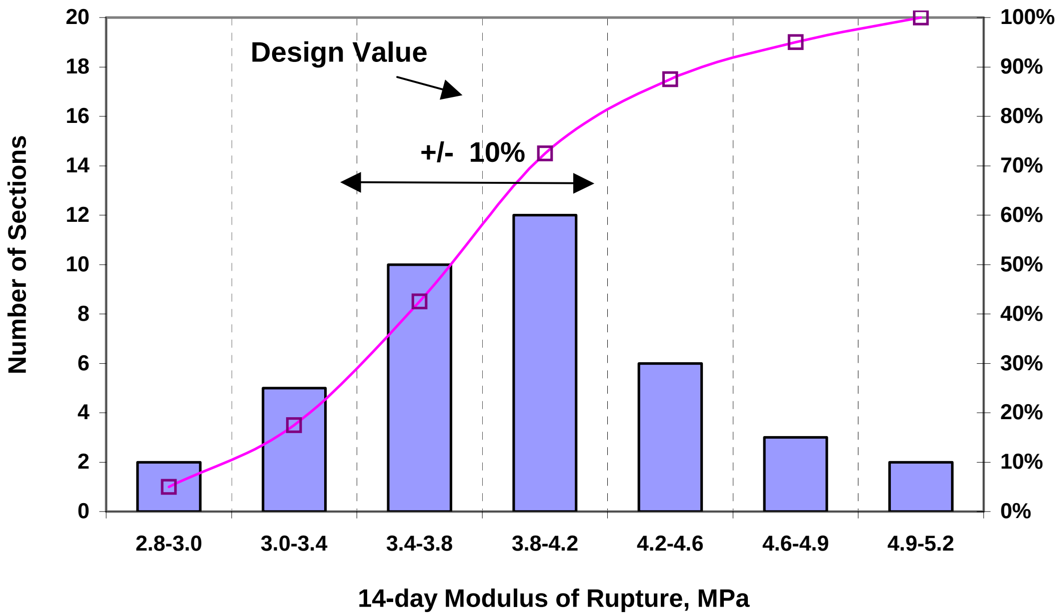


Figure 6. Frequency distribution of the 14-day modulus of rupture for SPS-2 3.8-MPa cells.

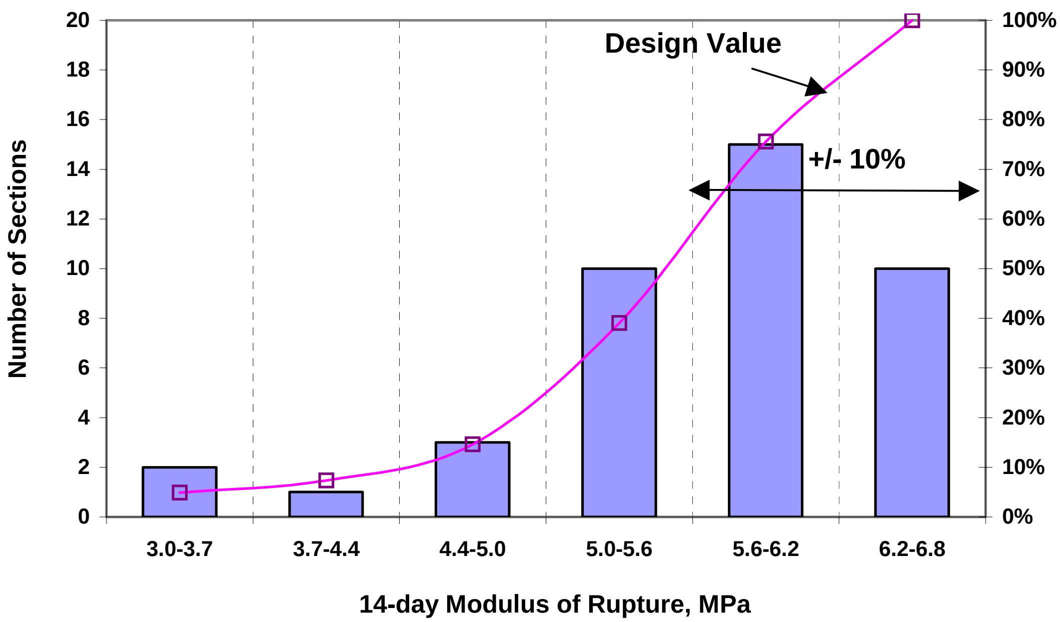
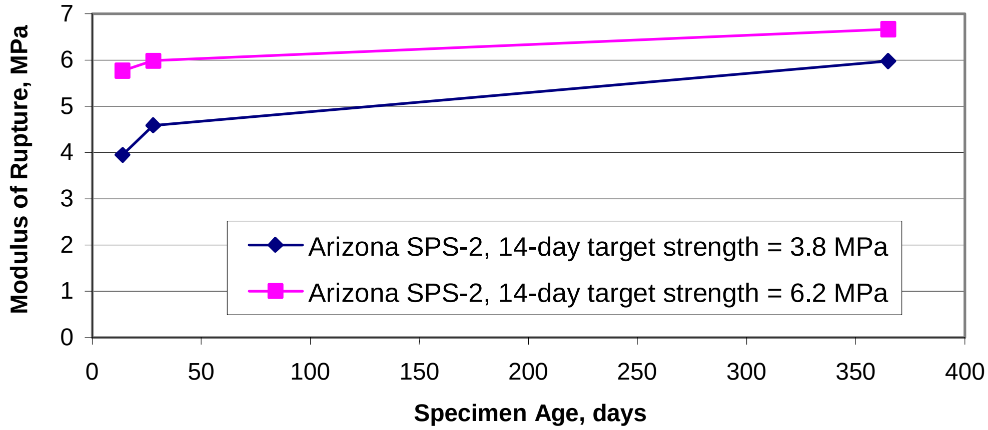
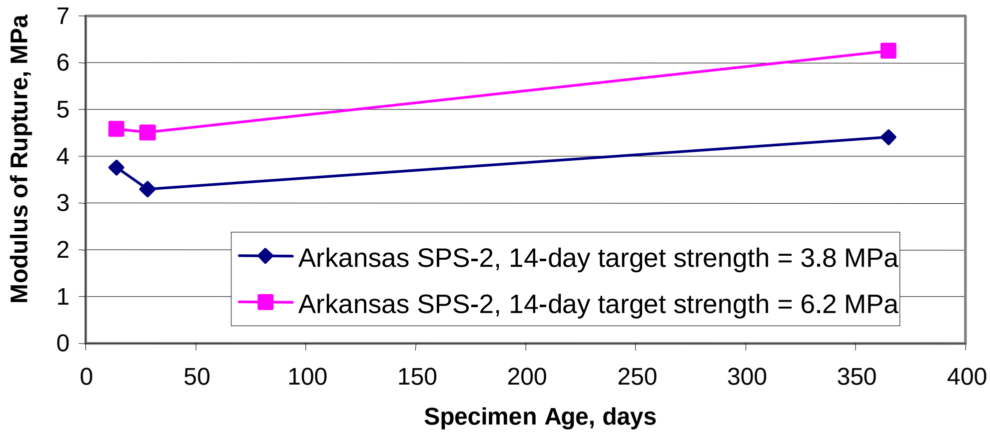


Figure 7. Frequency distribution of the 14-day modulus of rupture for SPS-2 6.2-MPa cells.





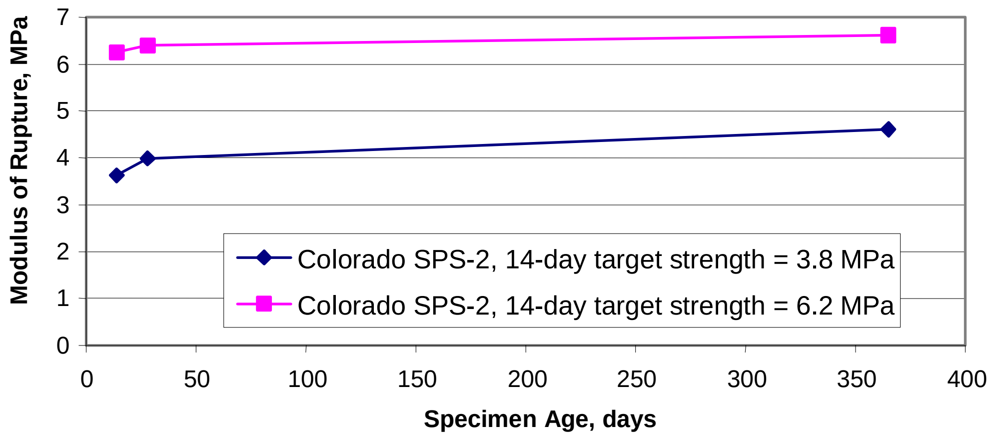
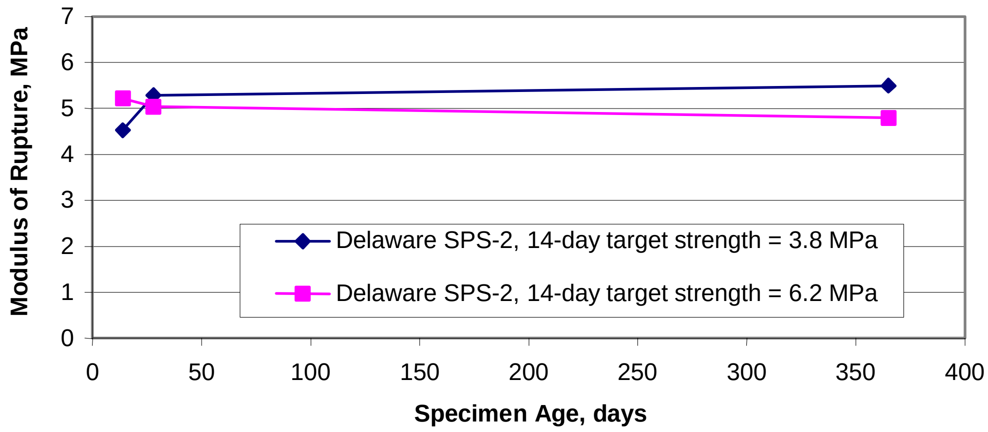
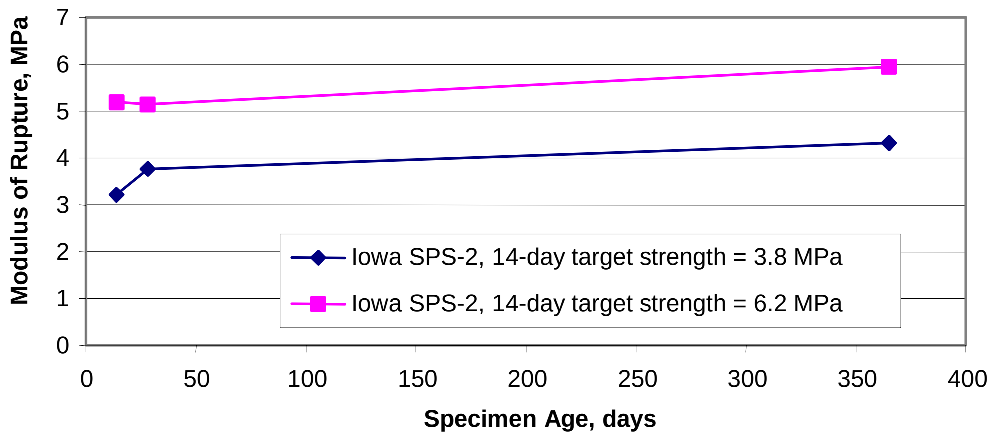


Figure 8. Time-series plot of modulus of rupture for SPS projects in Arizona, Arkansas, and Colorado.





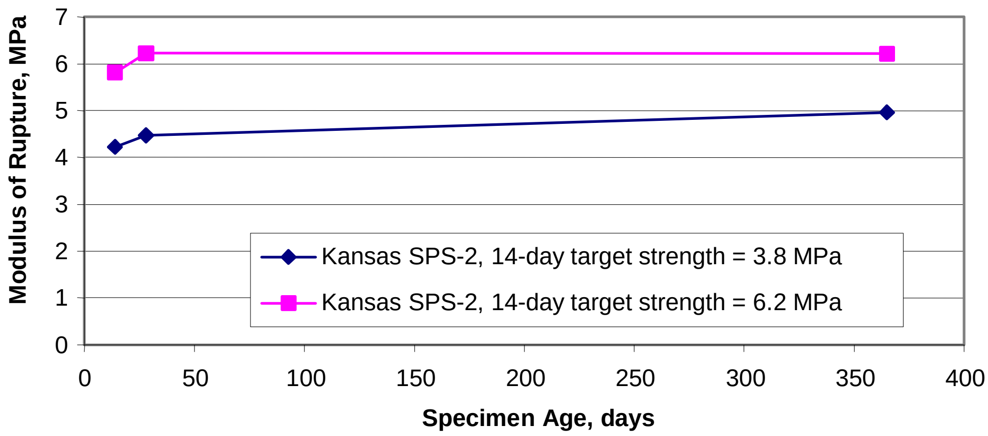


Figure 9. Time-series plot of modulus of rupture for SPS projects in Delaware, Iowa, and Kansas.

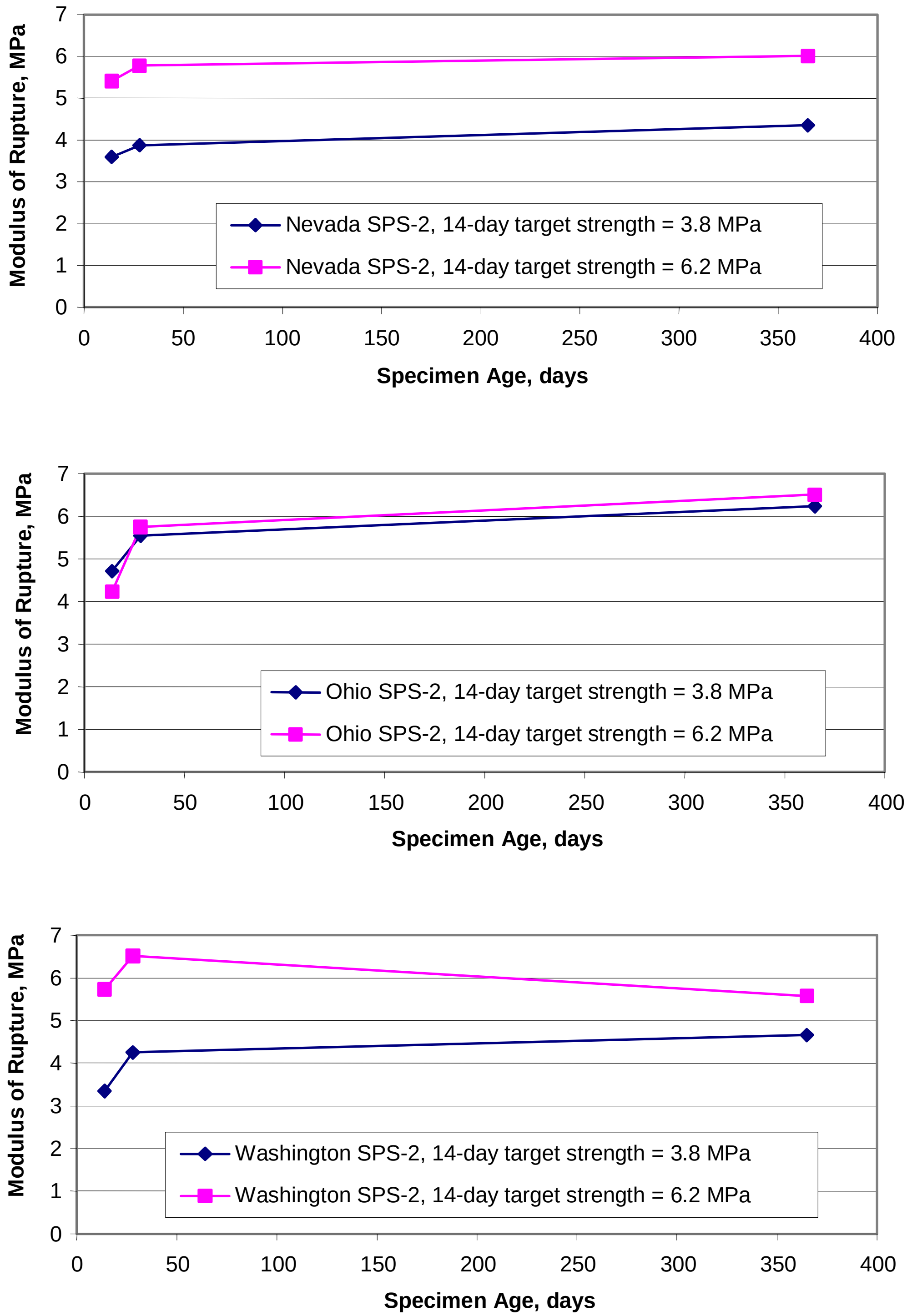


Figure 10. Time-series plot of modulus of rupture for SPS projects in Nevada, Ohio, and Washington.

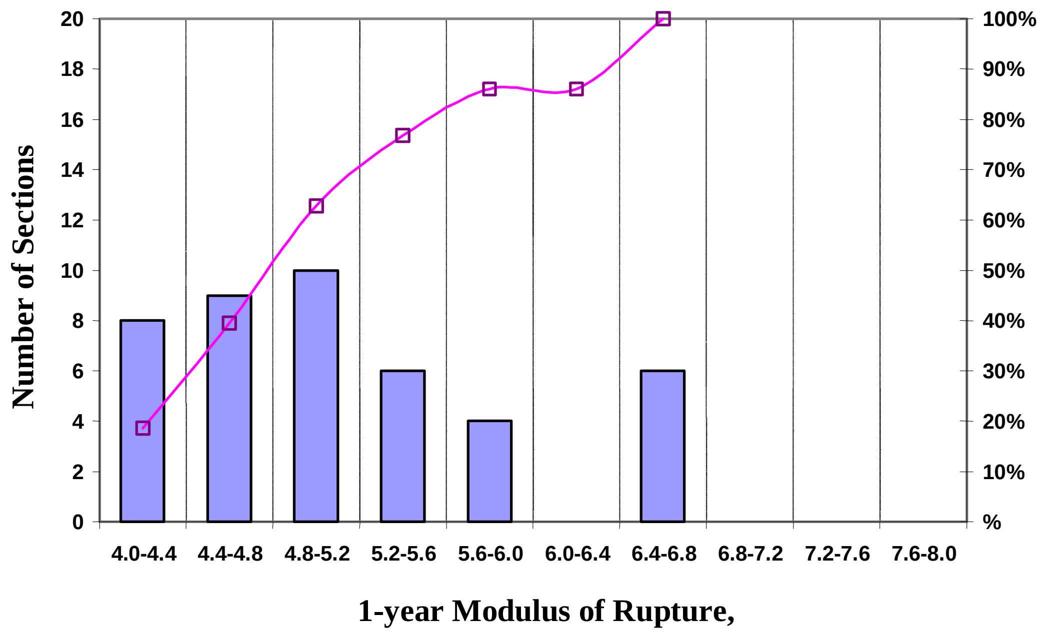


Figure 11. Frequency distribution of the 1-year modulus of rupture for 3.8-MPa cells.

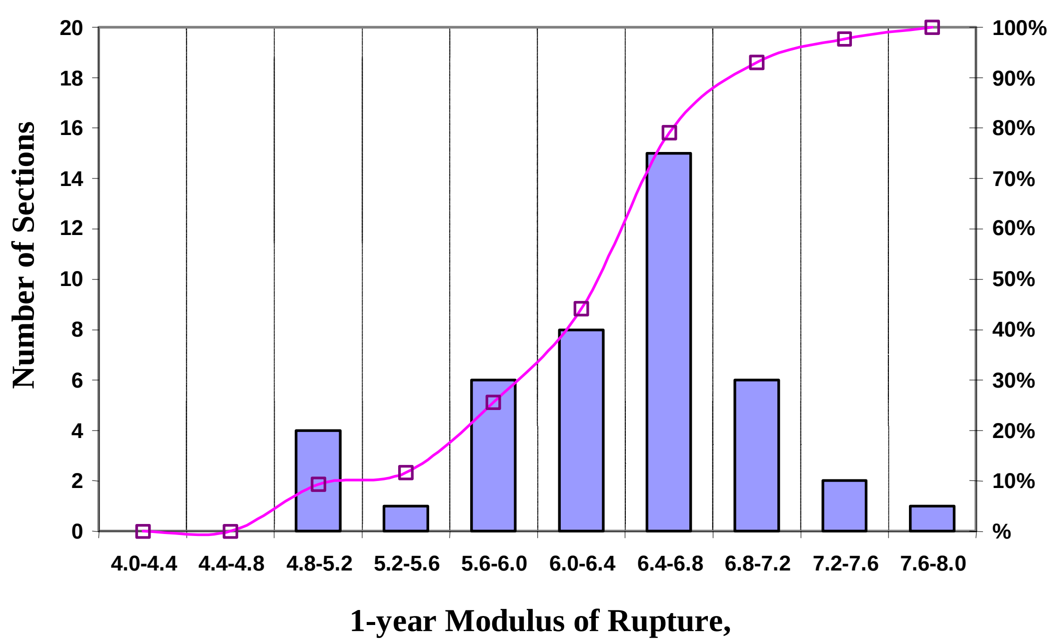


Figure 12. Frequency distribution of the 1-year modulus of rupture for 6.2-MPa cells.

The concrete strength factor should be examined in the future analysis to determine if it affects pavement performance. The actual strength measurements (instead of the target strength levels) should be used in any analysis due to the variation in strength of any given section.

Table 34. Summary statistics and *t*-test results for flexural strength data from all SPS-2 sites.

14-day Target Strength	Summary Statistics	Specimen Age	
		14-day	365-day
3.8 MPa	Mean	3.91	5.11
	Std. Dev	0.54	0.76
	No. specimens	40	43
6.2 MPa	Mean	5.62	6.34
	Std. Dev	0.79	0.68
	No. specimens	41	43
Mean difference		1.71	1.24
t-Stat		11.4	8.4
P(T<=t)		<0.0001	<0.0001

BASE LAYER

The following base types and thicknesses are specified in SPS-2 experiment design:

- DGAB—152 mm.
- LCB—152 mm.
- PATB—102 mm (on 102 mm DGAB).

IMS table TST_L05B was used to compare the designed versus constructed base types and thicknesses. The base types were confirmed to be constructed as designed for all the sections with base type information. For the base thicknesses, ±13-mm tolerance was used for the design ranges. The comparison results are provided in table 35. Twenty out of 131 SPS-2 sections (15 percent) have representative base thicknesses outside the design range, with 3 sections having base thicknesses below the design range and 17 above the design range.

Table 35. Designed versus mean constructed base thickness, mm.

State	Sections NOT Within Limit?	Section Number											
		0201, 0213	**0202, 0214**	**0203, 0215**	**0204, 0216**	**0205, 0217**	**0206, 0218**	**0207, 0219**	**0208, 0220**	**0209, 0221**	**0210, 0222**	**0211, 0223**	**0212, 0224**
		Design Value: 152 (140 to 165) mm								**102 (89 to 114) mm**			
		Base Type: DGAB				**LCB**				**PATB**			
AZ (0213-0224)	0	150	155	155	157	155	157	158	155	104	97	107	112
AR (0213-0224)	NA	–	–	–	–	–	–	–	–	–	–	–	–
CO (0213-0224)	1	150	150	152	147	160	157	155	160	94	114	107	**117**
DE (0201-0212)	2	158	165	155	160	140	155	**175**	152	**119**	97	94	94
IA (0213-0224)	4 (1 below)	155	160	147	150	165	163	**173**	**175**	99	**86**	89	**124**
KS (0201-0212)	1 (below)	155	150	144	**139**	152	152	150	152	99	94	107	112
MI (0213-0224)	1	155	147	157	149	157	**175**	160	147	107	107	104	109
NV (0201-0212)	4	150	147	145	157	**173**	**168**	**173**	**190**	102	94	104	–
NC (0201-0212)	5 (1 below)	**168**	152	142	**137**	165	**170**	142	150	**142**	**135**	91	109
ND (0213-0224)	2	145	158	163	155	165	**168**	165	**170**	112	97	104	102
OH (0201-0212)	0	155	147	157	147	157	150	160	160	102	104	99	112
WA (0201-0212)	1	147	165	**175**	150	155	157	155	165	99	97	99	89
WI (0213-0224)	NA	–	–	–	–	–	–	–	–	–	–	–	
Summary	**21 out of 131 sections (16%) are outside the design range, with 3 below and 18 above the limits.**												

Note: Bolded numbers are outside the design required range.

Table 36. Designed versus mean constructed lane width, m.

State	Sections NOT Within Limit?	Section Number											
		0201, 0214	0204, 0215	0205, 0218	0208, 0219	0209, 0222	0212, 0223	0202, 0213	0203, 0216	0206, 0217	0207, 0220	0210, 0221	0211, 0224
		Design Value: 3.66 m						**4.27 m**					
AZ (0213-0224)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
AR (0213-0224)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
CO (0213-0224)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
DE (0201-0212)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
IA (0213-0224)	**2**	3.66	3.66	3.66	**4.27**	3.66	3.66	4.27	**3.66**	4.27	4.27	4.27	4.27
KS (0201-0212)	**N/A**	–	–	–	–	–	–	–	–	–	–	–	–
MI (0213-0224)	**1**	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	**3.66**	4.27	4.27
NV (0201-0212)	–	3.66	3.66	3.66	3.66	3.66	-	4.27	4.27	4.27	4.27	4.27	4.27
NC (0201-0212)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
ND (0213-0224)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
OH (0201-0212)	–	-	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
WA (0201-0212)	–	3.66	3.66	3.66	3.66	3.66	3.66	4.27	4.27	4.27	4.27	4.27	4.27
WI (0213-0224)	**N/A**	–	–	–	–	–	–	–	–	–	–	–	–
Summary	**3 (of 131) sections are not as designated.**												

Note: Bolded numbers are outside the design required range.

DRAINAGE (EDGE DRAINS)

Edge drains were required for SPS-2 sections with PATBs. IMS table SPS_GENERAL contains drainage information for SPS-2 sections. Records were found for 130 SPS-2 sections in this table, and drainage designations were found to be as designed for all the sections.

LANE WIDTH

The SPS-2 experimental design specifies two levels for lane width: standard lane width of 3.66 m, and widened lane width of 4.27 m. The lane width information contained in IMS table SPS-GENERAL was examined for the designed versus constructed data, as shown in table 36. Three of the 131 SPS-2 sections (2 percent) have different lane width values from the design specifications (sections 19-0216 and 19-0219 in Iowa, and 26-0220 in Michigan).

SUMMARY

The experimental design specifications and the actual construction data of the key experimental factors for the SPS-2 project sites are summarized in table 37. As shown in the table, most SPS-2 sections meet the experimental design criteria for the large majority of the design factors. Most deviations from the experimental design are found for the concrete slab thickness and 14-day flexural strength.

A summary of experimental specifications versus as-constructed data for each SPS-2 project is provided in table 38.

Of the 13 SPS-2 projects, only the Wisconsin SPS-2 project does not have enough data in the IMS database to be evaluated. Eight projects can be characterized as good to excellent when comparing designed versus constructed data, while the remaining four projects are considered poor to fair.

Table 37. Designed versus constructed data summary for SPS-2 experiment.

Evaluation Element	Information Available (Total 13 sites, 155 sections)	Sites or Sections Not as Designed or Not Within Design Range
Climate Annual precipitation Freezing index	 12 sites (missing WI) 12 sites (missing WI)	 2 sites (KS and ND), both designated as in dry region but with over 508 mm annual precipitation. ✓ (All okay)
Traffic	8 sites (no data for AR, DE, ND, OH, WI)	2 sites (IA and WA). IA has annual ESAL of 56,406 in 1997. WA has annual ESAL of 819 in 1997.
14-day concrete flexural strength	11 sites (no 14-day flexural strength data for NC and ND)	For the 3.8-MPa design cells, 7 of the 11 sites (64%) have average flexural strength falling 10% outside of the design value (1 site 20% outside of the design value). For the 6.2-MPa design cells, 5 of the 11 sites (45%) have average flexural strength 10% outside of the design value (3 sites 20% outside).
Subgrade	13 sites (WI site information comes from SPS-2 layer table)	2 sites (CO and NV). CO site has 4 sections not as designed. NV site has 9 sections not as designed.
Slab thickness	13 sections (missing all 12 sections from WI site)	38 sections are outside the design ranges (design ±12.7 mm), with 4 sections below and 34 above the design range.
Base types and thickness	131 sections (missing all sections from KS and WI sites)	Base types are as designed. For base thickness, 21 sections are outside the design ranges (design ±12.7 mm), with 3 sections below and 18 above the design range.
Drainage	130 sections (missing all sections from KS and WI sites and 39-0201 in OH)	✓ (All okay)
Lane width	131 sections (missing all sections from KS and WI sites).	3 sections: 19-0216 and 19-0219 in Iowa, and 26-0220 in Michigan.

Table 38. Designed versus constructed SPS-2 PCC.

SPS-2 Sites	Climatic Zone	Traffic	Subg. Type	Flexural Strength, MPa		Average cell Slab Thick. mm		Base/ Long. Drain.	Lane Width	Comments
				3.8	6.2	203	279			
AZ	✓	✓	✓	✓	✓	✓	✓	✓	✓	Excellent
AR	✓	–	✓	✓	No (4.6)	✓	✓	✓	✓	Good
CO	✓	✓	No 4 sections	✓	✓	✓	✓	✓	✓	Good
DE	✓	✓	✓	No (4.5)	No (5.2)	No	No	✓	✓	Fair
IA	✓	No	✓	No (3.2)	No (5.2)	✓	No	✓	No- 2 sections	Poor
KS	Not Precip.	✓	✓	No (4.2)	✓	✓	✓	–	✓	Good
MI	✓	✓	✓	No (4.3)	✓	✓	✓	✓	No- 1 section	Good
NV	✓	✓	No 9 sections	✓	No (5.4)	No	✓	✓	✓	Fair
NC	✓	✓	✓	–	–	No	✓	✓	✓	Good—missing data
ND	Not Precip.	–	✓	–	–	✓	✓	✓	✓	Good—missing data
OH	✓	–	✓	No (4.7)	No (4.2)	✓	✓	✓	✓	Good—missing data
WA	✓	Not 1997	✓	No (3.3)	✓	No	✓	✓	✓	Fair
WI	–	–	–	No (4.4)	✓	–	–	–	–	Not enough data

Notes: ✓ = Indicates as-constructed value meets as-designed criteria.
– = No data

5. SPS-2 PROJECT STATUS SUMMARIES

This chapter summarizes key site information, pavement design factors, and monitoring data availability for each of the SPS-2 projects. For each SPS-2 project, the following are presented:

- General description of the pavement construction site and equipment installed.
- Key observations and deviations.
- Summary table of the key information and monitoring data availability.
- Project status summary.

The SPS-2 projects are presented in the following alphabetical order:

- Arizona (State code: 04).
- Arkansas (05).
- Colorado (08).
- Delaware (10).
- Iowa (19).
- Kansas (20).
- Michigan (26).
- Nevada (32).
- North Carolina (37).
- North Dakota (38).
- Ohio (39).
- Washington (53).
- Wisconsin (55).

ARIZONA SPS-2

The Arizona SPS-2 project site is located in the eastbound lanes of Interstate 10 (I-10) in southwestern Arizona, approximately 56 km west of Phoenix. I-10 is a rural interstate; in 1992 the AADT was 15,900. The initial annual ESALs was estimated at 1,052,626. The SPS-2 project was constructed as part of the rehabilitation of I-10. The typical pavement design consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 test sections were constructed on a portion of I-10 that is relatively straight and flat.

Bending plate WIM equipment was installed in the fall of 1993. Calibration was completed on January 24, 1994.

Construction of the SPS-2 site started in June 1993 with the removal of the existing pavement. Construction of individual test sections was completed and the area was opened to traffic on October 1, 1993.

All required core sections were constructed. Nine supplemental State test sections were constructed. Table 39 summarizes key project information and data available for all the sections.

Table 39. Arizona SPS-2 project summary.

Project level information and data availability **Construction date: 10/01/1993**

		Data Availability	*Average values*	*As planned?*
Climate - DNF	***CLM:***	17 years	FI: 0 °C days, Precip. 232 mm	**Yes**
	AWS:	6 years	FI: 0 °C day Precip. 199 mm	
Traffic	***WIM:***	2 to 3 years	1,052,625 ESALs/year (>200,000)	**Yes**
Subgrade type	Coarse-grained soil for all.		**As designed?**	**Yes**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	3.94	**Yes**
14-day MPa		6.2	5.77	**Yes**
PCC tests available	**On average 99% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							Monitoring data availability, No. of tests					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	201	203	AGG	No	4.27	**Yes**	4	6	3	1	3	**No**
0214	211		AGG	No	3.66	**Yes**	5	7	3	1	3	**No**
0215*	287	279	AGG	No	3.66	**Yes**	12	28	12	1	12	**No**
0216	284		AGG	No	4.27	**Yes**	5	6	2	1	2	**No**
0217	206	203	LCB	No	4.27	**Yes**	5	6	2	1	2	**No**
0218	211		LCB	No	3.66	**Yes**	5	6	2	1	2	**No**
0219	274	279	LCB	No	3.66	**Yes**	5	7	2	1	2	**No**
0220	287		LCB	No	4.27	**Yes**	5	6	3	1	3	**No**
0221	208	203	PATB	Yes	4.27	**Yes**	5	6	3	1	3	**No**
0222	***218***		PATB	Yes	3.66	**No**	5	6	3	1	3	**No**
0223	282	279	PATB	Yes	3.66	**Yes**	5	5	3	1	3	**No**
0224	272		PATB	Yes	4.27	**Yes**	5	6	3	1	3	**No**
Overall - Good, except for 0222							**Overall - Fair, late initial surveys.**					
Supplemental Sections - Total 9 sections, 7 PCC and 2 AC												
0260	216 mm AC on 102 mm DGAB						5	3	0	1	NA	**No**
0261	216 mm AC on 102 mm DGAB						5	5	0	1	NA	**No**
0262	203 mm undoweled JPC (3.8 MPa) on DGAB, 4.27 m lane						5	4	3	1	3	**Yes**
0263	203 mm undoweled JPC (3.8 MPa) on PATB, 4.27 m lane						5	1	3	1	3	**Yes**
0264	279 mm undoweled JPC (3.8 MPa) on PATB, 3.66 m lane						5	2	3	1	3	**Yes**
0265	279 mm undoweled JPC (3.8 MPa) on DGAB, 3.66 m lane						5	3	3	1	3	**Yes**
0266	318 mm doweled JPC (3.8 MPa) on BTB, 4.27 m lane						5	1	3	1	3	**Yes**
0267	279 mm doweled JPC (3.8 MPa) on BTB, 4.27 m lane						5	1	3	1	3	**Yes**
0268	203 mm doweled JPC (3.8 MPa) on BTB, 4.27 m lane						5	1	3	1	3	**Yes**
							Overall - Good, except for 0260, 0261.					

Note: ***** Indicates seasonal monitoring section(s)

Min. Req'd - Initial survey age less than 1 year and all long-term monitoring intervals less than 3 years for all monitoring types

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- American Association of State Highway and Transportation Officials (AASHTO) No. 57 coarse aggregate was utilized as the backfill material in the pavement base drain.
- A tapered transition of the PATB into the DGAB could not be achieved. This resulted in the PATB being placed against the DGAB at the end of section 040263.
- The class B geotextile supplied was not large enough to be wrapped around the PATB edge as per SHRP specifications. This unwrapped area could facilitate soil intrusion from the adjacent DGAB.
- Transverse drains were installed perpendicular to the roadway centerline instead of in a herringbone fashion.
- A 0.9-m-wide roll of filter fabric was placed with a 0.305-m-wide section under the median edge of the PATB. The remaining width was wrapped around the median edge of the PATB to prevent soil infiltration.
- Transverse cracking occurred in the LCB of sections 040217 through 040220 prior to placement of the PCC slab.
- Longitudinal joint tie bars were uncoated and were only 508 mm in length. SHRP specifications require epoxy-coated rebar at 762 mm in length. However, this is a dry no-freeze area.
- Paving was intermittently stopped in several of the test sections due to concrete unavailability, mix adjustments, and equipment failure.
- PCC segregation and/or slump variations occurred in several of the sections.
- The concrete temperature throughout construction generally ranged from 28 ºC to 31 ºC.

Project Status Summary

Overall, this project site is in excellent shape. The appendices to this report also contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Most key sections of this project were built adequately for the key experimental design variables. One exception is section 040222, which has excessively high slab thickness compared to the design value.
- Construction difficulties and deviations—Only relatively minor construction problems were noted in the construction report.

Data availability—Excellent overall.

- Site condition data—Very good.
- Key PCC materials testing data availability for core sections—Excellent, with 99 percent completed.
- Monitoring data availability— Excellent for all key monitoring data types for the core sections.

The Arizona SPS-2 site does not appear to exhibit significant problems that will cause difficulty in performance analysis.

ARKANSAS SPS-2

The Arkansas SPS-2 project site is located in the westbound lanes of I-30 in west central Arkansas, just to the west of the I-70/I-30 interchange. I-30 is classified as a rural interstate; in 1993 the AADT was 18,000, with 45 percent heavy trucks. The estimated initial annual ESALs is 2,069,550. The SPS-2 project was included in the reconstruction of I-30. Of the 12 test sections required for the SPS-2 project, 3 were located in original construction fill areas, 6 were located in original construction cut areas, and 3 sections were at grade. The typical roadway for this project consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside shoulder width of 1.22 m.

Construction of the SPS-2 site began in November 1993 with the removal of the existing pavement. Construction of individual test sections was completed on October 1, 1995, and the project site was opened to traffic on November 1, 1995.

All required core sections were constructed, and no supplemental State test sections were constructed at this site. Table 40 summarizes key project information and data available for all the sections.

Table 40. Arkansas SPS-2 project summary.

Project level information and data availability **Construction date:** **10/01/1995**

		Data Availability	*Average values*	*As planned?*
Climate - WNF	***CLM:***	17 years	FI: 38 °C days, Precip. 1,381 mm	**Yes**
	AWS:	0 years	NA	
Traffic	***WIM:***	NA	NA	**-**
Subgrade type	Fine-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	3.76	**Yes**
14-day MPa		6.2	4.59	**No**
PCC tests available	**On average 68% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	***188***	203	AGG	No	4.27	**No**	1	2	1	0	1	**No**
0214	211		AGG	No	3.66	**Yes**	1	1	1	0	1	**No**
0215	284	279	AGG	No	3.66	**Yes**	1	2	1	0	1	**No**
0216	277		AGG	No	4.27	**Yes**	1	2	1	0	1	**No**
0217	191	203	LCB	No	4.27	**Yes**	1	2	1	0	1	**No**
0218	***188***		LCB	No	3.66	**No**	1	2	1	0	1	**No**
0219	282	279	LCB	No	3.66	**Yes**	1	1	1	0	1	**No**
0220	272		LCB	No	4.27	**Yes**	1	2	1	0	1	**No**
0221	208	203	PATB	Yes	4.27	**Yes**	1	1	1	0	1	**No**
0222	213		PATB	Yes	3.66	**Yes**	1	1	1	0	1	**No**
0223	277	279	PATB	Yes	3.66	**Yes**	1	1	1	0	1	**No**
0224	277		PATB	Yes	4.27	**Yes**	1	1	1	0	1	**No**
Overall - Good, except for 0213 and 0218.							**Overall - Fair, late initial surveys.**					
Supplemental Sections - None constructed.												

Note: * Indicates seasonal monitoring section(s)
Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations and deviations were noted in the project construction and deviation report:

- On section 050208, the vibrators of the slip-form paver became entangled with the dowel basket assembly at station 2+50. This caused the augers of the paver to stop operating. The contractor removed and replaced the affected concrete and dowel basket assembly.
- Longitudinal joints were not sealed until early 1997. By this time, pumping was evident through these joints.

Project Status Summary

This project site was well constructed overall, but is in fair-to-poor shape mainly due to very limited performance monitoring data, traffic data, PCC materials testing data, and AWS data. There are no supplemental sections at this site. The following summarizes the status of this project:

- Designed versus constructed—Good, except that the mean 14-day flexural strength value (4.6 MPa) for the 6.2 MPa design cell was significantly below the design value.
- Construction difficulties and deviations—Minor.
- Data availability—Poor overall.
    - Site condition data—Poor. Traffic data and AWS data not available at the time of analysis.
    - Key PCC materials testing data availability for core sections—Fair, with 68 percent completed.
    - Monitoring data availability—Poor, with only one survey for many key monitoring data types.

The Arkansas SPS-2 site does not appear to have significant problems that will cause difficulty in performance analysis if construction, traffic, AWS, and monitoring data become available.

COLORADO SPS-2

The Colorado SPS-2 project site is located in the eastbound lanes of I-76 in central Colorado, approximately 32 km northeast of Denver. I-76 is a rural interstate; in 1988 the AADT was 8,400, with 16 percent heavy trucks. The initial annual ESALs are estimated at 347,646. Six SPS-2 test sections were included in the phase 1 section of I-76, which was constructed on a new alignment (sections 080217, 080220, 080221, 080222, 080223, and 080224). The remaining six sections (sections 080213, 080214, 080215, 080216, 080218, and 080219) were located within the phase 2 section of I-76, which was being reconstructed. The 136th Street interchange bisects this SPS-2 site. However, no appreciable difference in traffic loading is expected due to the presence of this interchange.

All sections are on a tangent. The vertical grade averages +1.4 percent in the direction of traffic. Six sections were located in a cut (sections 080217, 080218, 080219, 080220, 080223, and 080224), while all other sections were located on fills. The typical roadway for this project consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m.

Construction of the SPS-2 site began on July 1, 1993, and was completed on November 1, 1993. The phase 1 work (new alignment) was opened to traffic on October 7, 1993. The phase 2 work (I-76 reconstruction) began after phase 1 was opened to traffic. Phase 2 sections were opened to traffic on January 5, 1994.

All required core sections were constructed. One supplemental State test section was also constructed. Table 41 summarizes key project information and data availability for all the sections.

Table 41. Colorado SPS-2 project summary.

Project level information and data availability | **Construction date:** **10/01/1993**

		Data Availability	*Average values*	*As planned?*
Climate - DF	***CLM:***	17 years	FI: 327 ^{o}C days, Precip. 370 mm	**Yes**
	AWS:	5 years	FI: 394 ^{o}C days, Precip. 344 mm	
Traffic	***WIM:***	3 years	347,646 ESALs/year (>200,000)	**Yes**
Subgrade type	8 sections coarse-grained, 4 sections fine-		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	3.63	**Yes**
14-day MPa		6.2	6.25	**Yes**
PCC tests available	**On average 94% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	***221***	203	AGG	No	4.27	**No**	3	5	2	2	3	**Not quite**
0214	213		AGG	No	3.66	**Yes**	3	5	2	2	3	**Not quite**
0215	290	279	AGG	No	3.66	**Yes**	3	5	2	2	3	**Not quite**
0216	***300***		AGG	No	4.27	**No**	3	5	2	2	3	**Not quite**
0217	***218***	203	LCB	No	4.27	**No**	3	3	2	2	3	**Not quite**
0218	196		LCB	No	3.66	**Yes**	3	4	2	2	3	**Not quite**
0219	282	279	LCB	No	3.66	**Yes**	3	4	2	2	3	**Not quite**
0220	282		LCB	No	4.27	**Yes**	3	3	2	2	3	**Not quite**
0221	211	203	PATB	Yes	4.27	**Yes**	3	4	2	2	3	**Not quite**
0222	***221***		PATB	Yes	3.66	**No**	3	4	2	2	3	**Not quite**
0223	***300***	279	PATB	Yes	3.66	**No**	3	4	3	2	3	**Not quite**
0224	***297***		PATB	Yes	4.27	**No**	3	4	2	2	3	**Not quite**
Overall - Fair. Six sections outside design range.							**Good, except for late initial faulting surveys.**					
Supplemental Sections - 1 PCC section.												
0259	279 mm JPC (4.5 MPa) on subgrade and 3.66 m lanes.						3	3	2	2	3	**Not quite**
							Overall - Good.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range.
For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- Subgrade pumping occurred on several phase 1 sections due to rainy weather and a locally high water table. Pumping did not occur on the phase 2 sections. The embankment in these test sections consisted of stable fill material including pulverized concrete and asphalt capped by a fine sand layer.

- Several of the PATB sections contained too many fines in the mix. This resulted in removal and replacement of the mat in section 080221.
- Due to its high plasticity, the 6.2 MPa concrete mix was harder to work with than the 3.8 MPa mix.
- While paving section 080218, equipment and concrete delivery problems (muddy haul roads) caused several work stoppages. The dowel bars and basket assembly were torn up at station 141+50 but not replaced.
- No major problems occurred during construction of the DGAB and LCB layers.

Project Status Summary

Overall, this project site is in excellent shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good except for the following:
    - Subgrade type varies, with four sections having a fine-grained subgrade soil rather than a coarse-grained soil, as designed.
    - Mean slab thickness values for six sections are more than 12.7 mm higher than the design value.
- Construction difficulties and deviations—Minor. Several PATB sections contained too many fines in the mix.
- Data availability—Very good overall.
    - Site condition data—Meets experimental conditions.
    - Key PCC materials testing data availability for core sections—Excellent, with 94 percent completed.
    - Monitoring data availability—Very good, except that the initial survey for faulting data was very late, 2.5 years after the construction.

The Colorado SPS-2 site does not appear to exhibit significant problems that will cause difficulty in performance analysis.

DELAWARE SPS-2

The Delaware SPS-2 project site is located in the southbound lanes of U.S. 113 in central Delaware, between Milford and Georgetown. U.S. 113 is a rural principal arterial; in 1989 the AADT was 10,708, with 10 percent heavy trucks. The initial annual ESALs are estimated at 234,000. The SPS-2 project was included in the addition of two southbound lanes to an initial two-lane roadway. The two new southbound lanes were separated from the existing northbound lanes by a 7.92- to 12.8-m-wide median. Route S-625 and another access road bisect this SPS-2 site. The traffic from these routes is expected to have little impact on the SPS-2 site. There is no record of WIM installation at this site.

The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The subgrade consists of sand and silty sand. Test sections were constructed on shallow cuts or fills. The cut sections ranged up to 1.52 m in depth. Several wetland areas exist adjacent to the mainline pavement, where the water table is at or near the surface for an extended time period.

This project was completed and opened to traffic on May 1, 1996. All required core sections were constructed, and two supplemental State test sections were constructed. Table 42 summarizes key project information and data available for all the sections.

Table 42. Delaware SPS-2 project summary.

Project level information and data availability **Construction date:** **05/01/1996**

	Data Availability	*Average values*	*As planned?*
Climate - WF	***CLM:*** 17 years	FI: 103 °C days, Precip. 1,144 mm	**Yes**
	AWS: 0 years	N/A	
Traffic	***WIM:*** 0 years	N/A	**N/A**
Subgrade type	Coarse-grained soil for all.	**As designed?**	**Varies**
	Design value	*Actual Averages*	*Within 10%?*
Flexural strength	3.8	4.53	**No**
14-day MPa	6.2	5.22	**No**
PCC tests available	**On average 91% completed for core sections.**		

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0201	211	203	AGG	No	3.66	**Yes**	6	3	3	0	4	**Yes**
0202	***224***		AGG	No	4.27	**No**	6	2	2	0	3	**Yes**
0203	***297***	279	AGG	No	4.27	**No**	6	2	2	0	3	**Yes**
0204	279		AGG	No	3.66	**Yes**	6	2	2	0	3	**Yes**
0205	***234***	203	LCB	No	3.66	**No**	6	2	2	0	3	**Yes**
0206	***226***		LCB	No	4.27	**No**	6	2	2	0	3	**Yes**
0207	287	279	LCB	No	4.27	**Yes**	6	2	2	0	3	**Yes**
0208	***307***		LCB	No	3.66	**No**	6	2	2	0	3	**Yes**
0209	208	203	PATB	Yes	3.66	**Yes**	6	3	3	0	4	**Yes**
0210	211		PATB	Yes	4.27	**Yes**	6	2	2	0	3	**Yes**
0211	***300***	279	PATB	Yes	4.27	**No**	6	2	2	0	3	**Yes**
0212	***315***		PATB	Yes	3.66	**No**	6	2	2	0	3	**Yes**
Overall - Fair. Seven sections outside design range.							**Overall - Excellent.**					
Supplemental Sections - 2 PCC sections.												
0259	254 mm JPC (20.7 Mpa f'c) on 203 mm DGAB; 3.66 m lane; steel dowels						6	2	2	0	3	**Yes**
260	254 mm JPC (20.7 Mpa f'c) on 203 mm DGAB; 3.66 m lane; plastic dowels						6	2	2	0	3	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)
Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- During paving of the LCB layer, depressions in the subgrade occurred during stoppage of the paver. Transverse shrinkage cracks developed in the LCB layer prior to PCC paving, and some of these shrinkage cracks developed in the depression areas.

- During removal of the cracked PCC (Delaware Department of Transportation (DelDOT) mix) in section 100205, construction personnel noticed that some of the shrinkage cracks in the LCB had reflected through the PCC. Some areas of the LCB had bonded to the PCC; however, the underside areas of most of the slabs was smooth and clean, which is representative of an unbonded condition. The second application of a curing compound immediately before PCC paving appears to have been effective in debonding the PCC, except where surface depressions and irregularities existed in the underlying LCB.
- A longitudinal crack had developed by October 13, 1995, in section 100207 at 457 mm from the centerline and parallel to the centerline. This crack was near the underlying construction joint in the LCB. This crack was cored on October 26, 1995, and was not found to extend the full depth of the concrete pavement. This crack may be attributable to late sawing of the longitudinal joint, since this section was paved on June 28 but longitudinal joint sawing was not performed until July 3.
- Before removing the concrete in sections 100205, 100206, and 100207, coring of transverse and longitudinal shrinkage cracks was performed. These cracks were found to extend either entirely or partially through the PCC but not into the underlying LCB. No bond was found to occur between the PCC and the underlying LCB.
- Removal of some of the DGAB occurred in sections 100201 and 100202 with removal of the cracked JPC. Additional DGAB was added before JPC repaving in the test sections to create a uniform mat. The DGAB was then reshaped and recompacted.
- After full-depth repair was completed, several additional cracks developed during the winter of 1995–1996 in section 100205 (LCB). Two additional cracks developed in section 100201 (DGAB), but no additional cracking developed in section 100209 (PATB).
- Full-depth repair of these cracks was performed from April 18 to 19, 1996. At this time, 17 fine transverse cracks were noticed in various test sections. These cracks occurred at the edge of the pavement and only extended a few meters into the slab panel.
- No. 57 Stone was used as the edge drain backfill instead of PATB.
- Transverse joint sealant reservoirs were sawn to 19 mm width and 38.1 mm depth, while the longitudinal joints were sawn to a width of 6.4 mm and a depth of 13 mm. The transverse joints in all test sections except sections 100206, 100202, and 100210 were sealed with neoprene seals. The transverse joints in the remaining sections and all longitudinal joints were sealed with hot poured rubberized asphalt material.

The following deviations from SPS-2 guidelines were noted in the project deviation report:

- Eight of the 12 test sections contained partial shallow cuts, but the cut subgrades had to meet Type A borrow specifications. Those cut subgrades that did not meet the Type A specifications were excavated to receive 305 mm of Type A borrow (with prior approval) (sections 100201, 100203, 100204, 100205, 100207, 100208, 100209, and 100211).

- A transverse construction joint was placed within section 100212.
- The longitudinal joint was sawn five days after the concrete placement (sections 100211, 100203, and 100207).
- Bases did not extend the full width of the shoulder (with prior approval).
- Neoprene was used in the transverse joints (hot poured in three sections where the joints were rough), and hot-poured rubberized asphalt was used in the longitudinal joint.
- No joint sealant was used between the mainline concrete pavement and the asphalt shoulder.
- Joints were sealed in 1996 and in the second construction season.
- The road was opened to construction traffic before joint sealing.
- Tensile strength testing equipment was not obtained until after July 25, 1995, so cylinders and cores requiring this test prior to this time were missed.
- 365-day cores will not be obtained until the northbound lanes have been rehabilitated and opened to traffic.
- Samples have been sent to the laboratories, but the materials testing data available to date is not complete.
- For sections 100212, 100210, 100211, and 100209:
  - Edge drains were not located at the outside edges of the shoulder.
  - Edge drain outlets were spaced at distances greater than 76 m.
- Construction guideline deviations:
  - 3.8-MPa flexural strength concrete was not used on sections 100207, 100203, and 100211; 20.7 MPa compression strength tested concrete was used instead.
  - 3.8 MPa flexural strength concrete used on sections 100201, 100205, and 100209 was removed and replaced with 4.5 MPa flexural strength.
  - Sections 100202 and 100206 were placed with 6.2 MPa flexural strength 6.5-bag mix. Concrete was later removed and replaced with 6.2 MPa flexural 7.5-bag mix.
  - Profile index was greater than 158 mm/km for section 100205. This section is scheduled for diamond grinding.

Project Status Summary

Overall, this project site is in fair shape. The supplemental sections also contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Fair, with the following problems:
  - Subgrade type varies, with four sections having a fine-grained subgrade soil type and not as designed.

    - Mean slab thickness values for six sections are more than 12.7 mm higher than the design value.
    - PCC strength was not within design guidelines.
- Construction difficulties and deviations—Moderate.
- Data availability—Very good overall except for traffic WIM.
    - Site condition data—Very good except for change in subgrade.
    - Key PCC materials testing data availability for core sections—Excellent, with 91 percent completed.
    - Monitoring data availability—Very good, except that initial survey for faulting data was very late, 2.5 years after the construction.
    - Traffic data—No WIM data are available for this SPS site.

Data from the Delaware SPS-2 site will require special analysis techniques to adjust for various design factors that were not constructed as planned.

IOWA SPS-2

The Iowa SPS-2 project site is located in the northbound lanes of U.S. 65 in central Iowa, northeast of Des Moines. U.S. 65 is an urban/principal arterial; in 1994 the AADT was 17,400, with 16 percent trucks. The estimated initial ESALs for the section is approximately 600,000. The SPS-2 project was included in the relocation of U.S. 65 in both the northbound and southbound lanes.

The typical roadway consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 sections were constructed on a portion of U.S. 65 that included both tangent and superelevated sections. All sections were constructed on a tangent except sections 190215 and 190216. These sections were constructed on the high side of a horizontal curve with a superelevation rate of 2.5 percent. Vertical grades throughout the project area range from -2.6 percent to +2.2 percent. Sections 1902215 through 190220 were constructed on fill sections ranging from near 0 to 11.58 m in height. Sections 190221 through 190224 were constructed on cut sections ranging from 0.91 to 7.01 m.

WIM and AVC equipment were installed in June 1995 on U.S. 65, approximately 1.61 km north of the junction with IA-163 (state highway). Reconstruction was completed in 1994 during a period of relatively wet weather conditions. The project site was opened to traffic on December 1, 1994.

All required core sections were constructed. An additional supplemental State test section was also constructed. Table 43 summarizes key project information and data available for all the sections.

Table 43. Iowa SPS-2 project summary.

Project level information and data availability **Construction date:** **08/01/1994**

		Data Availability	*Average values*	*As planned?*
Climate - WF	***CLM:***	17 years	FI: 580 °C days, Precip. 900 mm	**Yes**
	AWS:	3 years		
Traffic	***WIM:***	1 year	56,400 ESALs/year (<200,000)	**No**
Subgrade type	Fine-grained soil for all.		**As designed? -**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	3.22	**No**
14-day MPa		6.2	5.19	**No**
PCC tests available	**On average 98% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	216	203	AGG	No	4.27	**Yes**	5	2	2	1	3	**Yes**
0214	213		AGG	No	3.66	**Yes**	5	1	1	1	2	**Yes**
0215	***300***	279	AGG	No	3.66	**No**	5	1	2	1	3	**Yes**
0216	***295***		AGG	No	4.27	**No**	5	1	2	1	3	**Yes**
0217	196	203	LCB	No	4.27	**Yes**	5	2	2	1	3	**Yes**
0218	208		LCB	No	3.66	**Yes**	5	2	2	1	3	**Yes**
0219	284	279	LCB	No	3.66	**Yes**	5	1	2	1	3	**No**
0220	290		LCB	No	4.27	**Yes**	5	2	2	1	3	**Yes**
0221	***239***	203	PATB	Yes	4.27	**No**	5	2	2	1	3	**Yes**
0222	211		PATB	Yes	3.66	**Yes**	4	1	1	1	2	**Yes**
0223	***297***	279	PATB	Yes	3.66	**No**	5	1	1	1	3	**Yes**
0224	***295***		PATB	Yes	4.27	**No**	5	1	1	1	2	**Yes**
Overall - Fair, five sections outside design range.							**Good, but late initial faulting survey for 0219.**					
Supplemental Sections - 1 PCC section.												
0259	279 mm JPC; 4.27 m wide lane						5	1	1	1	2	**Yes**
							Overall - Good.					

Note: * Indicates seasonal monitoring section(s)
Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- Underground structures were located in 6 of the 13 sections (sections 190213, 190214, 190215, 190217, 190219, and 190221). These ranged from a 0.61-m diameter concrete pipe at 2.44 m below profile grade to a 2.44-m by 3.05-m concrete pipe at 12.19 m below profile grade.
- The contractor removed at least 0.3 m of geotextile from the longitudinal edge drains due to the low permeability of the geotextile.

- The boundaries of section 190222 were relocated after construction because dowel bars with the wrong diameter were placed in the initial boundaries of this section.
- Four sections (190215, 190216, 190212, and 190223) had concrete thicknesses in excess of SPS-2 tolerances. These thicknesses ranged from 8 to 23 mm above the desired thickness.
- During placement of the PCC pavement for test section 190222, incorrect dowel baskets were placed. This area was removed, and the section location was shifted to avoid the replaced pavement area. Because of misinterpretation of guidelines, the section numbers were revised. The correct numbers should be from 13 through 24. This revision was done after most of the sampling and testing and data collection had been completed.

Project Status Summary

Overall, this project site is in fair shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Poor. The key deviations are listed below:
    - Mean slab thickness values for five sections are more than 12.7 mm higher than the design value.
    - Mean 14-day flexural strength values are more than 10 percent below the design values.
    - Lane width values of the two sections are wrong. This might be a data entry error. A feedback report was submitted.
- Construction difficulties and deviations—Moderate.
- Data availability—Excellent overall.
    - Site condition data—Good, except for deficient traffic data and missing AWS data.
    - Key PCC materials testing data availability for core sections—Excellent, with 98 percent completed.
    - Monitoring data availability—Very good, except that the initial survey for faulting data for section 190219 was very late.

Data from the Iowa SPS-2 site will require special analysis techniques to adjust for various design factors that were not constructed as planned.

KANSAS SPS-2

The Kansas SPS-2 project site is located in the westbound lanes of I-70 in central Kansas, east of Abilene. I-70 is a rural interstate; the estimated AADT is 13,750, with 21.4 percent trucks. The initial annual ESALs in the design lane are estimated at 639,131. The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 project was included in the reconstruction of I-70. The existing pavement was concrete. The SPS-2 test sections were constructed on a tangent section of I-70 with vertical grades ranging from -2.48 percent to +2.11 percent. All test sections were constructed on fills.

An onsite weather monitoring station had not been installed before completion of the project. However, installation was scheduled to occur by 1994. A Toledo Model 9430™ high-speed WIM system was installed onsite.

Construction of this SPS-2 project was completed on July 1, 1992. The project site was opened to traffic on August 1, 1992. All required core sections were constructed. A supplemental State test section was also constructed. Table 44 summarizes key project information and data available for all the sections.

Table 44. Kansas SPS-2 project summary.

Project level information and data availability **Construction date:** **07/01/1992**

		Data Availability	*Average values*	*As planned?*
Climate - DF	***CLM:***	17 years;	FI: 259 °C days, Precip. 819 mm	**No**
	AWS:	4 years	FI: 254 °C days, Precip. 698 mm	
Traffic	***WIM:***	1 year	639,131 ESALs/year (>200,000)	**Yes**
Subgrade type	Fine-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	4.23	**No**
14-day MPa		6.2	5.81	**Yes**
PCC tests available	**On average 66% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0201	196	203	AGG	No	3.66	**Yes**	7	5	2	2	2	**No**
0202	***188***		AGG	No	4.27	**No**	8	6	2	2	2	**No**
0203	282	279	AGG	No	4.27	**Yes**	8	7	2	2	2	**No**
0204	287		AGG	No	3.66	**Yes**	8	5	2	2	2	**No**
0205	198	203	LCB	No	3.66	**Yes**	8	5	2	2	2	**No**
0206	201		LCB	No	4.27	**Yes**	8	6	2	2	2	**No**
0207	287	279	LCB	No	4.27	**Yes**	8	5	2	2	2	**No**
0208	279		LCB	No	3.66	**Yes**	8	5	2	2	2	**No**
0209	216	203	PATB	Yes	3.66	**Yes**	8	5	2	2	1	**No**
0210	211		PATB	Yes	4.27	**Yes**	8	5	2	2	2	**No**
0211	282	279	PATB	Yes	4.27	**Yes**	8	5	2	2	2	**No**
0212	277		PATB	Yes	3.66	**Yes**	8	5	2	2	2	**No**
Overall - Excellent, except for 0202.							**Good, except for deficient faulting data.**					
Supplemental Sections - 1 PCC section.												
0259	305 mm doweled JPC (4.1 MPa mix) on 152 mm stabilized base and 152 mm modified Flyash subgrade, 3.66 m lane.						7	2	2	2	1	**No**
							Overall - Good, except for faulting.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- The LTPP SPS Project Construction Report indicates that the PATB was difficult to place. The contractor placed this material too thick in several of the test sections. The excess was removed with a trimmer. During initial construction operations, the PATB deformed when compacted. This problem was resolved as the contractor gained experience with this material.
- Underground structures were present in sections 200204, 200208, 200209, 200210, 200211, and 200212. Median drains were present in several test sections. However, these drains were at least 1.52 m below the pavement surface.
- Weather station was not installed until 1996 (4 years after construction was complete).
- The DOT staff experienced many problems with the sampling and testing requirements.
- An insufficient number of cores was specified in the sampling and testing plan.
- Field cores of the PATB could not be collected. Therefore, it was impossible to conduct tests on samples CA 01, 02, 03, 05, 47, 48, 51, and 54.
- Traffic monitoring data was only submitted for 1993 (78-day period).
- The first distress survey was not performed until April 1993.
- Vertical curves (-2.48 to +2.11 percent grade) exist within the limits of the test sections.
- Several underground structures exist within the limits of the test sections.
- Many of the sections contain 457-mm median drains. These drains are located >1.5 m below the surface of the pavement.
- Sections 200204, 200208, 200209, and 200211 have box culverts located within their limits.
- Section 200210 contains a transverse drain for the PATB.
- Section 200211 contains a median drain ±1.2 m below the surface of the pavement.
- Several sections have concrete pavement thicknesses that exceed the allowable tolerance of ±6.4 mm (200209 = +13 mm; 200210 = 7.6 mm; 200211 = -25.4 mm; 200212 = -48 mm; and 200204 = +10 mm).
- Construction was delayed due to an extremely wet and rainy season.
- The contractor experienced many problems while trying to place the PATB. Trimming was often required to obtain the desired thickness.
- Type C fly ash was used to help dry up and stabilize the subgrade.
- Section 200201 required one full-depth repair and two partial-depth patches in 1995.
- Section 200204 required two partial-depth patches in 1995.

Project Status Summary

Overall, this project site is in good shape. The appendices to this report contain a significant amount of monitoring data (except for faulting surveys). The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
    - Annual precipitation value is higher than the planned dry climatic zone.
    - Mean slab thickness values for five sections are more than 12.7 mm higher than the design value.
    - Mean 14-day flexural strength values for the lower strength concrete are more than 10 percent above the design values.
    - Construction difficulties and deviations—Relatively minor.
- Data availability—Excellent overall.
    - Site condition data—Good, except for deficient traffic data and missing AWS data.
    - Key PCC materials testing data availability for core sections—Poor, with only 66 percent completed.
    - Monitoring data availability—Good, except for deficient faulting data.

The Kansas SPS-2 site exhibits some significant problems (data unavailable and construction deviations) that will cause difficulty in performance analysis.

MICHIGAN SPS-2

The Michigan SPS-2 project site is located in the northbound and southbound lanes of U.S. 23 in southeastern Michigan, approximately 16 km west of Toledo. U.S. 23 is a rural principal arterial; in 1989 the AADT was 35,000, with 22 percent heavy trucks. The initial year ESALs was estimated at 1,346,045. The SPS-2 project was included in the reconstruction of 9.7 km of U.S. 23 in both the northbound and southbound lanes. Consear Road, a low-volume county road, bisects this SPS-2 site. Traffic counts taken in the northbound lanes reveal that traffic south of this interchange is 7 percent higher than traffic north of this interchange (AVC data only).

The roadway typically consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 sections were constructed on a portion of U.S. 23 that is relatively straight and flat. Vertical grades throughout the project area range from 0.00 percent to +0.55 percent. Most of the sections were constructed on shallow fills. However, sections 260214, 260218, and 260219 were constructed on deeper fills. All sections except 260214, 260218, and 260219 were constructed in tangent sections. Section 260214 was constructed in a superelevation transition area, while sections 260218 and 260219 were constructed on a full superelevation of 0.037 m/m.

WIM and AVC equipment were installed on U.S. 23 south of Consear Road. Only AVC equipment was installed north of Consear Road.

Reconstruction of U.S. 23 began in April 1993 with removal of the existing pavement layers. Construction of the subgrade progressed from mid-May through mid-June, and placement and compaction of the embankment was completed by mid-June. Undercuts were completed in Sections 260216, 260022, and 260223 due to unstable soil conditions revealed during proofrolling. These undercuts were 11 m wide and 0.3 m deep, but only extended for a partial length of each section. The undercuts were backfilled with embankment borrow clay. Base and subbase layer construction began by mid-June and was completed by mid-September 1993. Concrete paving commenced on September 13, 1993 (excluding control section) and was completed on September 21, 1993. The project site was opened to traffic in November 1993.

All required core sections were constructed, and one supplemental State test section was constructed. Table 45 summarizes key project information and data available for all the sections.

Table 45. Michigan SPS-2 project summary.

Project level information and data availability **Construction date:** **11/01/1993**

		Data Availability	*Average values*	*As planned?*
Climate - WF	***CLM:***	17 years;	FI: 382 °C days, Precip. 866 mm	**Yes**
	AWS:	3 years	FI: 140 °C days, Precip. 871 mm	
Traffic	***WIM:***	5 years	1,346,045 ESALs/year (>200,000)	**Yes**
Subgrade type	Fine-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	4.27	**No**
14-day, MPa		6.2	6.71	**Yes**
PCC tests available	**On average 82% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	***218***	203	AGG	No	4.27	**No**	9	5	5	1	5	**Yes**
0214	***226***		AGG	No	3.66	**No**	9	4	4	1	5	**Yes**
0215	284	279	AGG	No	3.66	**Yes**	9	4	4	1	5	**Yes**
0216	290		AGG	No	4.27	**Yes**	9	5	4	1	5	**Yes**
0217	***216***	203	LCB	No	4.27	**Yes**	7	4	4	1	4	**Yes**
0218	***180***		LCB	No	3.66	**No**	8	3	3	1	3	**Yes**
0219	277	279	LCB	No	3.66	**Yes**	9	4	4	1	5	**Yes**
0220	282		LCB	No	4.27	**Yes**	8	4	4	1	5	**Yes**
0221	208	203	PATB	Yes	4.27	**Yes**	8	5	5	1	6	**Yes**
0222	213		PATB	Yes	3.66	**Yes**	9	4	4	1	5	**Yes**
0223	279	279	PATB	Yes	3.66	**Yes**	9	4	4	1	5	**Yes**
0224	284		PATB	Yes	4.27	**Yes**	9	4	4	1	5	**Yes**
Overall - Good, except for 0213, 0214, 0218.							**Overall - Excellent.**					
Supplemental Sections - 1 PCC section.												
0259	267 mm JRC on 102 mm OGDB on 76 mm aggregate base.						9	4	4	1	4	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

For sections 260213 through 260220, the moisture content of the subgrade was not maintained within the range of 85 to 120 percent of the optimum moisture content. Moderate-to-severe desiccation cracks (up to 50.8 mm width and 254 mm depth) developed in the subgrade, which was compacted dry of optimum since the completed embankment was exposed to hot and dry weather conditions before construction of the overlying base or subbase layers. This occurred on all sections except those constructed with PATB. Cracking did not occur on PATB sections because the DGAB was placed soon after completion of the embankment.

Michigan DOT required the contractor to scarify the desiccated subgrade sections and recompact severely desiccated subgrade to SPS-2 requirements.

- The following are some specific observations:
    - The DGAB in section 260221 segregated but was reworked in the worst areas to minimize segregation.
    - The DGAB was not kept uniformly wet prior to paving in sections 260213 through 260216.
    - Rutting of the PATB in the outside shoulder area occurred due to construction traffic. This also caused deformation of the edge drains.
    - Cracking of the LCB occurred in the outside shoulder area of sections 260217 and 260220 due to construction traffic.
    - Bonding of the LCB to the PCC was noticed in two of the sampling cores. This was not evident in other LCB/PCC cores.
    - The LCB in sections 260218 through 260220 had a slump less than 25.4 mm.
    - The LCB was milled between inside and outside lane placements in section 260218. The surface grooves were filled with grout and the spray cure was reapplied.
    - The PCC in sections 260214, 260219, and 260220 had an air content less than 5 percent.
    - The PCC did not meet SHRP requirements for 3.8 MPa and 6.2 MPa flexural strengths at 14 days. The flexural testing indicated that the 3.8 MPa mix and the 6.2 MPa mix had similar flexural strengths at 365 days after placement. The flexural strengths at 1 year respectively averaged 6.1 MPa for the 3.8 MPa design mix and 6.6 MPa for the 6.2 MPa design mixes.
    - Several pavement layers were out of specifications with respect to thickness tolerances.
    - Elevation measurements were not taken on all embankment layers.
    - Longitudinal joint seal damage at the lane/shoulder joint occurred in several test sections by 1994. The entire length of this joint in all test sections (except control section) failed by 1995. No damage was evident in the control section, which was constructed with tied concrete shoulders.
    - Pumping was observed at the longitudinal joint and transverse joints in most of the sections constructed with a DGAB and all of the sections constructed with an LCB (undrained). No pumping was observed in PATB (drained) sections.
    - Low-severity transverse joint sealant damage occurred in several test sections by 1995.
    - Structural distresses including pumping, transverse joint faulting, transverse cracking, longitudinal cracking, and corner breaks occurred in section 260218.
    - The following data collection deviations were noted:
        1. Early in the project, elevation measurements were not taken at the required embankment layer locations.
        2. Elevation measurements have only a fair-to-poor correlation with the measured pavement thickness.

3. Fresh concrete samples of section 260259 were not obtained within the limits of that test section.
4. The AWS was not installed until 1996. Until then, climatic data was obtained from the Toledo, OH, airport, which is about 16 km from the project site.
5. Splitspoon samples were used in place of shelby tubes, due to the hardness of the subgrade and the presence of gravel and cobblestone.

- The following site location guideline deviations were noted:
    1. Section 260259 (control section) has tied concrete shoulders, neoprene transverse joint seals, and hot-poured rubberized asphalt longitudinal joint seals.
    2. A low-volume road intersects the test sections near the middle of the project site, which causes a minor difference in traffic volumes and loading across the test sections. To help monitor this difference, the WIM was located to the south of the interchange and an AVC was placed on each side of the interchange.
    3. Sections 260214, 260218, and 260219 are located on deep fills and on a superelevated horizontal curve (1°).
    4. Vertical curves, with grades ranging from -0.81 to +0.55 percent, exist within the test section limits.
    5. A 762-mm concrete culvert exists ±267 m below the top of the pavement surface in section 260224.
- The following construction guideline deviations were noted:
    1. The moisture content of the compacted subgrade was not within the range of 85 to 120 percent of optimum for sections 260213, 260214, 260215, 260216, 260217, 260218, 260219, and 260221. This resulted in severe desiccation cracking of the subgrade that the contractor had to rework.
    2. The DGAB layer in section 2602121 segregated. The contractor reworked and improved the area, but some segregation still existed.
    3. The surface of the DGAB was not kept uniformly moist in sections 260213, 260214, 260215, and 260216.
    4. The underdrain filter fabric did not extend the minimum of 0.305 m under the pavement.
    5. Traffic was allowed on the outside shoulder of the PATB, which resulted in rutting of 13 to 44.5 mm.
    6. A transverse construction joint in the LCB was located within the test section limits.
    7. The paving equipment was allowed to operate on the outside shoulder area of the LCB, which resulted in longitudinal cracking in sections 260217 and 260220.
    8. Fresh LCB samples revealed a slump lower than the 25.4 mm limit for sections 260218, 260219, and 260220.
    9. Cores of the LCB in section 260218 did not satisfy the thickness tolerance of design ±13 mm.

10. Fresh concrete samples revealed a slump lower than the 25.4 mm limit in sections 260215 and 260219, and air contents lower than the 5.0 percent limit in sections 260214, 260219, and 260220.
11. The 14-day flexural strength requirements were not satisfied.
12. Cores of the concrete in sections 260213, 260214, 260217, 260218, 260222, and 260259 did not satisfy the tolerance of design ±6.4 mm.
13. Test sections 260216, 260222, and 260223 had to be undercut because of unstable subgrade material.
14. Test sections 260221 and 260224 had areas of unstable subgrade but were not undercut.
15. The contractor had problems maintaining the proper elevation for the PATB because the paver was not using a stringline.
16. Test section 260213 was diamond ground to remove a "must-grind" bump.

Project Status Summary

Overall, this project site is in good shape except for the concrete strength (the high and low strengths are practically the same after 1 year). The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
  - Mean 14-day flexural strength value for 3.8 MPa cell is more than 10 percent below the design value.
- Construction difficulties and deviations—Moderate.
- Data availability—Excellent overall.
  - Site condition data—Very good.
  - Key PCC materials testing data availability for core sections—Good, with 82 percent completed.
  - Monitoring data availability—Excellent.

Data from the Michigan SPS-2 site will require special analysis techniques to adjust for various design factors that were not constructed as planned.

NEVADA SPS-2

The Nevada SPS-2 project site is located in north central Nevada, approximately 8 km west of Battle Mountain, in the outer eastbound lane of I-80. The SPS-2 sections extend from station 1596+65 to station 64+50 (milepost 223.7). The initial annual ESALs were estimated to be 812,944.

The construction work on this segment of I-80 consisted of removing the existing AC surfacing, cement-treated base (CTB), DGAB, and embankment. The original subgrade was stabilized with lime, and the embankment was replaced. The SHRP structural sections were then placed on top of the embankment. The terrain surrounding the test sections is generally flat with minimal ground cover.

All required core sections were constructed. One supplemental State section was also constructed. Table 46 summarizes key project information and data available for all the sections.

Table 46. Nevada SPS-2 project summary.

Project level information and data availability **Construction date:** **08/01/1995**

		Data Availability	*Average values*	*As planned?*
Climate - DF	***CLM:***	17 years;	FI: 276 °C-days, Precip. 222 mm	**Yes**
	AWS:	5 years	FI: 181 °C-days, Precip. 249 mm	
Traffic	***WIM:***	1 year	812,944 ESALs/year (>200,000)	**Yes**
Subgrade type	2 sections coarse-grained, 9 sections fine-grained.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	3.60	**Yes**
14-day MPa		6.2	5.41	**No**
PCC tests available	**On average 97% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							Data availability, No. of tests					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0201	***234***	203	AGG	No	3.66	**No**	4	6	2	1	2	**Yes**
0202	208		AGG	No	4.27	**Yes**	2	4	2	1	2	**Yes**
0203	***302***	279	AGG	No	4.27	**No**	4	3	2	1	2	**Yes**
0204*	***300***		AGG	No	3.66	**No**	8	15	8	1	8	**Yes**
0205	***216***	203	LCB	No	3.66	**No**	4	5	2	1	2	**Yes**
0206	198		LCB	No	4.27	**Yes**	2	3	2	1	2	**Yes**
0207	277	279	LCB	No	4.27	**Yes**	4	2	2	1	2	**Yes**
0208	279		LCB	No	3.66	**Yes**	4	2	2	1	2	**Yes**
0209	***226***	203	PATB	Yes	3.66	**No**	4	6	3	1	3	**Yes**
0210	***257***		PATB	Yes	4.27	**No**	4	5	3	1	3	**Yes**
0211	287	279	PATB	Yes	4.27	**Yes**	4	4	2	1	2	**Yes**
0212	NA - Section taken out of the SPS-2 study from the beginning.											
Overall - Fair, five sections outside design range.							**Overall - Excellent.**					
Supplemental Sections - 1 PCC section.												
0259	267 mm JPC on 38 mm leveling course, 27.6 MPa ± 20% 14-day compressive strength						4	2	2	1	2	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- This project site was constructed over an existing section of highway, and the removal of the existing pavement structure was required. When this was performed, it was discovered that the subgrade, which was sandy silt, was out of specifications for NDOT subgrade material. This required the lime stabilization of the top 0.3 m of subgrade material.
- After this stabilization, embankment material was placed and compacted. FWD testing on the embankment showed that sections 320201, 320205, 320207, and 320209 had significantly higher deflections than the other sections.
- The DGAB was placed on 8 of the 12 sections. The material was placed in either one or two lifts, depending on the design thickness. Sections 320201 and 320209 were found to have high variations in deflections during FWD testing, and section 320203 had deflections in the first 38.1 m, while the other five sections were more consistent.
- As per the SPS-2 experiment design, four sections received a 102-mm PATB. Edge drains were constructed on these sections utilizing a geotextile and open-graded rock placed in trenches.
- As per the SPS-2 experiment design, four sections had a 152-mm LCB placed directly on the embankment. The LCB was placed in one 12.19-m-wide pass and no joints were sawed. All sections except 320206 exhibited extensive cracking within 2 weeks of paving.
  The PCC consisted of three different mixes. Section 320259 was the State standard mix, six sections were constructed using a 3.3 MPa mix, and six were constructed using a 5.2 MPa mix. The typical SPS-2 project has six 3.8 MPa and six 6.2 MPa mixes, but it wasn't possible to reach the 6.2 MPa target using local materials, so the target strengths were revised. A number of other problems that occurred during PCC paving are detailed in appendix B.
- The majority of the problems with the PCC paving came as a result of the mixes being significantly different than those typically used by the paving crew. This was especially true for the 5.2 MPa mix. Proof of this fact is that section 320259, which was the State standard mix, had none of the problems with shrinkage cracks and tearing that were so common for the majority of the project. The primary conclusion that can be made on the basis of this project is that trying to perform nonstandard construction can cause significant problems. It is highly unlikely that the majority of the test sections will last anywhere close to their design lives.

Project Status Summary

Overall, this project site is in poor shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
  - Subgrade type varies, and nine sections have a different subgrade type than was designed.
  - Mean 14-day flexural strength value for the 6.2 MPa cell is 5.41 MPa, or more than 10 percent below the design value.
- Construction difficulties and deviations—Major problems that led to extensive early cracking.
- Data availability—Excellent overall.
  - Site condition data—Very good.
  - Additional traffic data are needed.
  - Key PCC materials testing data availability for core sections—Excellent, with 97 percent completed.
  - Monitoring data availability— Excellent.

Data from the Nevada SPS-2 site will require special analysis techniques to adjust for various design factors that were not constructed as planned. In particular, slab cracking will not be able to be evaluated in comparison with other sites; this may cause other problems that will complicate performance analysis.

NORTH CAROLINA SPS-2

The North Carolina SPS-2 project site is located in the southbound lanes of U.S. 23, near Lexington, NC. U.S. 52 is a rural principal arterial with an AADT of 23,500 to 26,100 (1994) and 13 percent heavy trucks. The annual ESALs was estimated at 750,902. The SPS-2 project was included in the construction of 7.8 km of U.S. 52 in both the northbound and southbound lanes. U.S. 64 bisects this SPS-2 site. All test sections except section 370204 are located north of the U.S. 64 interchange. This section will be monitored with AVC equipment to determine if the traffic south of U.S. 64 is different from traffic to the north of U.S. 64.

The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside width of 1.22 m. The shoulders were constructed with econocrete instead of the SPS-2 required flexible bituminous design. The majority of SPS-2 test sections were constructed on tangent sections with slight grades. Sections that include a 203-mm PCC slab were constructed as add-on lanes adjacent to the mainline travel lane. This parallel roadway section was constructed through some deep cuts and high embankments.

Seasonal monitoring sensors, strain gauges, and linear variable differential transducers were installed on several test sections. Reconstruction began in 1992 with earthwork grading.

All required core sections were constructed. Two supplemental State test sections were also constructed. Table 47 summarizes key project information and data available for all the sections.

Table 47. North Carolina SPS-2 project summary.

Project level information and data availability **Construction date:** **07/01/1994**

		Data Availability	*Average values*	*As planned?*
Climate - WNF	***CLM:***	17 years	FI: 47°C days, Precip. 1,151 mm	**Yes**
	AWS:	5 years	FI: 67°C days, Precip. 1,199 mm	
Traffic	***WIM:***	5 years	750,902 ESALs/year (>200,000)	**Yes**
Subgrade type	Fine-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	N/A	**N/A**
14-day MPa		6.2	N/A	**N/A**
PCC tests available	**On average 37% completed for core sections.**			**Fair**

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0201*	***229***	203	AGG	No	3.66	**No**	10	25	9	1	10	**Not quite**
0202	***259***		AGG	No	4.27	**No**	8	2	1	1	2	**Not quite**
0203	284	279	AGG	No	4.27	**Yes**	8	2	1	1	2	**Not quite**
0204	284		AGG	No	3.66	**Yes**	7	2	1	1	1	**Not quite**
0205*	203	203	LCB	No	3.66	**Yes**	8	3	1	1	2	**Not quite**
0206	213		LCB	No	4.27	**Yes**	8	2	1	1	2	**Not quite**
0207	***295***	279	LCB	No	4.27	**No**	8	2	1	1	2	**Not quite**
0208*	284		LCB	No	3.66	**Yes**	7	3	1	1	1	**Not quite**
0209	***218***	203	PATB	Yes	3.66	**No**	8	5	5	1	6	**Not quite**
0210	213		PATB	Yes	4.27	**Yes**	8	2	1	1	2	**Not quite**
0211	290	279	PATB	Yes	4.27	**Yes**	8	2	1	1	2	**Not quite**
0212*	277		PATB	Yes	3.66	**Yes**	8	3	1	1	1	**Not quite**
Overall - Fair, four sections outside design range.							**Overall - Good, except for late initial surveys.**					
Supplemental Sections - 2 PCC sections												
0259	254 mm JPC on 102 mm PATB on 25.4 mm AC on 203 mm lime-stabilized subgrade.						7	2	1	1	2	**Yes**
0260	279 mm JPC on 2.54 cm AC on 127 mm BTB on 203 mm cement-treated subgrade.						8	2	1	1	2	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- Edge drains were located at a 0.61-m offset from the pavement edge, rather than the SPS-2 required 2.4-m offset. Stone was used instead of PATB as trench backfill.
- Econocrete shoulders were approved for use instead of asphalt shoulders.
- The DGAB extended only 0.61 m into the shoulder from the pavement edge.
- Dowel bars (25.4 mm diameter) were utilized on sections, which included a 203-mm PCC. The LCB was constructed to extend only 0.61 m into the shoulder from the pavement edge.
- Cracks developed in the LCB layer in several sections before construction of the PCC. These cracks were covered with tar paper prior to PCC paving. Several of these cracks reflected through the PCC. Consequently, some of these slabs were repaired.

Project Status Summary

Overall, this project site is in fair shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
  - Mean PCC slab thickness for the 203-mm cell is more than 12.7 mm above the design value.
  - No data are available to evaluate the mean 14-day flexural strength adequacy.
- Construction difficulties and deviations—Minor data availability—Fair overall.
  - Site condition data—Very good.
  - Key PCC materials testing data availability for core sections—Poor, with only 37 percent completed.
  - Monitoring data availability—Very good. Some sections have late initial surveys.

NORTH DAKOTA SPS-2

The North Dakota SPS-2 project is located in the eastbound lanes of I-94 in eastern North Dakota, west of Fargo. I-94 is a rural interstate; in 1996 the AADT was 8,310, with 12 percent trucks. The initial annual ESALs in the design lane are estimated at 246,000.

The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 project was included in the reconstruction of a concrete pavement that included 229-mm concrete on 76-mm aggregate base on 152- to 229-mm aggregate subbase. The SPS-2 test sections were constructed on a portion of I-94 that is very flat and relatively straight. All sections except North Dakota supplemental sections 380260 and 360261 were constructed on tangent sections.

Several delays were encountered during subgrade preparation, due to the presence of extremely wet clayey soils. Construction of individual test sections was completed on October 1, 1994, and the pavements were opened to traffic on November 1, 1994.

All required core sections were constructed. Six supplemental State test sections were constructed. Table 48 summarizes key project information and data available for all the sections.

Table 48. North Dakota SPS-2 project summary.

Project level information and data availability **Construction date:** **11/01/1994**

		Data Availability	*Average values*	*As planned?*
Climate - DF	***CLM:***	17 years	FI: 1,313°C days, Precip. 545 mm	**Yes**
	AWS:	5 years	FI: 1,162°C days, Precip. 534 mm	
Traffic	***WIM:***	0 years	N/A	**N/A**
Subgrade type	Fine-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	N/A	**N/A**
14-day MPa		6.2	N/A	**N/A**
PCC tests available	**On average 81% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	208	203	AGG	No	4.27	**Yes**	2	2	2	1	3	**Not quite**
0214	201		AGG	No	3.66	**Yes**	2	2	1	1	2	**Not quite**
0215	279	279	AGG	No	3.66	**Yes**	2	2	1	1	1	**Not quite**
0216	284		AGG	No	4.27	**Yes**	2	2	1	1	1	**Not quite**
0217	201	203	LCB	No	4.27	**Yes**	2	1	2	1	3	**Not quite**
0218	201		LCB	No	3.66	**Yes**	2	1	1	1	2	**Not quite**
0219	277	279	LCB	No	3.66	**Yes**	2	1	1	1	2	**Not quite**
0220	277		LCB	No	4.27	**Yes**	2	1	1	1	2	**Not quite**
0221	206	203	PATB	Yes	4.27	**Yes**	2	1	2	1	3	**Not quite**
0222	208		PATB	Yes	3.66	**Yes**	2	1	1	1	2	**Not quite**
0223	282	279	PATB	Yes	3.66	**Yes**	2	1	1	1	2	**Not quite**
0224	274		PATB	Yes	4.27	**Yes**	2	1	1	1	2	**Not quite**
Overall - Excellent.							**Good, except for late initial profile surveys.**					
Supplemental Sections - 6 PCC sections.												
0259	254 mm doweled JPC (ND mix) on 203 mm salve with skewed joints and 3.66 m lanes.						2	1	1	1	2	**Yes**
0260	279 mm doweled JPC (ND mix) on DGAB with skewed joints and 4.27 m lanes.						2	2	1	1	1	**Yes**
0261	279 mm undoweled JPC (3.8 MPa MR) on DGAB with skewed joints and 3.66 m lanes.						2	2	1	1	1	**Yes**
0262	279 mm undoweled JPC on LCB with skewed joints (various lengths) and 4.27 m lanes.						2	1	1	1	2	**Yes**
0263	279 mm undoweled JPC on PATB with random skewed joints and 3.66 m lanes.						2	1	1	1	2	**Yes**
0264	279 mm undoweled JPC on PATB with skewed joints and 4.27 m lanes.						2	1	1	1	2	**Yes**
							Overall - Good.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- The LCB was difficult to place until the mix design was changed to increase the strength of this layer. The thickness tolerances on four core SPS-2 sections were not met (sections 380217, 380218, 380219, and 380220).
- Transverse cracks developed in section 380217. These cracks reflected through the 203-mm PCC within 5 days after construction of the PCC.
- The PATB deformed when compacted.
- The subgrade in section 380218 was unstable and should have been undercut. This caused some initial frost heave, but the condition has corrected itself.
- The layer thickness for the following sections contained deviations:
  - 380217—LCB not within the 0.012 m design tolerance, based only on rod and level.
  - 380218 and 380220—LCB not within the 0.012 m design tolerance, based on both rod and level, and core results.
  - 380219—LCB not within the 0.012 m design tolerance, based on core results.
- LCB was difficult to place, so the mix was made stronger than the guidelines.
- PATB was difficult to roll due to its fluid-like characteristics and its short length requirements.

- On section 380217, the transverse cracks in the LCB reflected through to the 203 mm of PCC pavement.
- Sections 380260 and 380261 were built on slight superelevations just after the on-ramp from Casselton.

Project Status Summary

Overall, this project site is in good shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good (however, PCC strength is unknown). The key deviations are listed below:
  - Annual precipitation value is higher than the planned dry climatic zone. Mean slab thickness values for five sections are more than 12.7 mm higher than the design value.
  - No data are available to evaluate the mean 14-day flexural strength adequacy.
- Construction difficulties and deviations—Relatively minor.
- Data availability—Good overall.

- Site condition data—Fair. Traffic data not available at the time of analysis.
- Key PCC materials testing data availability for core sections—Good, with 81 percent completed.
- Monitoring data availability—Very good, except for the initial surveys of the longitudinal profile.

The North Dakota SPS-2 site does not appear to exhibit significant problems that will cause difficulty in performance analysis.

OHIO SPS-2

The Ohio SPS-2 project site is located in the northbound lanes of U.S. 23 in central Ohio, approximately 48 km north of Columbus. U.S. 23 is a rural principal arterial; in 1994 the AADT was 20,210, with 12 percent trucks. The initial annual ESALs are estimated at 600,000. The roadway typically consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside asphalt shoulder width of 1.22 m. The SPS-2 test sections were constructed on a portion of U.S. 23 that is relatively straight and flat.

Permanent WIM equipment consisting of weigh plates was mounted in each lane of U.S. 23. Additional instrumentation was installed in the SPS-2 project area to collect environmental data, including temperatures of individual pavement layers and moisture freeze/thaw conditions of the subbase and subgrade layers. The load-response monitoring instrumentation that was installed included strain, deflection, and pressure gauges.

Construction started in the fall of 1994 with the subgrade preparation. Individual test sections were completed by October 1995, and the project was open to traffic on October 1, 1996. All required core sections were constructed. Seven supplemental State test sections were also constructed. Table 49 summarizes key project information and data available for all the sections.

Table 49. Ohio SPS-2 project summary.

Project level information and data availability **Construction date:** **09/01/1996**

		Data Availability	*Average values*	*As planned?*
Climate - WF	***CLM:***	17 years;	FI: 375 ^{o}C days, Precip. 972 mm	**Yes**
	AWS:	6 years	FI: 121 ^{o}C days, Precip. 730 mm	
Traffic	***WIM:***	0 years	N/A	**N/A**
Subgrade type	Fine-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	4.72	**No**
14-day MPa		6.2	4.23	**No**
PCC tests available	**On average 96% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							Data availability, No. of tests					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0201	201	203	AGG	No	3.66	**Yes**	4	5	1	1	2	**Yes**
0202	211		AGG	No	4.27	**Yes**	4	4	1	1	2	**Yes**
0203	277	279	AGG	No	4.27	**Yes**	4	3	1	1	2	**Yes**
0204*	282		AGG	No	3.66	**Yes**	8	15	1	1	3	**Yes**
0205	203	203	LCB	No	3.66	**Yes**	4	4	1	1	2	**Yes**
0206	201		LCB	No	4.27	**Yes**	4	3	1	1	2	**Yes**
0207	282	279	LCB	No	4.27	**Yes**	4	3	1	1	2	**Yes**
0208	279		LCB	No	3.66	**Yes**	4	3	1	1	2	**Yes**
0209	206	203	PATB	Yes	3.66	**Yes**	4	5	1	1	2	**Yes**
0210	203		PATB	Yes	4.27	**Yes**	4	5	1	1	2	**Yes**
0211	290	279	PATB	Yes	4.27	**Yes**	4	4	1	1	2	**Yes**
0212	269		PATB	Yes	3.66	**Yes**	4	4	1	1	2	**Yes**
Overall - Excellent.							**Overall - Excellent.**					
Supplemental Sections - 7 PCC sections.												
0259	279 mm JPC (3.8 MPa MR) on 152 mm DGAB						3	4	1	0	2	**Yes**
0260	279 mm JPC (3.8 MPa MR) on 102 mm PATB on 102 mm DGAB						4	4	1	1	2	**Yes**
0261	279 mm JPC (3.8 MPa MR) on 102 mm CTPB on 102 mm DGAB						4	3	1	1	2	**Yes**
0262	279 mm JPC on 102 mm CTPB on 102 mm DGAB						3	3	1	1	2	**Yes**
0263	279 mm JPC on 152 mm DGAB						3	3	1	0	2	**Yes**
0264	279 mm JPC on 152 mm DGAB						3	2	1	0	2	**Yes**
0265	279 mm JPC (3.8 MPa MR) on 102 mm PATB on 102 mm DGAB						4	4	1	1	2	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- The LTPP SPS Project Deviation Report indicates that some of the DGAB cracked during compaction (sections 390259 and 390204). Contaminated PATB was removed and replaced due to an oil spill in section 390260.
- Monolithic construction of base layers would have ensured that a layer of uniform thickness and material quality would have been constructed transversely across the typical pavement section. Monolithic construction would also have created the highest support conditions at the pavement edge, which is often the most critical stress area (edge stresses and positive curling stresses) for a doweled JPCP. Only the CTPB width can be considered monolithic.
- Individual pavement layer thicknesses are often in excess of LTPP tolerances. Variability of a single layer depth occurs both within an individual test section and from section to section for those test sections that have common layer depth requirements.
- 390259 and 390204—Some surface aggregate cracked due to compaction.
- 390260—Oil spilled on PATB. Contaminated sections removed and replaced.

Project Status Summary

Overall, this project site is in very good shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
  - Mean 14-day flexural strength values deviated by more than 10 percent from the design values.
- Construction difficulties and deviations—Relatively minor.
- Data availability—Excellent overall.
  - Site condition data—Good, traffic data not available.
  - Key PCC materials testing data availability for core sections—Excellent, with 96 percent completed.
  - Monitoring data availability—Excellent.

The Ohio SPS-2 site does not appear to exhibit significant problems that will cause difficulty in performance analysis.

WASHINGTON SPS-2

The Washington SPS-2 project site is located in the northbound lanes of S.R. 395 in eastern Washington, 4.8 km south of Ritzville. S.R. 395 is an urban principal arterial; in 1993 the AADT was 18,000. The initial annual ESALs are estimated at 461,759. The SPS-2 project includes construction of two new northbound lanes and the upgrade of S.R. 395 to a four-lane divided highway. The new lanes were constructed uphill from the existing lanes. Two sections were located in a cut (section 530203 and 530259), while all other sections were located on fills. The roadway design for this project consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m.

The initial SPS-2 sections were constructed on a horizontal curve to the left from the beginning of the SPS-2 project to Station 2050+00. Section 530201 is partially located within a horizontal curve and partially located within a superelevation runout area. Sections 530205, 530206, 530207, and 530208 are on tangent, while the remaining sections were constructed on a curve to the left. The maximum superelevation rate for this curve is 3 percent. Vertical grades range from 0.14 percent to 3 percent.

Construction of the SPS-2 site started in June 1993 with the removal of the existing pavement. Construction of individual test sections was completed by November 1, 1995.

All required core sections were constructed, as was one supplemental State test section. Table 50 summarizes key project information and data available for all the sections.

Table 50. Washington SPS-2 project summary.

Project level information and data availability **Construction date:** **11/01/1995**

		Data Availability	*Average values*	*As planned?*
Climate - DF	***CLM:***	17 years	FI: 265 °C days, Precip. 308 mm	**Yes**
	AWS:	5 years	FI: 138 °C days, Precip. 355 mm	
Traffic	***WIM:***	2 years	461,759 ESALs/year (>200,000)	**Yes**
Subgrade type	Coarse-grained soil for all.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	3.34	**No**
14-day MPa		6.2	5.73	**Yes**
PCC tests available	**On average 100% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							**Data availability, No. of tests**					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0201	***221***	203	AGG	No	3.66	**No**	4	5	3	0	3	**Yes**
0202	211		AGG	No	4.27	**Yes**	4	5	3	0	3	**Yes**
0203	282	279	AGG	No	4.27	**Yes**	4	5	3	0	3	**Yes**
0204	284		AGG	No	3.66	**Yes**	4	5	3	0	3	**Yes**
0205	216	203	LCB	No	3.66	**Yes**	4	4	3	0	3	**Yes**
0206	***218***		LCB	No	4.27	**No**	4	4	3	0	3	**Yes**
0207	282	279	LCB	No	4.27	**Yes**	4	4	3	0	3	**Yes**
0208	284		LCB	No	3.66	**Yes**	4	4	3	0	3	**Yes**
0209	***229***	203	PATB	Yes	3.66	**No**	4	5	3	0	3	**Yes**
0210	211		PATB	Yes	4.27	**Yes**	4	5	3	0	3	**Yes**
0211	***300***	279	PATB	Yes	4.27	**No**	4	5	3	0	3	**Yes**
0212	287		PATB	Yes	3.66	**Yes**	4	5	3	0	3	**Yes**
Overall - Fair, four sections outside design range.							**Overall - Excellent.**					
Supplemental Sections - 1 PCC section.												
0259	Undoweled 254 mm JPC (4.5 MPa MR) on 76 mm ATB on 51 mm crushed surfacing base course; 4.27 m lane.						4	4	3	0	3	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)
Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- Construction traffic helped to further consolidate the DGAB as evidenced by an average density of 2,106 kg/m^3 for those DGAB sections receiving construction traffic and an average density of 1,867 kg/m^3 for the control section (section 530259) that did not receive construction traffic.
- Six of the eight test sections constructed with DGAB had average thicknesses between 10 and 23 mm greater than SPS-2 specifications.

- The average PATB thickness was 66 mm with 10 mm standard deviation. The SPS-2 specified thickness was 76 mm ±6.4 mm.
- The average LCB thickness was either 155 or 157 mm for each test section paved with LCB. The SPS-2 specified thickness was 152 mm ±6.4 mm.
- The 203-mm PCC test sections had average thicknesses ranging from 211 to 206 mm. Sections 530201, 530206, and 530209 had thicknesses of 221, 218, and 216 mm, respectively.
- All 279-mm PCC test sections had PCC thicknesses within 7.6 mm of the specified depth.
- The 14-day core compressive strengths for three of the four LCB test sections were within SHRP tolerances of 3.4 to 5.2 MPa. Section 530207 had compressive strengths up to 2.5 times as high as other LCB test sections. This was attributed to a water-cement ratio lower than the mix design.
- All but one PATB section had an average thickness of either 97 or 99 mm. Section 530212 had an average PATB thickness of 89 mm.
- The 3.8 MPa mix had hairline cracks below the sawn transverse joint and 6.4 mm joint widths several days after paving for the DGAB and LCB sections. The 203-mm PCC on LCB (section 530205) had not cracked at the transverse joints by October 2, 1995.
- The 6.2 MPa mix had larger cracked joint widths than the 3.8 MPa mix for corresponding sections.
- The 3.8 MPa mix had cracked joints up to 7.9 mm in the PATB sections, while the 6.2 MPa mix had transverse and average joint crack width of 13 mm on PATB sections.
- Section 530206 developed shrinkage cracks from 1.6 to 3.2 mm in width. All but 1 slab was cracked, and 19 of the 32 slabs had more than 5 cracks per slab.
- Transverse and longitudinal joints were sealed with a hot pour material.
- FWD testing revealed that those sections constructed in cut areas had the most variability in support (0.4 to 1.4 mm), while those test sections constructed on embankments had more uniform support.

Project Status Summary

Overall, this project site is in good shape. The appendices to this report contain a significant amount of monitoring data. The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
    - Mean 14-day lower flexural strength values deviated by more than 10 percent from the design values.
    - Mean slab thickness value for the 279 mm cell deviated by more than 12.7 mm from the design value.
- Construction difficulties and deviations—Relatively minor.
- Data availability—Excellent overall.
    - Site condition data—Good; need more traffic data.

- Key PCC materials testing data availability for core sections—Excellent, with 100 percent completed.
- Monitoring data availability—Excellent.

The Washington SPS-2 site does not appear to exhibit significant problems that will cause difficulty in performance analysis.

WISCONSIN SPS-2

The Wisconsin SPS-2 project site is located on the westbound and eastbound lanes of Wisconsin State Highway 29 (STH-29), a rural arterial road, in Marathon County, WI. This site is roughly 5.6 km east of Hatley, WI. In 1995, the ADT was 6,650 vehicles with a truck distribution of 29.5 percent. The initial annual ESALs were estimated at 500,000. The site is located on a 0.3 percent downgrade with four curves in between. The maximum curve does not exceed 2 degrees with a superelevation equal to 0.055 l/l. The lanes are 3.66 m and 4.27 m wide, with an outside shoulder of 3.05 m and an inside shoulder of 1.83 m.

A WIM system was installed on August 29, 1997. The WIM equipment used was a DAW-1000 bending plate unit manufactured by PAT Equipment.

The subgrade preparation for this project began in early June 1997, and paving operations were completed by mid-October 1997.

All required core sections were constructed. Eight supplemental State test sections were also constructed. Table 51 summarizes key project information and data available for all the sections.

Table 51. Wisconsin SPS-2 project summary.

Project level information and data availability **Construction date:** **10/01/1997**

		Data Availability	*Average values*	*As planned?*
Climate - WF	***CLM:***	0 years	N/A	**N/A**
	AWS:	0 years	N/A	
Traffic	***WIM:***	0 years	N/A	**N/A**
Subgrade type	Designed as coarse-grained soil.		**As designed?**	**Varies**
		Design value	*Actual Averages*	*Within 10%?*
Flexural strength		3.8	4.37	**No**
14-day MPa		6.2	6.09	**Yes**
PCC tests available	**On average 71% completed for core sections.**			

Section level key design factors and monitoring data availability

Key pavement design factors							Data availability, No. of tests					
ID	**Slab Thick. mm**		**Base type**	**With Drain**	**Lane Width m**	**As Design ?**	**IRI**	**FWD**	**Distress**			**Meet Min. Req'd?**
	Actual	**Design**							**Manual**	**Photo.**	**Faulting**	
0213	**NA**	203	AGG	No	4.27	**NA**	3	1	1	0	1	**Yes**
0214			AGG	No	3.66		3	1	1	0	2	**Yes**
0215		279	AGG	No	3.66		3	1	1	0	1	**Yes**
0216			AGG	No	4.27		3	1	1	0	1	**Yes**
0217		203	LCB	No	4.27		3	1	1	0	1	**Yes**
0218			LCB	No	3.66		3	1	1	0	1	**Yes**
0219		279	LCB	No	3.66		3	1	1	0	1	**Yes**
0220			LCB	No	4.27		3	1	1	0	1	**Yes**
0221		203	PATB	Yes	4.27		3	1	1	0	1	**Yes**
0222			PATB	Yes	3.66		3	1	1	0	1	**Yes**
0223		279	PATB	Yes	3.66		3	1	1	0	1	**Yes**
0224			PATB	Yes	4.27		3	1	1	0	1	**Yes**
Overall - Not enough data.							**Overall - Excellent.**					
Supplemental Sections - 8 PCC sections.												
0259	279 mm JPC (3.8 MPa MR) on 152 mm DGAB						3	1	1	0	1	**Yes**
0260	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with alternate dowel bar placement						3	1	1	0	1	**Yes**
0261	203 mm JPC (3.8 MPa MR) on 102 mm OGDB on 102 mm DGAB						3	1	1	0	1	**Yes**
0262	203 mm JPC (6.3 MPa MR) on 152 mm DGAB, with tied concrete shoulder						3	1	1	0	1	**Yes**
0263	203-279 mm JPC (3.8 MPa MR) on 152 mm DGAB, variable pavement thickness						3	1	1	0	1	**Yes**
0264	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with composite dowels						3	1	1	0	1	**Yes**
0265	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with stainless steel dowels.						3	1	1	0	1	**Yes**
0266							3	0	0	0	0	**Yes**
							Overall - Excellent.					

Note: * Indicates seasonal monitoring section(s)

Bolded and italic letters represent constructed values that are either not as designed or outside the design range. For slab thickness, the design range is set at (design value +/- 12.7 mm)

Key Observations and Deviations

The following key observations were noted in the project construction report:

- During the splitspoon testing, a number of areas had existing concrete slabs located beneath the old pavement structure. These areas of concrete were removed and fill was placed in these areas.
- Because of the process used to remove the existing pavement, it was not possible to obtain undisturbed samples of the existing base or subbase material.
- Soil boring records were provided that made it unnecessary to perform shoulder probes. The depth to rigid layer exceeded 6.1 m.

Project Status Summary

Overall, this project site is in good shape, given the available data. The appendices to this report contain a significant amount of monitoring data as well as significant materials testing data. However, site condition data, pavement structure data, and other key pavement design feature data are deficient at this time. Not enough data exist in the database to assess the designed versus constructed status of the project.
The following summarizes the status of this project:

- Designed versus constructed—Good. The key deviations are listed below:
    - Mean 14-day lower flexural strength values deviated by more than 10 percent from the design values.
- Construction difficulties and deviations—Relatively minor.
    - Data availability—Some missing data, probably in the pipeline.
    - Site condition data not available at the time of analysis.
    - Key PCC materials testing data availability for core sections—Fair, with 71 percent completed.
    - Monitoring data availability—Excellent.

6. INITIAL EVALUATION OF KEY PERFORMANCE TRENDS

This chapter provides an initial review and evaluation of the key performance trends for the SPS-2 project. Note that this initial evaluation is cursory in nature, because it is not within the scope of this study to conduct a comprehensive evaluation. Furthermore, the long-term performance may be different from short-term performance. The following key performance data are reviewed:

- Edge joint faulting.
- Transverse cracking.
- Longitudinal cracking.
- Pavement smoothness.

The SPS-2 project sites are relatively young pavements, ranging from 2 years old in Wisconsin to 7.5 years old in Kansas. Therefore, as expected, most SPS-2 sections are showing good performance and low distress levels. Table 52 summarizes the SPS-2 sections showing noticeable distress, along with key pavement design factors. (Noticeable distress is defined as a section that has a mean edge faulting greater than 1.0 mm or that exhibits longitudinal or transverse cracking.) As of January 2000, only 43 out of 155 sections (28 percent) showed any noticeable distress.

Because of the very low distress level of most sections, distress prediction models were not developed at this time. However, an initial evaluation of what affects the distressed sections was conducted. The results of the distress and profile data review and evaluation are presented in the following sections.

Note that these are early performance trends; the long-term performance may be different.

JOINT FAULTING REVIEW AND EVALUATION

The distribution of the mean joint faulting values (the lane at edge) in SPS-2 sections, recorded as of January 2000, is illustrated in figure 13. Note that these values are the maximum mean faulting values recorded over the life of the section to date.

Ninety-five percent of the SPS-2 sections (148 sections) currently have a mean edge faulting less than 1 mm. Of the seven sections having greater than 1-mm faulting, six were constructed with an aggregate base and one was constructed with a lean concrete base. Additionally, three sections are from the heavily trafficked Michigan SPS-2 project site, and two sections are from the Nevada SPS-2 project site.

Table 52. SPS-2 sections with noticeable distress.

Section ID		Age as of Jan. 2000	Climate Zone	Subgrade	Base / Drain	Lane Width, m	Slab Thick mm	14-day F.S. MPa	Maximum Distress Recorded		
State	SHRP								Fault mm	Trans. Cracks, % slab cracked	Long. Cracks, length, m
4	0213	6.3	DNF	Coarse	AGG	4.27	201	3.9	0.4	0	**5**
4	0215	6.3	DNF	Coarse	AGG	3.66	287	3.9	**1.1**	0	0
4	0217	6.3	DNF	Coarse	LCB	4.27	206	3.9	-0.1	**15**	**11**
4	0218	6.3	DNF	Coarse	LCB	3.66	211	5.8	0	**24**	**11**
4	0219	6.3	DNF	Coarse	LCB	3.66	274	3.9	-0.1	0	**2**
4	0220	6.3	DNF	Coarse	LCB	4.27	287	5.8	0.8	**6**	**1**
4	0221	6.3	DNF	Coarse	PATB/ Drain	4.27	208	3.9	0.8	0	**8**
4	0222	6.3	DNF	Coarse	PATB/ Drain	3.66	218	5.8	0.5	0	**2**
8	0217	6.3	DF	Coarse	LCB	4.27	218	3.6	0.4	**30**	**12**
8	0218	6.3	DF	Coarse	LCB	3.66	196	6.2	1.0	**3**	0
8	0221	6.3	DF	Coarse	PATB/ Drain	4.27	211	3.6	0.1	0	**1**
8	0222	6.3	DF	Coarse	PATB/ Drain	3.66	221	6.2	0.1	0	**1**
10	0205	3.7	WF	Coarse	LCB	3.66	234	4.5	**1.3**	**30**	**15**
10	0207	3.7	WF	Coarse	LCB	4.27	287	4.5	0	0	**46**
19	0213	5.4	WF	Fine	AGG	4.27	216	3.2	0.4	0	**3**
19	0217	5.4	WF	Fine	LCB	4.27	196	3.2	0.4	**6**	**0**
19	0218	5.4	WF	Fine	LCB	3.66	208	5.2	0.3	**3**	0
20	0201	7.5	WF	Fine	AGG	3.66	196	4.2	0	**21**	**8**
20	0202	7.5	WF	Fine	AGG	4.27	188	5.8	0	**12**	**11**
20	0206	7.5	WF	Fine	LCB	4.27	201	5.8	0	0	**2**
26	0213	6.2	WF	Fine	AGG	4.27	218	4.3	**1.2**	**9**	0
26	0214	6.2	WF	Fine	AGG	3.66	226	6.7	**1.4**	0	0
26	0215	6.2	WF	Fine	AGG	3.66	284	4.3	**2.6**	**6**	0
26	0217	6.2	WF	Fine	LCB	4.27	216	4.3	0.3	0	**6**
26	0218	6.2	WF	Fine	LCB	3.66	180	6.7	0.9	**36**	**20**
32	0201	4.4	DF	Coarse	AGG	3.66	234	3.6	1.0	**100**	**119**
32	0202	4.4	DF	Coarse	AGG	4.27	208	5.4	**1.1**	**100**	**170**
32	0203	4.4	DF	Coarse	AGG	4.27	302	3.6	0.8	**100**	**107**
32	0204	4.4	DF	Coarse	AGG	3.66	300	5.4	**1.4**	**70**	**11**
32	0205	4.4	DF	Coarse	LCB	3.66	216	3.6	0.8	**100**	**216**
32	0206	4.4	DF	Coarse	LCB	4.27	198	5.4	0.3	**100**	**273**
32	0207	4.4	DF	Coarse	LCB	4.27	277	3.6	0.8	**6**	**18**
32	0208	4.4	DF	Coarse	LCB	3.66	279	5.4	0.8	**85**	**18**
32	0210	4.4	DF	Coarse	PATB/ Drain	4.27	257	5.4	0.8	**94**	**12**
32	0211	4.4	DF	Coarse	PATB/ Drain	4.27	287	3.6	0.8	**12**	**2**
37	0201	5.5	WNF	Fine	AGG	3.66	229		0.9	0	**4**
37	0210	5.5	WNF	Fine	PATB/ Drain	4.27	213		0.1	**6**	**0.2**
38	0217	5.3	WF	Fine	LCB	4.27	201		0.1	**12**	**26**
38	0224	5.3	WF	Fine	PATB/ Drain	4.27	274		0.5	0	**9**
39	0205	3.3	WF	Fine	LCB	3.66	203	4.7	0.5	**9**	0
39	0206	3.3	WF	Fine	LCB	4.27	201	4.2	0	0	**21**
53	0205	4.2	DF	Coarse	LCB	3.66	216	3.3	0.5	**3**	0
53	0206	4.2	DF	Coarse	LCB	4.27	218	5.7	0.6	**6**	**53**

Note: Significant distress level is defined as: (1) Faulting > 1.0 mm, or (2) with at least one transverse or longitudinal crack.

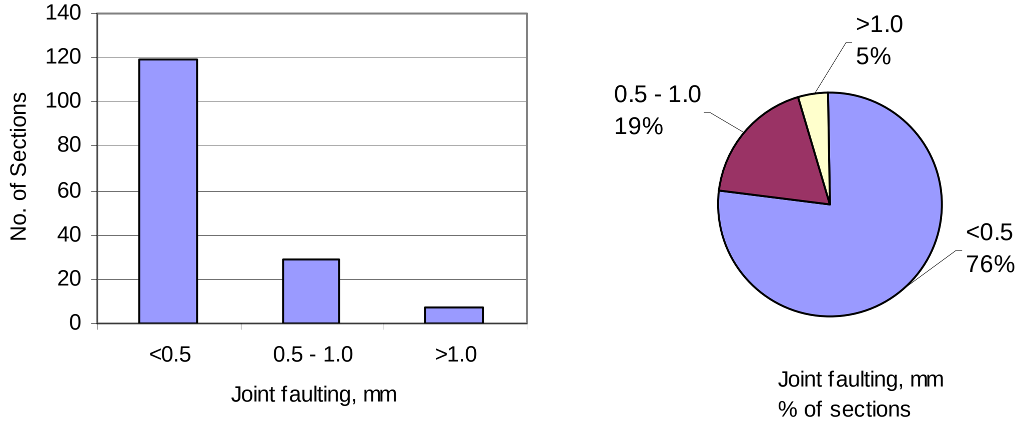


Figure 13. Distribution of the mean joint faulting values for SPS-2 sections (total 155 sections).

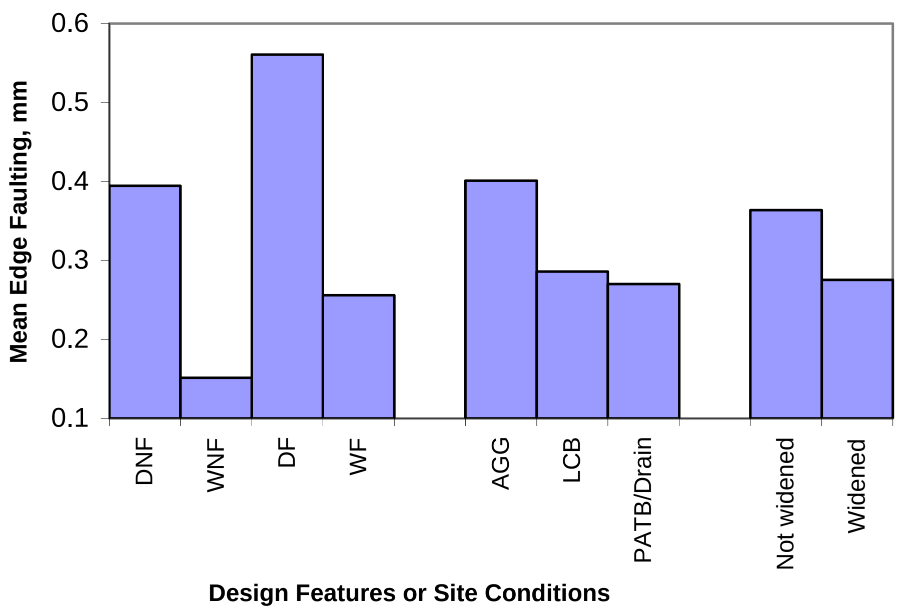


Figure 14. Mean edge joint faulting for different categories.

The mean edge faulting for different groups of design features or site conditions is depicted in figure 14. The following conclusions can be drawn from this initial evaluation:

- Climatic zone—Joint faulting is the most prevalent in the dry freeze zone, followed by the dry no-freeze zone and the wet freeze zone. Sections in the wet no-freeze climate have the least faulting so far.
- Base type—Sections with an aggregate base have the highest joint faulting level. Sections with an LCB and PATB have the lowest joint faulting.
- Widened slab—Widened slab sections are showing less faulting than conventional width slabs.

An example of a faulting time history trend using data from the Michigan SPS-2 project site is provided in figure 15. Faulting measurements from different sections were averaged by base types. The aggregate base type shows the highest faulting trend over time.

TRANSVERSE CRACKING REVIEW AND EVALUATION

The distribution of the transverse cracking is depicted in figure 16. As of January 2000, 82 percent of the sections had zero transverse cracks. However, 5 percent (eight sections) of the SPS-2 sections showed more than 50 percent slabs cracked. As shown in table 52, all eight sections are from at the Nevada SPS-2 project site, which was less than 4.5 years old. This excessive early cracking appears to indicate serious construction problems at the site. A review of the field distress maps will be very helpful in explaining these cracking observations.

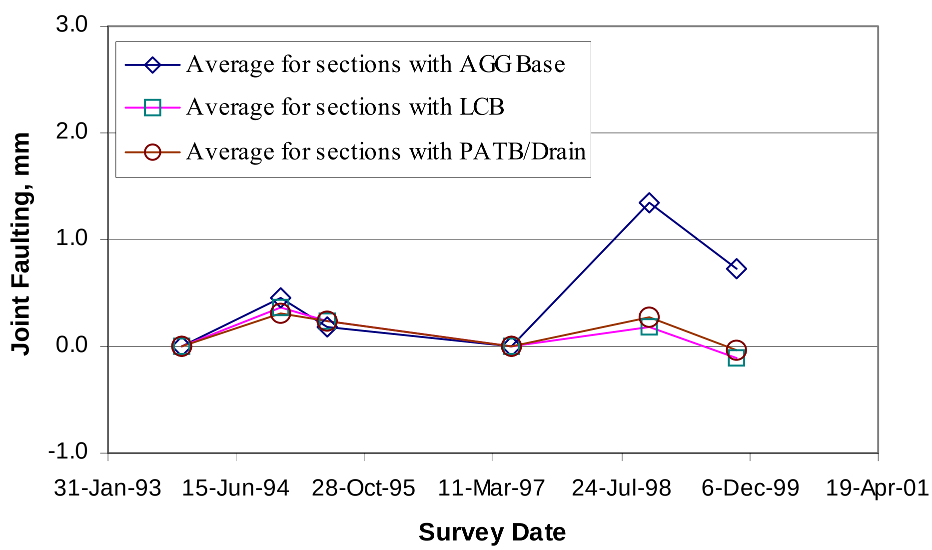


Figure 15. Sample faulting time history plot—heavily trafficked Michigan SPS-2 sections by base types.

Due to the excessive cracking at the Nevada site, the cracking data from that site were not included in the following evaluation. The mean percentage of slabs cracked transversely, grouped by design features or site conditions, is given in figure 17. The following preliminary conclusions can be drawn from these mean value comparisons:

- Climatic zone—The percentage of slabs cracked transversely is highest in the dry no-freeze zone, followed by the wet freeze and dry freeze zones. The smallest percentage is in sections in the wet no-freeze zone.
- Base type—Sections with PATB show the lowest percentage of slabs cracked transversely, while the sections with an LCB show the highest transverse cracking.
- Slab thickness—Thinner (203-mm) slabs show more transverse cracks than thick slabs.

A sample time history for transverse cracking, using data from the Michigan SPS-2 project site, is shown in figure 18. Section 260218, which has a very thin slab and LCB, shows the highest level of cracking. Nine of 12 sections (75 percent) show no transverse cracking.

LONGITUDINAL CRACKING REVIEW AND EVALUATION

The distribution of longitudinal cracking is shown in figure 19. As of January 2000, 78 percent of the sections have no longitudinal cracks, whereas 4 percent (6 sections) have more than 50 m longitudinal cracking. Five of these six sections are at the Nevada SPS-2 project site, which was less than 4.5 years old at that time. Again, this excessive early cracking indicates serious construction problems at the site, and a review of the field distress maps will be very helpful in explaining these cracking observations. The Nevada site data were not included in the following plots and analyses.

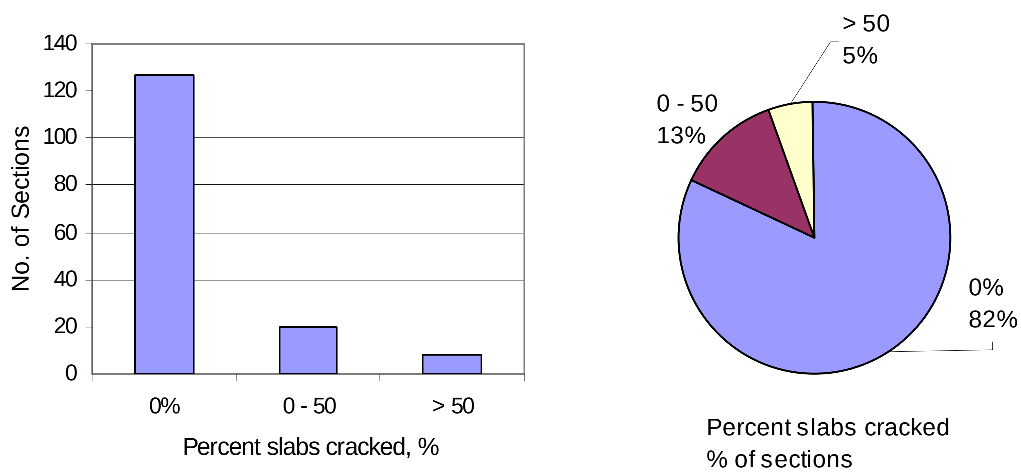


Figure 16. Distribution of the transverse cracking for SPS-2 sections (total 155 sections).

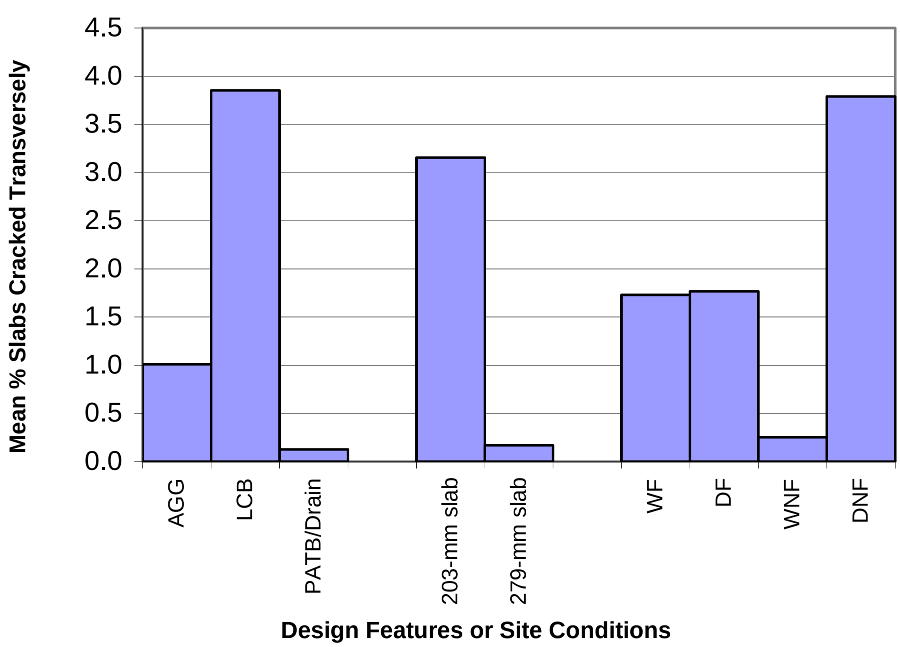


Figure 17. SPS-2 mean percentage of slabs cracked transversely for different categories.

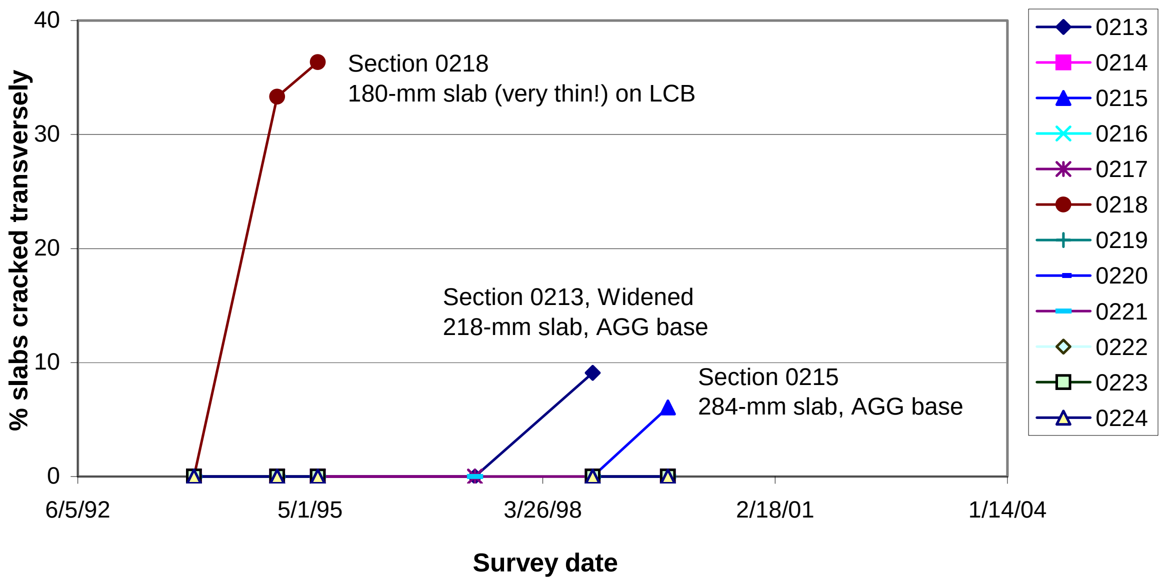


Figure 18. Sample time history plot for transverse cracking, Michigan SPS-2 project site.

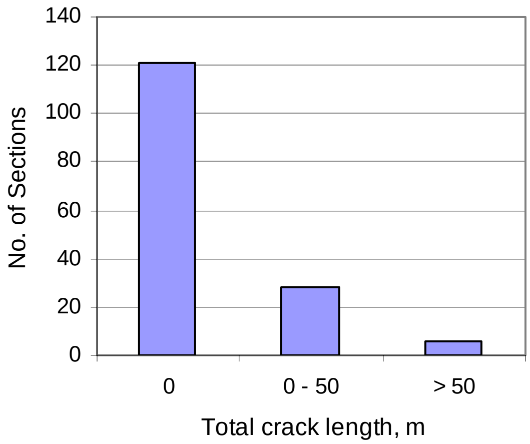
The average total longitudinal crack length per section for different groups of design features or site conditions is given in figure 20. The conclusions drawn from these comparisons are very similar to those for transverse cracking.

- Climatic zone—The total longitudinal crack length is largest in the dry no-freeze zone, followed by the dry freeze and wet freeze zones. The smallest crack length is found in sections in the wet no-freeze zone.
- Base type—Sections with PATB show the lowest total longitudinal cracking levels, while the sections with an LCB show the highest longitudinal cracking.
- Slab thickness and widened slab—Thinner (203-mm) slabs show more longitudinal cracks. Sections with a thinner slab and widened slab show by far the highest level of longitudinal cracking by far.

A sample time history for the longitudinal cracking, using data from the Michigan SPS-2 project site, is provided in figure 21. Again, section 260218 shows the highest level of cracking. Ten out of 12 sections (83 percent) show no longitudinal cracking.

PAVEMENT SMOOTHNESS REVIEW AND EVALUATION

Pavement smoothness affects ride quality, and therefore is a very important performance indicator. In this study, the initial IRI and the IRI over time were both evaluated.



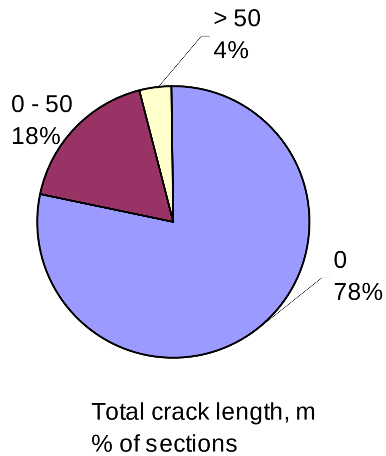


Figure 19. Distribution of the longitudinal cracking for SPS-2 sections (total 155 sections).

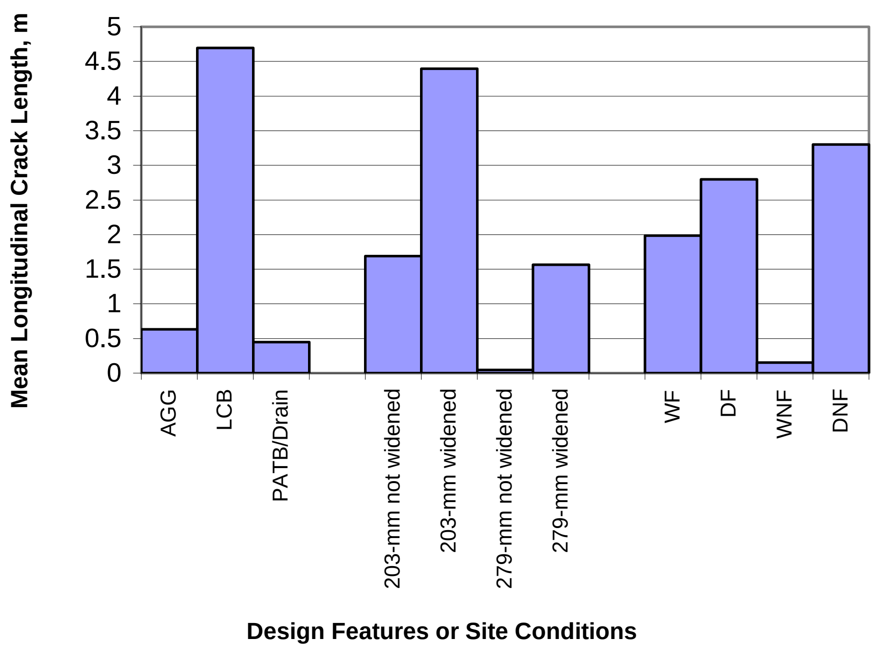


Figure 20. SPS-2 mean total longitudinal cracking for different categories.

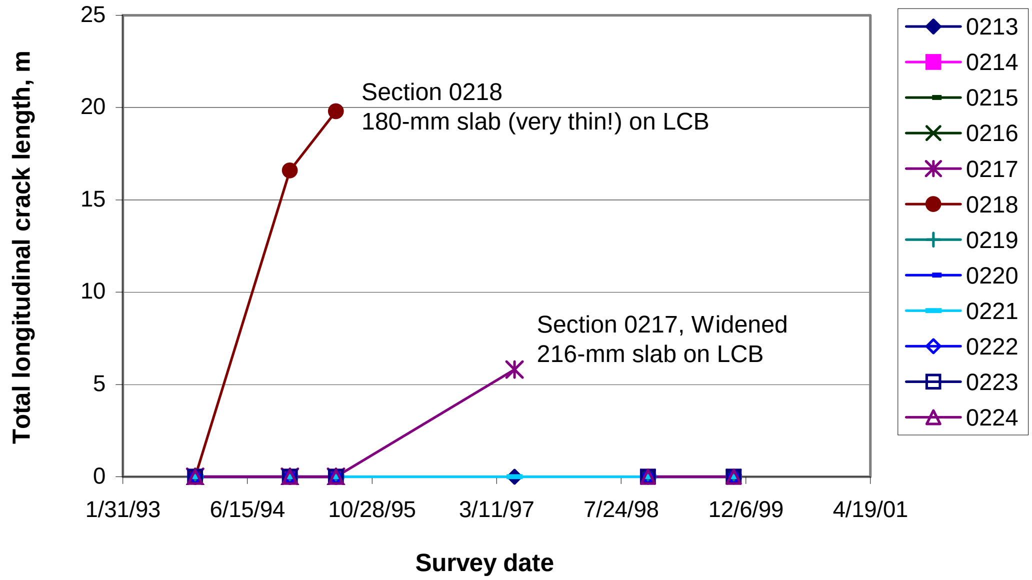


Figure 21. Sample time history plot for longitudinal cracking—heavily trafficked Michigan SPS-2 project.

Initial IRI Measurements

The initial IRI measurements represent the smoothness of the pavement soon after construction. Previous studies showed that initial smoothness significantly affects future smoothness of the pavements. For SPS-2 sections, the distribution of the initial IRI is shown in figure 22. The mean initial IRI was 1.30 m/km, and ranged from 0.76 to 2.19 m/km.

The effects of the key design features on initial IRI were analyzed statistically. An analysis of variance (ANOVA) was conducted that showed the following factors as significant:

- JPCP constructed on coarse-grained subgrades were smoother than those constructed on fine-grained subgrade soil. This could be due to a stiffer foundation upon which to build the pavement.
- Permeable base with edge drain or aggregate base (versus lean concrete base) was found to be smoother. It has commonly been thought that it is more difficult to build a smooth pavement on a permeable base than on a treated base, but these results show this to not be the case for SPS-2 sites.
- Widened slab sections were smoother than conventional slab sections.
- Thinner slabs and lower 14-day strength concrete slabs were constructed smoother than thicker and higher 14-day strength slabs.

The mean initial IRI values for different design features and site conditions conditions are shown in figure 23.

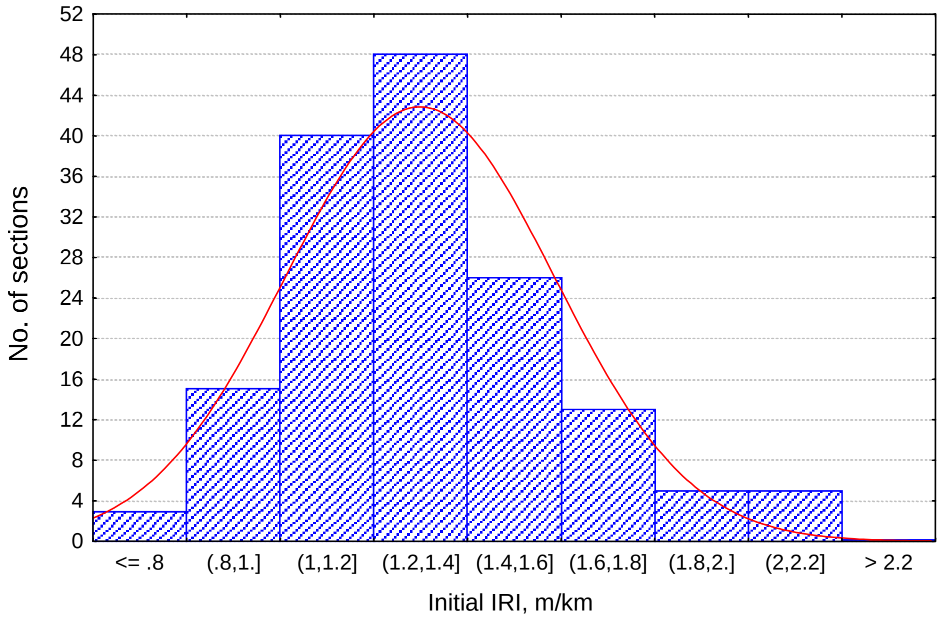


Figure 22. Distribution of the initial IRI for SPS-2 sections
(total 155 sections, mean = 1.30 m/km).

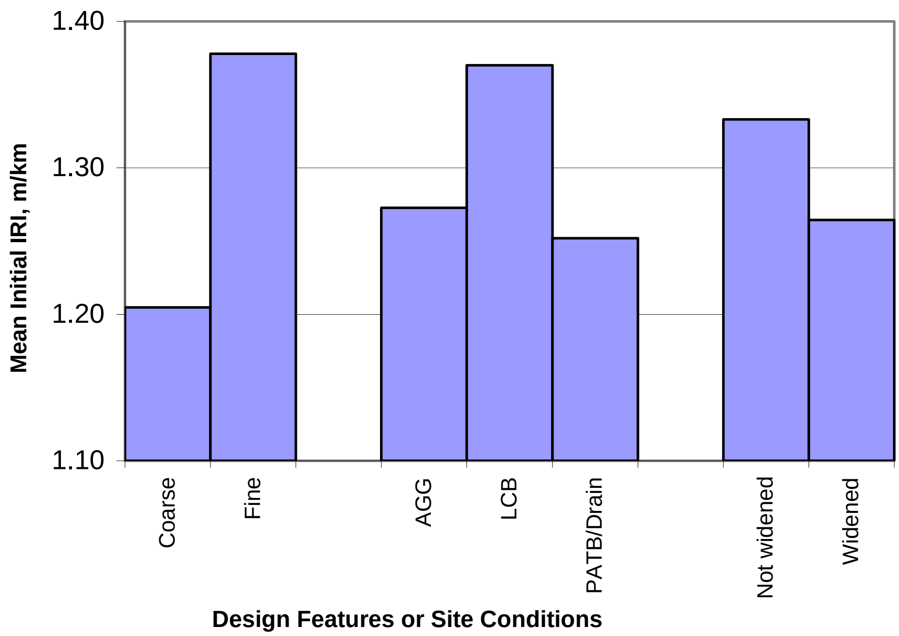


Figure 23. SPS-2 mean initial IRI for different site conditions and design features.

IRI Evaluation

The IRI of each SPS-2 section was measured over time. The maximum value over time was determined and analyzed (typically, this occurred at the latest survey date). Distribution of the maximum IRI of all SPS-2 sections as of January 2000 is shown in figure 24. A majority of the SPS-2 sections (66 percent) are still very smooth, with an IRI less than 1.5 m/km. However, three sections are very rough, with an IRI greater than 2.5 m/km. These three sections are all in Michigan (sections 260214, 260217, and 260218).

It is a common belief that smoothness or IRI over time is a function of the initial IRI, cumulative traffic, and surface distresses. A multiple regression analysis was conducted on the maximum IRI, and the following regression model was developed:

$$\begin{aligned} IRI = {} & 0.08777 + 0.993 * Init\_IRI + 0.1630\ Fault \\ & + 0.006045 * KESAL * Survey\_Age \end{aligned} \quad (1)$$

$R^2 = 72\%$
SEE = 0.192
N = 56

Where:

IRI	=	IRI value at SPS-2 sections over time, m/km.
Init_IRI	=	Initial IRI measurements, m/km.
KESAL	=	Average annual KESAL (1,000 equivalent single axle loads).
Survey_Age	=	Age when IRI was measured, year.
Fault	=	Mean joint faulting, mm.

This model shows that the initial IRI, joint faulting, and total KESALs (Age * annual KESALs) affect future IRI values. Transverse cracking and longitudinal cracking did not show significant effects on future IRI. This may be due to the low severity levels of these cracks at SPS-2 sections.

SUMMARY

The SPS-2 sections are relatively young, and a large majority show little distress. As of January 2000, only 43 of 155 sections (28 percent) are showing any noticeable distresses. Ninety-five percent of the SPS-2 sections have less than 1 mm of edge joint faulting. Eighty-seven percent of the SPS-2 sections show zero transverse cracking, and 78 percent of the sections have zero longitudinal cracking.

Based on the preliminary statistical analyses and comparisons, the following preliminary and early performance trends are observed (note that long-term performance may be different from short-term performance):

- The initial IRI of SPS-2 sections ranged from 0.76 to 2.19 m/km with a mean of 1.30 m/km. JPCP constructed on coarse-grained soil were smoother (lower IRI) than those constructed on fine-grained soils. JPCP constructed on PATB were smoother than those constructed on other base types.

- The IRI over time (up to 7.5 years) depended heavily on the initial IRI, the traffic loadings, and the extent of joint faulting.
- Sections with PATB show the lowest total longitudinal cracking levels, while the sections with LCB show the highest longitudinal cracking.
- Thinner (203 mm) slabs show more longitudinal cracks. Sections with a thinner slab and widened slab show the highest level of longitudinal cracking.
- Sections with PATB are showing the lowest percentage of slabs cracked transversely, while the sections with LCB show the highest transverse cracking.
- Thinner (203 mm) slabs show more transverse cracks than thicker slabs. Sections with a thinner slab and a widened slab show the highest level of transverse cracking.
- Sections with aggregate base show the highest joint faulting level. Sections with an LCB and PATB have the lowest joint faulting.
- Widened slab sections show less faulting than conventional width slabs.
- The Nevada SPS-2 site sections showed excessive cracking after only 4 years, most of which occurred during construction. These sections will need special care in analysis of the data.

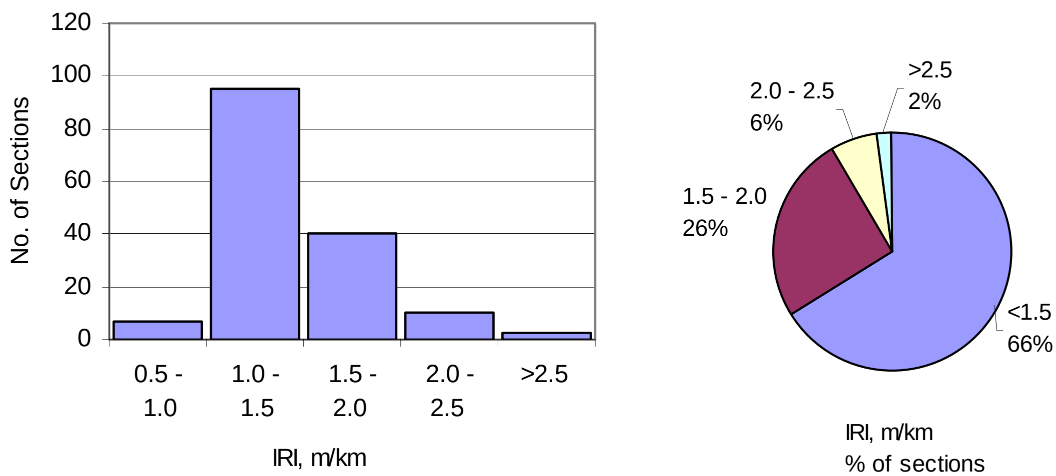


Figure 24. Distribution of the IRI for SPS-2 sections (January 2000) (total 155 sections).

7. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The SPS-2 project, *Strategic Study of Structural Factors for Rigid Pavement of Jointed Plain Concrete Pavements*, is one of the key experiments in the LTPP program. The main objective is to determine the relative influence and long-term effectiveness of the JPC pavement strategic factors that affect its performance. There are some concerns about the ability of the SPS-2 experiment to meet those expectations, given that several SPS-2 sites were not constructed. In addition, at the SPS-2 sites that were constructed, some construction deviations and data collection deficiencies exist.

This study provides the first comprehensive review and evaluation of the SPS-2 experiment. Issues of experimental design, construction quality, data availability and completeness, and early performance trends are addressed in this report. The key findings are summarized in this chapter. This chapter also presents the research team's recommendations on improving the SPS-2 experiment and its data availability, expectations of the SPS-2 experiment, and future data collection and analysis topics.

SPS-2 EXPERIMENTAL SITE STATUS

As of January 2000, 13 SPS-2 sites have been constructed throughout the United States. An additional site in California has been nominated and is currently under construction. The full factorial of the original experimental design is not completely satisfied by the constructed sites, leaving a portion (31 percent) of the desired inference space (subgrade and climate) with no experimental sites.

These 14 sites fill in 11 of the 16 SPS-2 experiment factorial cells and provide good coverage of major areas of the United States. Five sites (31 percent) are missing from the original experiment design, as listed below:

- Two sites in a wet no-freeze climate (southeast United States) with a coarse-grained subgrade. Data from each site will fill half of the design factorial.
- One site in a dry freeze climate (northwest United States) with a fine-grained subgrade. Data from each site will fill half of the design factorial.
- Two sites in a dry no-freeze climate (southwest United States), one on a fine-grained subgrade and the other on a coarse-grained subgrade. Data from each site will fill half of the design factorial.

Five additional SPS-2 sites (60 sections total) are needed to complete the design factorial. While it is impossible to determine the exact effects at this time, the lack of data from these missing sites will limit the results obtainable from the SPS-2 experiment, as summarized below:

- There will be no performance data and, thus, no performance findings from the missing sites.

- The missing site corresponding to the site in Arizona (dry/no-freeze and coarse subgrade) will make it impossible to determine (without confounding factors) the main effects and interactions for the dry/no-freeze climate.
- The missing section at the Nevada site will reduce the findings for that site and corresponding cell (although the Washington site appears to be a replicate).
- Adequate SPS-2 sites exist in wet-freeze climates, making a full inference space of performance data available. All main effects and interactions in this climate should be ascertainable.
- The number of SPS-2 sites in no-freeze climates, both wet and dry areas, is deficient. There will be difficulties in determining the main effects and interactions in these climates.

Some of these deficiencies can be overcome through use of mechanistic data analysis. However, there is no mechanistic analysis method that considers all factors involved, and the missing data cells will always limit in the verification and calibration of any performance models.

In addition to the 12 core sections at each site required by the SPS-2 experiment, SHAs have built a total of 40 supplemental sections with SPS-2 projects. The main value of the supplemental sections will be as a direct comparison to the core sections. For example, one supplemental section in Washington did not have dowels, and this will provide a direct comparison to a similar design with dowel bars. Various other comparisons are possible, including skewed joints, base types, subdrainage, slab thickness, asphalt concrete pavement, jointed reinforced concrete, special dowels, and variable slab thickness. These sections are valuable to the States, and efforts should be made to ensure that their construction and monitoring data are collected and stored.

DATA AVAILABILITY AND COMPLETENESS

The SPS-2 project is an extremely valuable source of performance data for modern concrete pavements. The performance of these sections will be of great interest to all highway agencies building concrete pavements, and of course to the concrete industry as well. This report will be the major reference document for all future SPS-2 research studies, and will be the major reference document for the 14 SHAs that have constructed SPS-2 project sites.

Data elements that were considered to be essential to the SPS-2 experiment analyses have been assessed. The data availability and completeness for the SPS-2 sites are considered good overall. A high percentage of the SPS-2 data is at level E—greater than 82 percent for all data types, and greater than 99 percent for many.

However, a significant amount of data is still missing, especially traffic, distress and faulting surveys, and key materials testing data. These deficiencies need to be addressed before serious analysis of the SPS-2 experiment can occur.

The SPS-2 data deficiencies are summarized below:

- Wisconsin—newly constructed, data processing under way and all data expected to be complete.
- Arizona, Arkansas, and North Carolina—late initial survey for most monitoring types. Backcasting of IRI and distress data will be required.
- Colorado and North Dakota—late initial survey for one monitoring collection activity, either longitudinal profile measurements, deflection testing, faulting, or distress data.
- Kansas SPS-2—very deficient faulting data. Faulting measurements currently under way.
- Traffic data are very deficient for 5 of 13 sites (40 percent).
- Joint faulting data are not being collected as required by LTPP, and this will limit the analyses that can be conducted.
- Arkansas, Kansas, North Carolina, and Wisconsin are missing significant PCC materials testing data.

Note that the LTPP program is embarking on a systemwide effort to resolve all missing data. Some missing data have already been obtained. This effort will greatly improve the data availability for future analysis.

EXPERIMENTAL DESIGN VERSUS ACTUAL CONSTRUCTION

Required experimental design factors were compared with the actual constructed values stored in the IMS database. This database includes both the site condition factors and pavement design features. Most SPS-2 sections follow the experiment design for the large majority of the design factors. Most deviations from the experiment design are found in the concrete slab thickness and 14-day flexural strength. A number of sections were more than 12 mm too thick or too thin, or were more than 10 percent too high or too low in flexural strength.

Out of the 13 SPS-2 project locations, only the recently constructed Wisconsin site does not have enough data in the IMS database to be evaluated. These data are currently in the pipeline. Of course, data from the California site under construction also are not in the database at this time.

Eight projects can be characterized as good to excellent when comparing designed versus constructed values, while the remaining four projects are considered poor to fair.

In addition to the comparison of the designed versus constructed values from the IMS database, construction reports of from all 13 SPS-2 reports sites were reviewed to identify construction deviations and difficulties. Four projects were found to have experienced more than minor construction difficulties or deviations.

The following summarizes significant deviations when comparing the designed versus constructed factorial factors, and more than minor construction deviations:

- Delaware—significant deviations in both slab thickness and flexural strength. This project is also considered to have moderate construction deviations.

- Iowa—significant deviations in flexural strength, slab thickness, and lane width. The lane width deviation may be a data entry error. This project is also considered to have moderate construction deviations.
- Michigan—moderate construction deviations.
- Nevada—significant deviations in subgrade type, flexural strength, and slab thickness. This project is also considered to have moderate construction deviations. Extensive cracking occurred early, and one section has been taken out of service.
- Washington—significant deviations in flexural strength and slab thickness. The traffic level for 1997 appears to be erroneous.

EARLY PERFORMANCE TRENDS

The SPS-2 sections are relatively young, and a large majority show little distress. As of January 2000, only 43 of 155 sections (28 percent) are showing any noticeable distresses. Ninety-five percent of the SPS-2 sections have less than 1 mm of edge joint faulting. Eighty-seven percent of the SPS-2 sections show zero transverse cracking, and 78 percent of the sections have zero longitudinal cracking.

Based on the initial statistical analyses and comparisons, the following initial performance trends are noted (note that long-term performance may be different from short-term performance):

- The initial IRI of SPS-2 sections after placement ranged from 0.76 to 2.19 m/km with a mean of 1.30 m/km.
- JPCP constructed on coarse-grained soil were smoother (lower initial IRI) than those constructed on fine-grained soils.
- JPCP constructed on PATB were smoother than sections constructed on LCB or untreated aggregate base.
- The IRI trend over time depends heavily on the initial IRI, the traffic loadings, and the extent of joint faulting.
- Sections with PATB show the lowest total longitudinal cracking levels, while the sections with LCB show the highest longitudinal cracking.
- Thinner (203 mm) slabs show more longitudinal cracks. Sections with a thinner slab and widened slab show the highest level of longitudinal cracking.
- Sections with PATB show the lowest percentage of slabs cracked transversely, while the sections with an LCB show the highest transverse cracking.
- Thinner (203 mm) slabs show more transverse cracks than thicker slabs. Sections with a thinner slab and a widened slab show the highest level of transverse cracking.
- Sections with aggregate base show the highest joint faulting level. Sections with LCB and PATB have the lowest joint faulting.
- Widened slab sections show less faulting than conventional width slabs.

STATES' EXPECTATIONS FROM THE STATES FOR THE SPS-2 EXPERIMENT

Two national workshops were held where input was received from the States on the SPS-2 project. The meetings were held on November 2-3, 1999, in Columbus, OH, and on April 27, 2000, in Newport, RI. The research team made presentations at both conferences about the status of SPS-2 data collection, analysis, and availability, and the near- and long-term LTPP products. Several participating States made presentations on the status and analyses of their SPS-2 projects, as well as the States' expectations of the SPS-2 experiment. There were many discussions of the future directions of the SPS-2 experiment and the analyses of the SPS-2 data at both conferences. Those discussions are summarized below.

In general, the States are satisfied with the SPS-2 experiment and fully expect to get valuable information about different design features from the project. Many States have been conducting or planning their own analyses of their SPS-2 projects. Some analyses have already yielded useful results. The States would like to see a focus on implementation of SPS-2 findings as they evolve over time.

First and foremost, what the States want to know the effect on pavement performance and cost-effectiveness of the SPS-2 experimental factors, such as:

- Drainage and base type.
- Widened lanes.
- Slab thickness.
- Concrete strength.

In addition to the structural design features, the States also would like to know what major site condition factors influence performance of concrete pavement, including:

- Climate.
- Traffic volume and loading.
- Subgrade type properties.
- Embankment.

Other specific expectations from the States include:

- Evaluation of existing performance prediction equations (i.e., AASHTO formulas).
- Validation of pavement analysis models.
- Mechanistic/empirical calibration of lab materials properties, backcalculated materials properties, and WIM data.
- Effects of soil type, base type, drainage, and climate on long-term subgrade moisture gradients.
- Cost-effectiveness of drainable bases, underdrains, high-strength concrete, different base types, and other features.

- Dynamic load response of the concrete pavements (response of PCC to single, tandem, and tridem axles; effect of vehicle speed on dynamic response of PCC).
- Using stiffness rather than density for subgrade acceptance.

As to the future analysis of SPS-2 data, the States believe that it is worthwhile to complete the missing data (backcast if necessary) to obtain traffic and materials data. Participants believed that many fundamental studies can be conducted to see how SPS-2 sections are responding to load and environmental stresses. It was also suggested that an integrated analysis plan is needed for future research.

CAN SPS-2 MEET EXPECTATIONS?

The specific experiment expectation of the SPS-2 project was to determine the main effects and interactions of the following key design features:

- Slab thickness.
- Concrete strength (14 day).
- Base type including PATB, LCB, and untreated aggregate base.
- Lane width.

These main effects and interactions were to be determined for each of the following subgrade and climatic conditions:

- Fine-grained and coarse-grained subgrade soils.
- Wet freeze, wet no-freeze, dry freeze, and dry no-freeze climates.

This evaluation has identified several significant problems that will limit the results that can be obtained from the SPS-2. Specifically, SPS-2 project sites are missing for certain subgrade–climate combinations. Some SPS-2 sites had construction deviations. Significant materials data and traffic data are missing from some sites or sections. The missing traffic data and key materials data must be obtained or forecasted before meaningful global analysis can be performed. The Nevada site has excessive early cracking that will limit its usefulness.

However, these problems do not mean that many important and useful findings and results cannot be obtained from SPS-2 experiments. Some interesting and important early trends have already been identified that will be useful in the design and construction of JPCP, even though the sections are only a maximum of 7.5 years old. As time and traffic loadings accumulate on the SPS-2 sites, additional valuable performance data will be obtained.

Because of FHWA's intense ongoing effort to obtain missing data (construction, materials, traffic, and monitoring), valuable results can be obtained from the SPS-2 sites. It is further believed that even more results can be obtained if a concerted effort is made to perform proper analyses of the data.

RECOMMENDATIONS FOR THE SPS-2 EXPERIMENT

Missing SPS-2 Sites. To complete all the SPS-2 factorial cells, construction of the following sites is recommended:

- Two sites are needed to complete the factorial in the dry no-freeze climate: one site with a fine-grained subgrade, and the other site with a coarse-grained subgrade (i.e., Arizona, California in the southwest United States).
- Two sites are needed to complete the factorial and match sites in the wet no-freeze climate with a coarse-grained subgrade (southeast United States).
- One site is needed to complete the factorial and match a dry freeze climate with a fine-grained subgrade (north central United States).

Missing SPS-2 Data. Significant effort is recommended to obtain the following missing data:

- Materials—PCC strength.
- Traffic—5 sites completely missing traffic data, 11 sites missing continuous WIM data.
- Faulting—collect faulting data from several SPS-2 sites immediately.

Expectations from SPS-2. The overall objective is for the SPS-2 project's results to provide SHAs with documented findings to help them improve their management, design, construction, and materials procedures related to JPCP. The following specific information is expected to be gained from the SPS-2 project:

Specific design, subgrade, climate, and traffic effects

- Effect of subdrainage on performance (faulting, transverse cracking, longitudinal cracking, IRI).
- Effect of widened slab on performance (faulting, transverse cracking, longitudinal cracking, IRI).
- Effect of base type (lean concrete, permeable asphalt, dense aggregate) on performance (faulting, transverse cracking, longitudinal cracking, IRI).
- Effect of concrete slab thickness on performance (faulting, transverse cracking, longitudinal cracking, IRI).
- Effect of 14-day concrete strength on performance (faulting, transverse cracking, longitudinal cracking, IRI).
- Effect of climatic region on performance (precipitation, temperature).
- Effect of subgrade soil on performance (fine-grained, coarse-grained).
- Interactive effect of subdrainage, widened slab, base type, slab thickness, concrete strength, climatic region, and subgrade soil on performance.
- Effect of traffic loading on performance of various design treatments.

Data for use in calibration of mechanistic-empirical distress models

- *2002 Design Guide* distress models.
- Subsequent improvement in future versions of the guide over time.

Data for use in empirical performance modeling (for pavement management)

Data for use in a variety of mechanistic modeling (backcalculation, structural analysis)

Data for use in a variety of cost/benefit analyses

- SPS-2 performance data are ideally suited for use in cost and benefit studies to determine the relative cost-effectiveness of each design feature in various climates and subgrades.

Future Data Ccollection. It is recommended that the following areas receive special emphasis in SPS-2 data collection:

- Routine data collection.
  - WIM and AVC traffic monitoring: ensure that LTPP guidelines are followed.
  - Joint faulting: follow LTPP guidelines closely.
  - Resolve irregular distress measurements over time for each SPS-2 section (variations of distress quantities over time).
  - Thermal coefficient of expansion of concrete: ensure that all SPS-2 sites are tested in 2000.
- Collect new data required for *2002 Design Guide* calibration.
  - Slab curvature measurements: measure slab curvature when the thermal gradients are zero. Measure this during two seasons of year, wet and dry.
  - Conduct video surveys of edge drains to ensure they are working, and schedule maintenance if needed.
  - Cores along the cracks in JPCP to determine the initiation of the crack and the direction of its propagation. In other words, where did the crack initiate: top-down or bottom-up?

RECOMMENDED FUTURE ANALYSES FOR SPS-2 EXPERIMENT

As stated previously, a very small percentage of the SPS-2 test sections currently have significant levels of distress, and only a few have been taken out service. The real benefit from this experiment will occur over the next 15 years, as more and more test sections exhibit higher levels of distress, magnifying the effect of the experimental and other structural factors on performance.

This report focuses on the quality and completeness of the SPS-2 construction and monitoring data and on the adequacy of the experiment to achieve the original expectations and objectives.

Detailed analysis of the effect of different design factors on performance was outside the scope of work for this study. Thus, future studies using the SPS-2 experimental data should be planned and prioritized so they can be initiated as the SPS-2 projects exhibit higher levels of distress.

These future studies should be planned for in two stages that focus on local and national expectations from the experiment. The first stage is to conduct a detailed assessment or case study on each experimental cell in the project (companion SPS-2 sites that define a full factorial experiment) to ensure data adequacy, assess construction deficiencies, and support local interests and expectations. The second stage evaluates the effect of different structural features across the entire national experiment. Both analysis stages are briefly discussed in the following sections. After the sections are 15 to 20 years of age, a third-stage analysis will ultimately be needed to fully reap the benefits of the SPS-2 experiment.

Initial Stage—Analysis of Individual Factorial Cells

Each major cell in the SPS-2 experiment consists of at least two companion projects. One of these companion projects contains experimental sections 1 through 12, and the other contains sections 13 through 24. These companion SPS-2 sites constitute a full factorial of design factors and make it possible to evaluate the main effects and interactions of each experimental factor for those site conditions. A detailed evaluation of the companion projects within each major cell should be completed as soon as possible to ensure that all of the data exist and are acceptable. The purposes of the case studies in the first stage are listed below:

- Resolve construction and monitoring data anomalies and experimental cell differences for those projects that changed cell locations from the original experiment design, as they relate to the specific cell in the experiment.

- Conduct comparative analyses of the individual test sections at each site, *including the supplemental test sections*, to identify differences in pavement performance and response. These comparative studies should include performance measures, material properties, and as-built conditions.

- Determine the effect of any construction difficulties and problems and material noncompliance issues with the SPS-2 project specifications, if any, on pavement performance and response at each site.

- Develop findings regarding comparisons made between the companion projects and test sections and prepare a case study report that will be useful for the SHAs involved. Such information will also be useful for the national studies.

This first-stage analysis is considered absolutely essential prior to initiation of the second-stage analyses.

Second Stage—Analysis of Experimental Findings

The second-stage analyses should not be pursued until the first-stage analysis has been completed. It is expected that the analyses performed at this stage will be coordinated with the *Strategic Plan for LTPP Data Analysis*. The SPS-2 experiment can contribute to the following specific analyses outlined in the strategic plan:

- Develop relationships to enable interchangeable use of laboratory- and field-derived material parameters (Strategic Plan No. 2B).
- Establish procedures for determining as-built material properties (2C).
- Identify quantitative information on the performance impact of different levels of material variability and quality (2D).
- Estimate material design parameters from other materials data (2E).
- Quantify information as to the relationship between as-designed and as-built material characteristics (2F).
- Develop recommendations for climatic data collection to adequately predict pavement performance (3D).
- Develop models relating functional and structural performance (4C).
- Calibrate relationships (transfer functions) between pavement response and individual distress types (5C).
- Identify quantitative information on the impact of design features on measured pavement responses (deflections, load-transfer, strains, etc.) (7A).
- Identify quantitative information on the impact of design features on pavement distress (7B).
- Develop guidelines for the selection of pavement design features (7C).

In summary, the following future analysis objectives are recommended for the SPS-2 experiment. These analysis topics are discussed in more detail in figures 25 through 31.

1. Perform site-by-site analyses of SPS-2 projects to resolve data problems and gain understanding of performance of individual test sections (figure 25).
2. Determine the effect of the SPS-2 experimental factors on the performance of the jointed plain concrete pavements (figure 26).
3. Determine the optimum JPCP design features for specific site conditions and traffic loading (figure 27).
4. Determine the effect of concrete slab thickness variations on LTPP and initial ride quality (figure 28).
5. Calibrate and validate relationships (transfer functions) between pavement structural response and individual distress types (figure 29).
6. Conduct mechanistic analyses of SPS-2 sites (particularly Ohio and North Carolina) to gain knowledge of critical stresses and deflections to explain their performance in terms of joint faulting and slab transverse and longitudinal cracking (figure 30).
7. Conduct cost-benefit analyses of SPS-2 data to determine the cost-effectiveness of various design features (figure 31).

The full results from the SPS-2 experiment will require 20 years of monitoring for the majority of sections. Additional studies beyond these proposed will be required.

<table>
<tr><td colspan="2">OBJECTIVE NO. 1
Perform site-by-site analyses of SPS-2 projects to gain understanding of performance of individual test sections. (Initial stage, expected timeframe 2001 to 2002)</td></tr>
<tr><td>TOPIC AREA
Pavement design.</td><td>PROBABILITY OF SUCCESS
High.</td></tr>
<tr><td>LTPP STRATEGIC PLAN
7A, 7B, and 7C.
(Study of the Experimental Factors)</td><td>SUPPLEMENTAL EXPERIMENTS
None.</td></tr>
<tr><td>END PRODUCT
• Identification of test sections that perform well and poorly at each SPS-2 site, including supplementals.
• Determination of the effect of any construction difficulties and material noncompliance issues on pavement performance and response.</td><td>POTENTIAL PRODUCT USE
Design of new or reconstructed cost effective and reliable jointed plain concrete pavements.</td></tr>
<tr><td colspan="2">GENERAL TASKS
• Conduct evaluation of permeable base and edge drains and outlets to determine their proper construction performance, and maintenance.
• Resolve construction and monitoring data anomalies and experimental cell differences for those projects that changed cell locations from the original experiment design, as they relate to the specific cell in the experiment.
• Conduct comparative analyses of the individual test sections at each site, including the supplemental test sections, to identify differences in pavement performance and response.
• Determine the effect of any construction difficulties, problems, and material noncompliance issues with the SPS-2 project specifications, if any, on pavement performance and response.
• Develop findings regarding comparisons made between the companion projects and test sections and prepare a case study report that will be useful for the State highway agencies involved and also will be useful for the national studies.</td></tr>
</table>

Figure 25. Recommended future analyses for SPS-2—Site-by-site analyses of SPS-2 projects to gain understanding of performance of individual test sections (initial stage).

OBJECTIVE NO. 2 *Determine the effect of the SPS-2 experimental factors on the performance of the jointed plain concrete pavements. (Expected timeframe 2003 to 2006)*	
TOPIC AREA Pavement design.	***PROBABILITY OF SUCCESS*** High (assuming that subdrainage was evaluated in Objective No. 1).
LTPP STRATEGIC PLAN 7A, 7B, and 7C.	***SUPPLEMENTAL EXPERIMENTS*** None.
END PRODUCT • Effect of a permeable base drainage system on the performance of the jointed plain concrete pavements. • Effect of different base types on the performance of the jointed plain concrete pavements. • Effect of widened lane on the performance of the jointed plain concrete pavements. • Identification of site conditions where thicker concrete slab will and will not contribute to improved performance. • Effect of thicker slabs on the performance of the jointed plain concrete pavements. • Effect of higher strength concrete on the performance of the jointed plain concrete pavements. • Identification of site conditions where these design features will contribute to improved performance of the jointed plain concrete pavements.	
POTENTIAL PRODUCT USE Design of new or reconstructed cost effective and reliable jointed plain concrete pavements.	
GENERAL TASKS • Review results and findings from each SPS-2 site. • Conduct statistical analysis to determine significant factors and interactions on performance. • Conduct mechanistic-empirical analyses for cracking, joint faulting, and IRI. • Based on statistical and mechanistic analyses, determine the effect of different experimental factors or design features and interaction on pavement performance and response. • Prepare practical presentations of the results, including software, decision trees, etc., for use by practicing engineers, that aid them in determining the end products above.	

Figure 26. Recommended future analyses for SPS-2 experiment—study of the effect of the experimental factors on rigid pavement performance.

OBJECTIVE NO. 3 *Determine the optimum design features for specific site conditions and traffic loading for JPCP. (Expected timeframe 2003 to 2005)*	
TOPIC AREA Pavement design.	***PROBABILITY OF SUCCESS*** High.
LTPP STRATEGIC PLAN 7A, 7B, and 7C. (Study of the Experimental Factors)	***SUPPLEMENTAL EXPERIMENTS*** GPS-3 and SPS-8.
END PRODUCT A guideline, catalog, or a design tool for selecting optimum combinations of design features for specific site conditions and traffic level.	***POTENTIAL PRODUCT USE*** Design new cost effective and reliable jointed plain concrete pavements.
GENERAL TASKS • Review results from each SPS-2 site. • Conduct statistical analyses to determine significant factors and interactions. • Conduct mechanistic-empirical analyses for transverse cracking, joint faulting, and IRI for JPCP. • Obtain representative construction cost data for all needed features of JPCP over selected regions that include an SPS-2 experiment. • Based on statistical and mechanistic analyses, identify the optimum combination of pavement design features to be used for various site conditions to provide cost effective and reliable JPCP. • Prepare practical presentations of the results, including software, guidelines, catalogs, and other tools that aids practicing engineers in determining the end products above.	

Figure 27. Recommended future analyses for SPS-2 experiment—determination of the optimum pavement design features.

OBJECTIVE NO. 4 *Determine the effect of concrete slab thickness variations on long-term pavement performance and initial ride quality. (Expected timeframe 2005 to 2007)*	
TOPIC AREA Pavement design and construction.	***PROBABILITY OF SUCCESS*** High.
LTPP STRATEGIC PLAN 2C and 2F.	***SUPPLEMENTAL EXPERIMENTS*** GPS-3 and SPS-8.
END PRODUCT A relationship between increased thickness variations and reduced pavement service life or reduced initial ride quality.	***POTENTIAL PRODUCT USE*** Develop pay reduction factors based on concrete slab thickness variation.
GENERAL TASKS • Review results from each SPS-2 site. • Establish the variation in the concrete slab thickness for each of the SPS-2 test sections. • Conduct statistical analyses to determine the effect of the slab thickness variation on pavement performance and response. • Develop reductions in service life based on these increased variations in concrete slab thickness.	

Figure 28. Recommended future analyses for SPS-2 experiment—quantify the relationships between as-designed and as-built concrete slab thickness and strength.

OBJECTIVE NO. 5 *Calibrate and validate relationships (transfer functions) between pavement response and individual distress types. (Expected timeframe 2005 to 2007)*	
TOPIC AREA Pavement design.	***PROBABILITY OF SUCCESS*** High.
LTPP STRATEGIC PLAN 7A, 7B, and 7C.	***SUPPLEMENTAL EXPERIMENTS*** GPS-3 and SPS-8.
END PRODUCT A calibrated and/or validated relationship between pavement structural responses (stress) and individual distresses.	***POTENTIAL PRODUCT USE*** Design of new cost effective and reliable jointed concrete pavements (would contribute to upgrading of *2002 Design Guide*).
GENERAL TASKS • Establish a comprehensive input database that includes design, construction, materials testing, traffic, climatic, and monitoring data, for the response model. • Perform mechanistic analysis to determine the critical response stress and cumulative fatigue damage for the traffic loading applied until the time of the distress measurement (utilize the relationships in the *2002 Design Guide* as well as others). • Establish the relationships between the cumulative fatigue damage and the measured distress. • Perform model assessment and develop calibration coefficients.	

Figure 29. Recommended future analyses for SPS-2 experiment—calibration and validation of the pavement transfer functions.

OBJECTIVE NO. 6 *Conduct mechanistic analyses of SPS-2 sites (particularly Ohio and North Carolina) to gain knowledge of critical stresses and deflections to explain their performance in terms of joint faulting and slab transverse and longitudinal cracking. (Expected time frame 2005 to 2007)*	
TOPIC AREA Pavement design and construction.	***PROBABILITY OF SUCCESS*** Moderate to high.
LTPP STRATEGIC PLAN 2D and 7B.	***SUPPLEMENTAL EXPERIMENTS*** None.
END PRODUCT In-depth, field-verified knowledge as to the effects of critical measured structural responses and curling that will be useful in pavement design, evaluation, and rehabilitation.	***POTENTIAL PRODUCT USE*** Knowledge gained from this experiment will be useful to researchers and others for improving design procedures to make JPCP a more cost effective and reliable pavement (upgrade *2002 Design Guide*).
GENERAL TASKS • Establish a comprehensive input database that includes design, construction, materials testing, traffic, climatic, monitoring data, and structural monitoring data (deflections, strains, stresses, others). • Analyze slab curling at all sites using longitudinal profile data or other slab curling measurements available (Note: if insufficient data are available, measure curling at several sites in different climates). • Perform mechanistic analysis to determine the critical response stress and cumulative fatigue damage for the traffic loading and slab curling. • Analyze results and develop findings and recommendations as to impacts of loading and curling on JPCP performance (cracking and faulting).	

Figure 30. Recommended future analyses for SPS-2 experiment—mechanistic analyses of JPCP.

OBJECTIVE NO. 7 *Conduct cost/benefit analyses of SPS-2 sites to gain knowledge of the cost-effectiveness of design features in different site conditions. (Expected timeframe 2005 to 2007)*	
TOPIC AREA Pavement design and construction.	***PROBABILITY OF SUCCESS*** High.
LTPP STRATEGIC PLAN 7B and 7C.	***SUPPLEMENTAL EXPERIMENTS*** None
END PRODUCT In-depth, field-verified knowledge as to the cost-effectiveness of key design features including slab thickness, widened slab, base type, concrete strength, and a permeable base layer.	***POTENTIAL PRODUCT USE*** Knowledge gained from this experiment will be directly useful to pavement designers in improving the cost-effectiveness of their designs.
GENERAL TASKS • Establish a comprehensive input database that includes design, construction, materials testing, traffic, climatic, and monitoring data. • Establish typical costs of various design features from the State highway agencies in the States where SPS-2 sites are located. • Analyze results and develop findings and recommendations as to the cost-effectiveness of each design feature in each of the main climatic zones covered by the SPS-2 experiment.	

Figure 31. Recommended future analyses for SPS-2 experiment—cost/benefit analyses of JPCP.

APPENDIX A. SUMMARY OF SPS-2 PROJECT NOMINATION AND CONSTRUCTION GUIDELINES

To ensure proper selection of the SPS-2 sites and to maximize the uniformity of the design and construction details at all SPS-2 sites and sections, the following two documents were prepared for the SPS-2 experiment:

- *Specific Pavement Studies Guidelines for Nomination and Evaluation of Candidate Projects for Experiment SPS-2 Strategic Study of Structural Factors for Rigid Pavements*(11)
- *Specific Pavement Studies Construction Guidelines for Experiment SPS-2 Strategic Study of Structural Factors for Rigid Pavements* (12)

These guidelines were developed to control the quality and integrity of the SPS-2 experiment results and findings. A summary of the documents is provided in this appendix.

PROJECT SELECTION CRITERIA

The following criteria are to be used to evaluate candidate projects for inclusion in the SPS-2 experiment:

- The project must include new construction of all pavement lanes for a new route, realignment, reconstruction, or construction of an experimental parallel roadway. Projects in which the experimental sections are constructed as additional lanes or as a partial reconstruction (removal and replacement on surface layers only) are not acceptable.
- The construction project must be of sufficient length to accommodate all of the experimental test sections. Transition zones are required between test sections, and the length of these zones depends on site conditions such as location of cut and fills and drainage provisions. A minimum transition length of approximately 54.9 m should be provided between test sections.
- All test sections at one site must be constructed on soils classified as either fine-grained or coarse-grained. Further, it is desired that all of the test sections be located on subgrade soils of similar characteristics and classification. Variation in soil characteristics at each site should be minimized as much as possible.
- Test sections should be located on portions of the project that are relatively straight and have a uniform vertical grade. Horizontal curves greater than 3 degrees, and vertical grades greater than 4 percent, should be avoided. Left-hand horizontal curves in which superelevation forces surface water to flow toward the inside shoulder should be avoided. All test sections of a project must have the same transverse cross section profile of the pavement surface to obtain the same surface drainage conditions.
- Ideally, all test sections should be located on shallow fills. However, the entire length of each test section should be located completely on either a cut or a fill. Cut-fill transitions and side hill fills should be avoided.

- It is highly desirable that the portion of the project that includes the proposed test sections be opened to traffic at the same time.
- Culverts, pipes, and other structures beneath the pavement should be avoided within the limits of each test section. It is recommended that subsurface structures, if required, be located in the transition zones between test sections.
- It is desired that the project be located on a route with an expected traffic loading level in the study lane in excess of 200,000 ESALs per year. However, projects on the primary system with high traffic relative to the region, but less than the desired rate, will be considered.
- Traffic flow over all the test sections of a project should be uniform. All sections should carry the same traffic stream. Intersections, rest stops, on-off ramps, weaving areas, quarry entrances, and so on must be avoided on and between test sections on a project.

These criteria and considerations will help identify projects in which the relative performance of the test sections is due to the design parameters used and not to external factors (e.g., changes in the subgrade or traffic patterns). They also serve to identify projects at different locations with relatively similar details so that differences in performance from one location to another are primarily due to differences in climatic conditions and subgrade types.

It is recognized that "perfect" projects containing all of the desirable characteristics are rare. Each proposed site will be evaluated individually and compared to other candidates in order to select the best set of projects to satisfy experimental considerations. Some deviation from the desired project characteristics may be necessary to obtain sufficient projects for the experiment. For example, projects will be considered where it is not possible to locate all of the test sections completely in cuts or on fills. In this case, it may be necessary to locate some test sections in cuts and others in fills.

PREPARATION AND COMPACTION OF SUBGRADE

Ideally, the test sections shall be located in shallow fills. However, if the test section cannot be placed in a fill, the entire length of the section shall be located completely in a cut section. Cut-fill transitions or side hill fills should not be located within a test section. In addition, rock cut sections should be avoided unless all test sections are located within the cut.

Subgrade soils shall be prepared according to the following requirements:

- The subgrade soil shall be tested according to AASHTO T99, method D, to determine the moisture-density relationship.
- Fill material shall be compacted to a minimum of 95 percent of AASHTO T99 density for the top 305 mm. Expansive soils shall be compacted to a minimum of 90 percent of AASHTO T99 for the top 305 mm.
- Moisture content of the compacted subgrade soil should be in the range of 85 percent to 120 percent of the optimum moisture content.
- Sections built as part of a reconstruction project shall have the upper 305 mm of subgrade compacted to the appropriate specification.

- Subgrade shall be compacted for the width of the travel lanes plus the width of the inside and outside shoulders, except where sections are built as part of reconstruction of an existing pavement. In this case, reconstruction must extend a minimum of 914 mm outside the edge of the travel lanes to allow proper preparation of the subgrade and base course.
- Where sections are constructed on newly placed fill material, the thickness of the fill should be as uniform as possible along the test section. Geotextile reinforcement shall not be used to stabilize the subgrade.
- Proof rolling should be performed to verify the uniformity of support and to identify unstable areas that might require remedial construction (undercutting and replacement).
- Surface irregularities shall not exceed 6 mm between two points longitudinally or transversely using a 3.05-m straightedge.
- Final subgrade elevations shall not vary from design more than 12 mm, based on rod and level survey readings taken at a minimum of five locations (edge, outer wheel path, midlane, inner wheel path, and inside edge of lane) at longitudinal intervals no greater than 15.25 m.
- Modifiers, lime, portland cement, and the like can be added to provide a stable working platform to facilitate construction. The use of modifiers shall be limited to materials and quantities that will alter the index properties of the subgrade (e.g., reduce the plasticity index) without unduly increasing the strength of the subgrade in the pavement structure. Working platforms consisting of thin asphalt concrete layers placed directly on subgrade are not permitted.

Note: The working platform is considered a pavement layer; therefore, sampling and testing, in addition to that required for the subgrade, must be planned and performed.

BASE LAYERS

The discussion of construction guidelines for base materials is divided into two categories: undrained and drained base structures. The drained and undrained designations do not refer to external pavement drainage features such as cross-slope and ditches. Undrained base structures refer to relatively impermeable dense graded base layers consisting of DGAB or lean concrete LCB. The drained base structures refer to a system that consists of PATB drainage layer and edge drains.

Undrained Base Layers

Sections 1 through 8 and 13 through 20 of the primary experiment (25 through 28, 31 through 34, 37 through 40, and 45 through 48 of the supplemental experiments) are constructed with undrained base layers that incorporate DGAB or LCB. Drainage layers and longitudinal edge drains shall not be used on these sections.

Dense-Graded Aggregate Base

DGAB is an untreated, crushed material. Requirements and construction guidelines for this material are presented in the following sections.

Aggregate Requirements

The quality and gradation criteria for selection of the aggregate required in the construction in the DGAB shall be as follows:

- The base material must consist of a high quality crushed stone, crushed gravel, or crushed slag.
- The base aggregate shall consist of a minimum of 50 percent of material retained on the No. 4 sieve. Of the particles retained on the 8-mm (3/8-in) sieve, at least 75 percent shall have two or more fracture faces.
- A 38-mm top size aggregate is preferred; however, the maximum top size normally specified by the State agency, if less than 38 mm, may be used.
- The final aggregate mixture must be dense graded.
- The fraction passing the No. 200 sieve shall be less than 60 percent of the fraction passing the No. 30 sieve and not more than 10 percent of the total sample.
- The fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and plasticity index not greater than 4.

- Aggregate tested with L.A. Abrasion™, which shows loss of more than 50 percent at 500 revolutions, shall not be used.
- No additives, other than water, are allowed in the DGAB.

Construction Requirements

The base course shall be prepared to grade according to the participating agency's practice and the following requirements:

- No segregation or degradation of materials should occur during laydown and compaction. Areas of excessive segregation shall be removed and replaced with proper aggregate.
- Maximum lift thickness shall be 152 mm compacted.
- Maximum dry density and optimum moisture content shall be determined by AASHTO T180, method D.
- DGAB course must be compacted to an average of not less than 95 percent of AASHTO T180 density.
- The DGAB shall be compacted for the width of the travel lanes plus the width of the inside and outside shoulders, except in cases where sections are built as part of reconstruction of an existing pavement. In this case, reconstruction must extend a minimum of 914 mm outside the edge of the travel lanes to allow proper preparation of the subgrade and base course.
- For those sections incorporating a PATB layer, a DGAB base course will be constructed over the subgrade prior to placement of the PATB. Low-viscosity asphalt shall be used to prime the surface of the DGAB before placing the PATB. Application and curing will be according to the participating agency’s practice.

- In-place density for purposes of construction quality control shall be measured and recorded prior to application of an asphalt cement prime coat (in drained sections), if used.
- Prior to the placement of the PCC surface layer, the DGAB shall be kept uniformly moist; however, the method of moistening shall not be such as to form mud or pools of water.
- Surface irregularities shall not exceed 6 mm between two points longitudinally or transversely using a 3.05-m straightedge.
- Final DGAB elevations shall not vary from design more than 12 mm, based on a rod and level survey conducted taking readings at a minimum of five locations (edge, outer wheel path, midlane, inner wheel path, and inside edge of lane) at longitudinal intervals no greater than 15.25 m.

Lean Concrete Base

The LCB shall consist of a mixture of aggregate, hydraulic cement, water, and admixtures. The variability in specifications used by the different highway agencies makes it impractical to specify the same materials or mix design for all test locations. Therefore, the participating agency's procedures and specifications shall be used to produce and place an LCB with a target average compressive strength, slump, and air content as follows:

- Compressive strength—3.4 MPa (5.2 MPa maximum) at 7 days.
- Slump (slip-formed paving) —25 to 76 mm.
- Air content—4 to 9 percent.

Material Properties

Cement and aggregate used in producing the LCB shall meet the following requirements:

- Only Type I or Type II portland cement shall be used and shall meet the requirements of AASHTO specification M85.
- Coarse aggregate (retained on the No. 8 sieve) shall consist of crushed gravel or crushed stone particles meeting the requirements of AASHTO M80. It is recommended that the coarse aggregate meet the gradation requirements of AASHTO 57 gradation. The following specific requirements shall be met by the coarse aggregate:
    - Abrasion loss, maximum—50 percent.
    - Magnesium sulfate ssoundness, maximum—12 percent.
    - Crushed particles, minimum—55 percent.

It is important that the coarse aggregate meet the highest standard of durability specified by the agency. Coarse aggregate must be reasonably free from deleterious substances such as chert, gypsum, iron sulfide, amorphous silica, and hydrated iron oxide, and must be obtained from a source approved by the agency. Coarse aggregate for use in LCB that will be subject to wetting or extended exposure to moist ground shall not contain any materials that are deleteriously reactive with alkalies in the cement in an amount sufficient to cause excessive expansion of mortar or concrete. The potential reactivity should be determined in accordance with the procedure given in AASHTO M80.

Construction Requirements

Construction requirements for the LCB include the following:

- LCB shall be 152 mm thick.
- For new construction, the LCB layer will be constructed the full width of the travel lanes plus the width of the inside and outside shoulders. For sections built as part of reconstruction (inlay), the LCB will be placed to a width not less than 914 mm outside the edges of the travel lanes.
- Wax-base curing compound (AASHTO description: M 148, Type 2) shall be used at a rate of 4 liters per 10 m^2. A second coat of curing compound shall be applied within 24 hours before concrete placement at a rate of 4 liters per 15 square meters.
- The LCB surface shall not be textured and shall be finished to a smooth surface, free from mortar ridges and other projections, before the curing compound is applied.
- Final LCB elevations shall not vary from design more than 12 mm based on a rod and level survey. Readings shall be taken at a minimum of five locations (edge, outer wheel path, midlane, inner wheel path, and inside edge of lane) at longitudinal intervals no greater than 15.25 m.
- Surface irregularities shall not exceed 6 mm between two points longitudinally or transversely using a 3.05-m straightedge.
- LCB constructed in widths greater than 7.92 m shall be constructed with a longitudinal joint offset not more than 914 mm from the centerline of the width being constructed.
- A longitudinal joint in the LCB shall not be within 0.305 m of the planned longitudinal joint in the concrete pavement.
- Procedures normally used for placing concrete pavements shall be used for placing LCB. The use of slip-form paving is recommended.
- Traffic will not be allowed on the LCB surface for 7 days or until the compressive strength of the LCB has reached a minimum of 3.4 MPa. No traffic should be allowed onto the LCB after the second application of curing compound.

Drained Base Structures

Sections 9 through 12 and 21 through 24 of the primary experiment (and sections 29, 30, 35, 36, 41 through 44, and 49 through 52 of the supplementary experiments) are constructed with drained base structures that incorporate a PATB and edge drains. The PATB is constructed in combination with the DGAB materials previously described.

Permeable Asphalt-Treated Base

The PATB serves as a drainage layer in the pavement structure. Material and construction requirements for the PATB are presented below.

Material Requirements

The PATB material shall meet the following requirements:

- PATB shall be an open graded, hot laid, central plant mixed, asphalt base material.
- The use of asphalt cement emulsion in the mix is prohibited.
- An AASHTO No. 57 size stone, or such other gradation used by an agency as a highly permeable drainage material in pavement structures, shall be used. It is required that this gradation has no more than 2 percent passing the No. 200 sieve. The aggregate shall consist of crushed material having more than 90 percent with at least one fractured face.
- The mix shall be designed with an asphalt cement content of 2 to 2.5 percent.
- Additives or modifiers may be used to reduce stripping of asphalt if such use represents the participating agency's practice. Experimental additives or modifiers shall not be used in the sections.
- Asphalt grade and type may vary according to agency practice. Experience on early SPS-2 projects indicated good placement experience when using AC-30 for the PATB mix.
- No recycled asphalt concrete shall be permitted in the PATB.

Table 53. Gradation table

Sieve	Percent Passing
38 mm	100
25 mm	95-100
13 mm	25-60
No. 4	0-10
No. 8	0-5
No. 200	0-2

Construction Requirements

Construction requirements for the PATB include the following:

- A static steel wheel roller shall be used to compact the permeable base, applying 14.6 kN to 29.1 kN per meter of roller width.
- No portion of the PATB layer shall be daylighted.
- Appreciable amounts of distortion shall be avoided on the permeable base.

- A roller may be used immediately in front of the paver to dress up the permeable base, if required.
- A track-mounted paver is strongly recommended for operation on the permeable base. It has been the experience on early projects in this experiment that the PATB may be sufficiently stable, after cooling or with the use of stiffer asphalt grades or modifiers, to allow wheeled pavers and construction trucks to operate on the PATB surface. However, sharp turning movements do cause significant distortion and should be avoided.

- Other than the paver and roller, no other equipment or vehicles should be allowed to operate or park on the travel lane or outside shoulder portion of the permeable base. Limited operation of equipment on the inside lane may be permitted. The use of side-dump delivery for layers constructed on the PATB should be encouraged to minimize damage to the PATB layer. Limited construction traffic (with reduced loads) may be allowed if the contractor is cautioned that excessive shoving and tearing of the PATB surface will be cause for prohibiting traffic. This requirement is intended to prevent damage to the PATB layer, which would affect layer thicknesses in subsequent layers, and also to prevent damage to the drainage properties of the finished PATB layer.
- Transverse interceptor drains shall be installed on the down slope end of the permeable base layers. They shall be placed in the transition zone between drained and undrained base structure test sections. They should be placed at least 30.5 m past the end of the 152.5-m monitoring section, or in the center of transitions, which are shorter than 30.5 m. The interceptor drains shall be placed along the midlength of a slab panel and will not be placed along a transverse joint.

Filter Fabrics

Filter fabric (or geotextiles) shall be used to prevent the clogging of the permeable material in the edge drains and transverse interceptor drains due to the migration of fine aggregates from untreated layers, the shoulder, and the subgrade. The requirements for the filter fabrics used in the edge drains are given below.

Material Requirements

Nonwoven or woven geotextile materials, which conform to recommendations for Class B drainage applications where installation stresses are low, will be used in edge drains. Fabric for the transverse interceptor drains shall meet Class A requirements. The following physical requirements on an average per roll basis sampled in accordance with ASTM D4354 shall be met.

Table 54. Geotextile material properties.

Property	**Minimum Value**		Test Method
	Class A	**Class B**	
Grab strength, N	800 (180)	355 (80)	ASTM D 4632
Puncture strength, N	355 (80)	111 (25)	ASTM D 3787
Trapezoid Teartear, N	222 (50)	111 (25)	ASTM D 4533
Burst strength, kPa	2000 (290)	896 (130)	ASTM D 3786
Permeability	$k_{fabric} > k_{soil}$		ASTM D 4491
Apparent opening size 1. Soil with ≤ 50% passing No. 200 sieve. 2. Soil with > 50% passing No. 200 sieve.	AOS < 0.6 mm > #30 U.S. std. sieve AOS < 0.3 mm > #50 U.S. std. sieve		ASTM D 4751

Construction Requirements

Construction requirements include the following:

- Filter fabrics shall extend around each of the edge drains. The filter fabric must extend around each edge drain and wrap around the outer edge of the PATB layer, but does not need to extend under the full width of the pavement. The fabric must wrap around the edges of the PATB layer, extending over the top of the PATB and under the pavement a minimum of 610 mm beyond the shoulder joint.
- Filter fabrics must be installed according to manufacturer's specifications and as shown in the typical drawings.
- Exposure of geotextiles to the elements between laydown and cover shall not exceed 14 days or manufacturer's manufacturer's specifications, whichever is less.
- Any filter fabric that is ripped or torn during the construction process shall be replaced or repaired with a patch that extends 914 mm beyond the perimeter of the tear or damage.
- Any filter fabric that is ripped or torn during the construction process shall be replaced or repaired with a patch which extends 914 mm beyond the perimeter of the tear or damage.
- Geotextile shall be overlapped a minimum of 610 mm at all longitudinal and transverse geotextile joints. Joints may be sewn if required by agency practice.

Edge Drains

Edge drains shall be used in the shoulders of the pavement sections with PATB to collect the water from the permeable base. The following requirements must also be met:

- Both inside and outside edge drains shall be constructed for crowned pavement cross-sections. For pavements with cross-slope, only one edge drain will be required.
- The edge drains should be located a minimum of 914 mm outside the edge of the mainline pavement.
- The edge drains shall run continuously throughout each of the 183-m minimum length permeable base test sections.
- The PATB is recommended as backfill in the edge drain trench; however, other approved open-graded material may be used.
- Collector pipes shall be a minimum 76-mm-diameter slotted plastic pipe and outlet pipes shall be a minimum 76-mm-diameter unslotted rigid plastic pipe. Pipes must be capable of withstanding the temperature of the PATB without damage if PATB is used as backfill.
- Transverse collector subdrains shall be located in transition zones between drained and undrained sections where a longitudinal slope exists. The drain should be installed at an acute angle relative to the downslope direction.

- Drainage pipes should be sized for the expected flows determined as part of design. Discharge outlet pipes should be located at maximum intervals of 76.2 m and rodent protected. Outlets must be at least 152 mm above the expected 10-year flow elevation of the collector ditches to prevent backflow into the drainage system.

Shoulders

For the SPS-2 experiment, participating agency practice shall be used to provide asphalt concrete or PCC shoulders. PCC shoulders shall not be tied to the mainline pavement. Also, if the concrete shoulder is placed monolithically with the traffic lanes, then the shoulder joint shall be sawed to full depth. Tied PCC shoulders may be constructed in additional supplemental test sections. The longitudinal joint between the mainline concrete pavement and the shoulder shall be sealed.

PORTLAND CEMENT CONCRETE MIX DESIGN

The quality of concrete as delivered, as placed, and the subsequent strength development in concrete are critical factors in concrete pavement performance. Although only the strength property (flexural strength) is normally considered in evaluating the structural behavior of concrete pavements, durability-related properties (entrained air content, aggregate type, degree of consolidation) are also important in evaluating long-term performance.

The test sections in the SPS-2 experiment will be constructed for two levels of flexural strength (3.8 and 6.2 MPa) as determined from third-point loading tests at 14 days. The concrete mixture should be designed according to the procedures and specifications followed by the participating agency. It is recommended that a slip-form method be used for placement of the concrete. In such a case, slump of the as-delivered concrete shall not exceed 64 mm.

Concrete with an average 14-day flexural strength of 3.8 MPa is considered standard and readily available. However, some agencies have reported difficulty in achieving this strength level while maintaining sufficient cement content for acceptable durability. In such cases, an average strength level of 4.1 MPa is considered acceptable for the lower strength level criterion. For the higher strength concrete, well-planned laboratory testing may be required to design a mixture capable of achieving an average flexural strength of 6.2 MPa at 14 days. The higher strength should be achievable by using a higher cement content (cement factor). Laboratory mix design will be in accordance with participating agency practice, except for the determination of the strength level.

The following is a summary of the requirements for the portland cement concrete PCC:

- Flexural strength—3.8 or 6.2 MPa average at 14 days.
- Slump (slip-form paving)—25 to 64 mm.
- Air content—6.5 ± 1.5 percent for freeze-thaw areas.

Materials

Material requirements for the concrete should be based on the normal practice of the participating highway agency. Many agencies have specific requirements for coarse and fine aggregates based on durability concerns and local availability of quality aggregates. However, it is necessary to maintain a high degree of uniformity and consistency in the construction of the test sections to achieve the objectives of a coordinated national experiment. Therefore, concrete materials must conform to certain minimum requirements to ensure consistency in the concrete quality at the different sites.

Portland Cement

Only Type I or Type II portland cement shall be used and shall meet the requirements of AASHTO specification M85.

Fly Ash

Fly ash may be used as substitute for a portion of the portland cement. The amount of substitution shall not exceed 15 percent by weight of cement. The fly ash replacement amount shall be determined through laboratory trial mix investigations using the specific materials proposed for the project. Use of Class F fly ash meeting the specific requirements of the agency is permitted. The use of Type C fly ash is not permitted. Participating agency practice concerning the use of fly ash in concrete in certain months of the year should be observed.

Fine Aggregate

Fine aggregate (passing the No. 8 sieve) shall consist of natural sand, manufactured sand, stone screenings, slag screenings, or a combination thereof, and meet the quality requirement of AASHTO M29. The fineness modulus of the fine aggregate shall not be less than 2.3 and shall not be greater than 3.1.

Coarse Aggregate

Coarse aggregate (retained on the No. 8 sieve) shall consist of crushed gravel or crushed stone particles meeting the requirements of AASHTO M80. It is recommended that the coarse aggregate conform to AASHTO 57 gradation as follows:

Table 55. Gradation table.

Sieve Size	Percent Passing
38 mm	100
25 mm	95-100
13 mm	25-60
No. 4	0-10
No. 8	0-5
No. 200	0-2

Coarse aggregate with a 25.4-mm maximum size aggregate may be used if such use represents the common practice of the participating agency.

The coarse aggregate shall conform to the following specific requirements:

Table 56. Course aggregate requirements.

Course Aggregate	Value
1. Abrasion loss, maximum %	50
2. Magnesium sulfate soundness, maximum %	12
3. Thin and elongated pieces, maximum %	15
4. Crushed particles, minimum %	55
5. Total of deleterious materials including chert, shale, and friable particles, maximum %	3

It is important that the coarse aggregate meet the highest standard of durability specified by the participating agency. Coarse aggregate must be obtained from a source approved by the agency and must be reasonably free from deleterious substances such as chert, gypsum, iron sulfide, amorphous silica, and hydrated iron oxide.

Coarse aggregate for use in concrete that will be subject to wetting, extended exposure to humid conditions, or contact with moist ground shall not contain any materials that are deleteriously reactive with alkalies in the cement in an amount sufficient to cause excessive expansion of mortar or concrete. However, if such materials are present in injurious amounts, the coarse aggregate may be used with a cement containing less than 0.6 percent alkalies calculated as sodium oxide equivalent or with the addition of a material that has been shown to prevent harmful expansion due to the alkali-aggregate reaction. The potential reactivity should be determined in accordance with the procedure given in AASHTO M80.

Other Items

Other items used in the production of concrete, such as water and admixtures, shall conform to the requirements normally specified by the agency for interstate concrete pavement construction. Use of microsilica (silica fume) as an additive is not permitted. Also, the use of additives to accelerate the strength gain of the concrete is not permitted for the SPS-2 experiment.

CONCRETE PAVEMENT REQUIREMENTS

Concrete pavement requirements for SPS-2 are summarized in the following sections.

The primary experiment SPS-2 addresses doweled JPCPs. The concrete pavement design for this experiment stipulates the following details:

- Slab thickness—203 and 279 mm.
- Joint spacing—4.57 -m uniform spacing.

- Lane width—3.66 and 4.27 m. A solid white line shall be painted to delineate the 3.66-m-wide travel portion of the widened lane.
- Joint load transfer—Doweled perpendicular transverse joints, with 32-mm dowel bars for the 203-mm-thick pavement and 38-mm dowel bars for the 279-mm-thick pavement. Dowels are to be epoxy coated, 457 mm long, spaced at 305 mm, and conforming to the requirements of AASHTO M254. Dowels are to be placed middepth using basket assemblies or dowel bar inserters with each bar aligned parallel to the longitudinal direction (with a tolerance of 1 mm per 50 mm of length) and located such that the bars will be centrally located (longitudinally) at the joint. Dowels shall be placed no closer than 152 mm from the longitudinal joints.
- Longitudinal joints—Between lanes should be sawcut, preferably using up to an 8-mm-wide blade, to a depth of D/3 (where D = slab thickness). The sealant reservoir may be formed later using a second sawcut to provide an 8-mm-wide by 25-mm-deep cut. The use of plastic inserts to form longitudinal joints is not permitted. The longitudinal joint between lanes will be tied using epoxy-coated deformed steel bars, No. 5 grade 40 steel, spaced at 762 mm center to center and 762 long. The tie bars shall be placed perpendicular to the longitudinal joint at a target depth of D/2.

CONCRETE PAVEMENT CONSTRUCTION OPERATIONS

The concrete pavement for the SPS-2 test sections shall be constructed in accordance with the practices and specifications that have proven successful for the participating highway agencies. It is strongly recommended that slip-form-paving procedures be used for concrete placement, and that the test lane and adjacent lane be slip-formed in one operation. The key items related to construction are outlined below.

Concrete Placement and Finishing

The test sections at each site incorporate several variables pertaining to the concrete slabs, including pavement thickness, concrete strength, and lane width. Therefore, it is recommended that special consideration be given to arranging the test sections at the site in a manner that will facilitate construction operations. Concrete placement for each test section should be done in a single continuous operation.

When dowel baskets are used at transverse joints, concrete placement using side-dump procedures will facilitate placement of dowel bars ahead of concrete placement. Therefore, this procedure shall be used for placement of concrete.

Use of slip-form equipment is recommended. The equipment shall spread, consolidate, screed, and float-finish the concrete so that a minimum of hand finishing will be necessary and a well-consolidated and homogeneous pavement is produced. The machine shall vibrate the concrete for the full width and depth of the concrete. Internal spud-type vibrators shall be used at a spacing of no more than 610 mm.

Jointing

Transverse contraction joints with dowel bars shall be provided at a spacing of 4.57 m and 9.14 m, respectively. These joints shall be sawed perpendicular to the longitudinal direction of the pavement. At these joints, dowel bars shall be provided using basket assemblies or dowel bar inserters. Dowels should be properly aligned and the dowel baskets, if used, should be securely anchored to the base layer and placed at pavement middepth. Dowels should be lightly coated with grease or other suitable lubricant over their entire length to prevent bonding of the dowel to the concrete.

All joints shall be sawed. For transverse contraction joints, an initial sawcut of D/3 is required, preferably made using up to an 8-mm-wide blade. A second sawcut should be made later, if necessary, to provide the required shape factor for the sealant material. Longitudinal joints between lanes should be sawed initially, preferably using up to an 8-mm-wide blade, to a depth of D/3. A second sawcut should be made later to provide for an 8-mm-wide by 25.4-mm-deep sealant reservoir.

The use of plastic inserts to form longitudinal joints is not permitted. The longitudinal joint will be tied using epoxy-coated deformed steel bars, No. 5 grade 40 steel, spaced at 762 mm center to center and 762 mm long. The tie bars shall be placed perpendicular to the longitudinal joint at a target depth of D/2.

If a concrete shoulder is used along the test sections, then the longitudinal joint between the outside shoulder and the travel lanes shall not be tied. The joint will be formed by placing the shoulder separately or by sawcutting to full depth if the concrete is placed at the same time as the travel lanes.

Timing of initial sawing of both transverse and longitudinal joints is critical. Therefore, sawing should begin as soon as the concrete is strong enough to both support the sawing equipment and prevent excessive raveling of the concrete surface. Longitudinal sawing shall be initiated at the same time as the transverse sawing. All sawing shall be completed within 24 hours of placement.

Curing

Only liquid curing compound is permitted for curing the concrete pavement. Curing compound shall be applied to the concrete surface within 15 minutes after the surface texturing operation and no later than 45 minutes after concrete placement. Participating agency practice shall be followed for surface texturing and in specification of the type of curing compound and application rate.

Joint Sealing

Joint sealing shall be accomplished using only silicone sealants. The sealant shall be either self-leveling or a tooled, no-slump material proven by the agency to work satisfactorily. Neither new or experimental sealants nor field poured liquid sealants shall be used for test sections. All pavement joints shall be sealed before opening to traffic.

Thickness Tolerance

It is necessary that every effort be made to obtain slab thickness as close to the target values of 203 and 279 mm as possible. Neither a deficiency nor an excessive thickness is desired. Final pavement thickness should be within 6 mm of the target values, as determined from cores and rod and level survey elevation changes. Elevation measurements are to be taken at intervals of 15.25 m or less within the test sections, both before and after concrete placement.

Pavement Smoothness

The surface of the finished pavement shall be tested with a California-type profilograph. Profiles shall be made in both wheel paths parallel to each edge of the pavement. The pavement shall have a prorated profile index of less than 158 mm per 1,000 m, as evaluated using California test 526. The contractor shall remove high pavement areas with vertical deviations greater than 11 mm in 8 m using diamond grinding devices or multiple-saw devices as approved by the agency. Only localized grinding is permitted; wholesale grinding of the finished pavement surface is not permitted.

Opening to Traffic

The test section pavements shall not be opened to traffic before 7 days after concrete placement, or before concrete flexural strength has reached 3.8 MPa. Joint sealing must be completed before opening to traffic. No construction traffic will be allowed on the test section until that time.

Repair of Defective Slabs

Pavement slab panels exhibiting cracking before the test sections are opened to traffic shall not be repaired. In cases where slab panels are damaged to the extent that structural repairs are necessary, the FHWA Pavement Performance Division shall be consulted prior to performing any repair activity.

Construction Operations

Construction operations shall be performed in compliance with the guidelines and specifications established by the participating agency for road and bridge construction. The agency’s high-quality construction practice should be enforced for the experiment. Adequate attention shall be given to details and control of the mix plant, hauling, placement, and consolidation operations to prevent construction practices that result in poor pavement performance. In addition, care should be taken to ensure that the test sections are constructed in a manner consistent with normal highway construction.

TRANSITIONS

The 183 m overall length of each test section includes 152.5 m for monitoring and 15.25 m before and after the section for materials sampling. The distance between these 183-m sections must be sufficient to allow changes in materials and thicknesses during construction. This distance is required to accommodate changes in concrete mix and slab thickness in a manner that will reduce the effect on the properties of the finished pavement. A minimum transition length of 36.6 m is recommended between the test sections to provide sufficient production in order to develop consistency after changes in materials, thicknesses, or lane widths.

SECTION STATIONING

The test site shall be surveyed to the extent that the limits of each test section location will be known to an accuracy of 0.305 m. The first test section occurring in the direction of traffic at a site will have the project station 0+00 at the beginning of the monitoring section. Subsequent test sections will have a test section station 0+00 at the beginning of each monitoring section. Site and individual test section beginning stations will be located 3.05 m before the first joint of the monitoring section. The ending stations will be 3.05 m beyond the last joint in the monitoring section.

DEVIATIONS FROM GUIDELINES

An agency that wants to participate in the SPS-2 experiment, but finds it necessary to deviate from some of the guidelines described in the report, should review these deviations with the LTPP Regional Office or LTPP Division. These authorities will assess the implications of these deviations on the study objectives. If the implications of the noncompliance appear minimal, the deviations will be accepted; otherwise, LTPP will suggest alternatives for consideration by the participating agency.

APPENDIX B. SUMMARY OF SPS-2 PROJECT CONSTRUCTION AND DEVIATION REPORTS

After the construction of each SPS-2 site, a comprehensive construction report was prepared to provide a general description of the project, summarize the construction activities, describe the paving materials, and note any key observations about the project. In many cases, a deviation report was also prepared to indicate significant events and deviations from the planned factorial designs. These construction and deviation reports provide valuable information about the SPS sites and are essential to a good understanding of these projects and their performance.

These reports are not readily available to the general public. Therefore, a summary of each of these reports is provided in this appendix. The report summary is presented in the following alphabetical order:

- Arizona (State code: 04).
- Arkansas (05).
- Colorado (08).
- Delaware (10).
- Iowa (19).
- Kansas (20).
- Michigan (26).
- Nevada (32).
- North Carolina (37).
- North Dakota (38).
- Ohio (39).
- Washington (53).
- Wisconsin (55).

ARIZONA SPS-2: I-10 EASTBOUND, MARICOPA COUNTY

Project Description

The Arizona SPS-2 project site is located in the eastbound lanes of I-10 in southwestern Arizona, approximately 56 km west of Phoenix. I-10 is a rural interstate that carried average annual daily traffic (AADT) of 15,900 (1992 statistics). The SPS-2 project was constructed as part of the rehabilitation of I-10. The typical pavement design consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 test sections were constructed on a portion of I-10 that is relatively straight and flat.

This test site is classified by LTPP to be in the dry no-freeze zone. The automated weather station has been functional since July 1994.

International Road Dynamics bending plate WIM equipment was installed in the fall of 1993. Calibration was completed on January 24, 1994.

Construction of the SPS-2 site started in June 1993 with the removal of the existing pavement. Construction of individual test sections was completed and opened to traffic on October 1, 1993.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 57. Arizona test section pavement designs.

Test Section No.	Typical Pavement Design
040260	216 mm dense-graded asphalt concrete on 102 mm DGAB
040261	216 mm dense-graded asphalt concrete on 102 mm DGAB
040262	203 mm undoweled jointed plain concrete (JPC) (3.8 MPa MR) on DGAB, 4.27-m lane
040263	203 mm undoweled JPC (3.8 MPa MR) on PATB, 4.27-m lane
040264	279 mm undoweled JPC (3.8 MPa MR) on PATB, 3.66-m lane
040265	279 mm undoweled JPC (3.8 MPa MR) on DGAB, 3.66-m lane
040266	317.5 mm doweled JPC (3.8 MPa MR) on bituminous-treated base, 4.27-m lane
040267	279 mm doweled JPC (3.8 MPa MR) on bituminous-treated base, 4.27-m lane
040268	203 mm doweled JPC (3.8 MPa MR) on bituminous-treated base, 4.27-m lane
040269	203 mm asphalt concrete on 102 mm DGAB

Materials and Construction Methods of Individual Pavement Layers

Subgrade

Overexcavation of the subgrade for test section construction was accomplished with a front-end loader, a sheepsfoot roller, scrapers, and a water truck. Compaction of the subgrade ranged from 95 to 97 percent of maximum density. Only 8 percent of the surveyed elevations ranged from the SHRP tolerance of 13 mm in 15.2 m.

Base Layers

Dense-Graded Aggregate Base

The DGAB consisted of a crushed river gravel with a 25.4-mm top size. Six percent of the fraction passed the No. 200 sieve. The DGAB was placed (belly dump trucks unloaded on grade) in 102- to 152-mm lifts and was compacted with a rubber tired roller.

A CMI ™ trimming machine was used to achieve proper layer thicknesses; however, layer thicknesses were generally in excess of design requirements.

Permeable Asphalt-Treated Base

The PATB contained an AC-20 binder at 2.5 percent of the total mix volume. This layer was placed (124 °C laydown temperature) at a 114-mm lift thickness with a Blaw-Knox™ PF500 track- mounted paver. Profile grade control was maintained with a wire guide line and/or paving skis. The PATB was compacted by two steel-wheeled vibratory steel-wheeled rollers. They achieved a rolldown of approximately 19 mm at a mat temperature under 77 ºC.

Lean Concrete Base

The mix design for this material included 105 kg of Type II cement, 23 kg (50 lb) Class F fly ash, 1,505 kg coarse aggregate, water reducers, and air entraining agents. The mix had a water-cement ratio of 0.88.

The LCB was paved with a Gomaco™ GP3000 slip-form paver and a Gomaco 9500 spreader, which had a laydown width of 6.7 m. Initial laydown of the LCB mix produced a dry mix, which resulted in considerable surface checking and dragging. These areas were hand patched with grout. Batch plant modifications remedied the mix problems (water demand), and construction of a smooth LCB mat was maintained for the remainder of the project. Finishing was initially provided with an automatic float and a burlap drag, but was later changed to a hand float and no texturing. A membrane-curing compound was applied to the LCB. Partial monolithic construction into the outside shoulder was achieved (single pass 0.9 m to 6.7 m right of the centerline). Three transverse cracks developed in section 040217 while 8, 4, and 17 cracks developed in sections 040218, 040219, and 040220, respectively.

Portland Cement Concrete

The 3.8 MPa and 6.2 MPa SHRP mixes had a maximum aggregate size of 25.4 mm, while supplemental State sections (undoweled) used a 3.8 MPa flexural strength AZDOT mix with a maximum aggregate size of 38.1 mm.

All mix design utilized on this project utilized Type II cement.

The 3.8 MPa mix had a water-cement ratio of 0.47 and a 14-day flexural strength of 3.9 MPa. The 6.2 MPa MPa mix had a water-cement ratio of 0.36 and a 14-day flexural strength of 5.8 MPa.

The PCC was paved with a Gomaco GP3000 slip-form paver and a Gomaco 9500 spreader/distributor. Cold joints were installed at the end of the day in several of the test sections. Test sections were paved in September. Corresponding concrete temperatures in some test sections were at least 27 ºC.

Key Observations

Construction Report and Data Evaluation

AASHTO No. 57 coarse aggregate was utilized as the backfill material in the pavement base drain.

A taper transition of the PATB into the DGAB could not be achieved. This resulted in the PATB being placed against the DGAB at the end of section 040263.

The width of the Class B geotextile supplied was too short to be wrapped around the PATB edge according to SHRP specifications. This could facilitate soil intrusion from the adjacent DGAB.

Transverse drains were installed perpendicular to the roadway centerline instead of in a herringbone fashion.

A 0.9-m-wide roll of filter fabric was placed with a 0.305 m width under the median edge of the PATB. The remaining width was wrapped around the median edge of the PATB to prevent soil infiltration.

Transverse cracking occurred in the LCB in sections 040217 through 040220 before placement of the PCC.

Longitudinal tie bars were uncoated and were only 508 mm in length. SHRP specifications require epoxy-coated rebar, 762 mm in length.

Paving was intermittently stopped in several of the test sections due to concrete availability, mix adjustments, and equipment failure.

PCC segregation and/or slump variations occurred in several of the sections.

The concrete temperature throughout construction generally ranged from 28 ºC to 31 ºC.

ARKANSAS SPS-2: I-30 WESTBOUND, HOT SPRINGS COUNTY

Project Description

The Arkansas SPS-2 project site is located in the westbound lanes of I-30 in west central Arkansas. The project is located just to the west of the I-70/I-30 interchange. I-30 is classified as a rural interstate with a 1993 AADT of 18,000 and 45 percent heavy trucks. The SPS-2 project was included in the reconstruction of I-30. Of the 12 test sections required for the SPS-2 project, 3 were located in original construction fill areas, 6 were located in original construction cut areas, and 3 sections were at-grade. The roadway typical for this project consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside shoulder width of 1.22 m.

This test site is in the wet no-freeze zone. The subgrade is classified as fine-grained. A weather monitoring station and WIM equipment were installed onsite.

Construction of the SPS-2 site began in November 1993 with the removal of the existing pavement. Construction of individual test sections was completed on October 1, 1995, and the project was opened to traffic on November 1, 1995.

Test Sections Constructed

All required core sections were constructed.

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The existing subgrade is classified as a fine-grained material.

Portland Cement Concrete:

The PCC was batched in 6.9 m^3 loads and delivered to the project, which was located approximately 5 km from the batch plant.

For the 6.2 MPa mix (sections 050204, 050208, and 050212), the air content ranged from 4.9 to 6.0 percent while the slump ranged from 25.4 to 38.1 mm.

Key Observations

Construction Report and Data Evaluation

Existing edge drains from the original I-30 roadway construction were removed during construction of the SPS-2 test sections.

On section 050208 (279 mm 6.2 MPa JPC on 152 mm LCB), the vibrators of the slip-form paver became entangled with the dowel basket assembly at station 2+50. This caused the augers of the paver to stop operating. The contractor removed and replaced the affected concrete and dowel basket assembly.

Longitudinal joints were not sealed until early 1997. By this time, pumping was evident through these joints.

Deviations

Construction Guideline Deviations

- No major deviations, although at station 2+50 on section 8, the slip-form paver's augers became entangled with the dowel assembly. The contractor removed the entire affected area (dowel assembly and concrete) and repaired the area.
- Also, longitudinal joints were not sealed. Pumping became evident, and joints were sealed in early 1997.

Data Collection and Materials Sampling and Testing Deviations

None.

COLORADO SPS-2: I-76 EASTBOUND, ADAMS COUNTY

Project Description

The Colorado SPS-2 project is located in the eastbound lanes of I-76 in central Colorado, approximately 32 km northeast of Denver. I-76 is a rural interstate with a 1988 AADT of 8,400 and 16 percent heavy trucks. The design KESALs for this project is 15,600 for a 20-year design life. Seven SPS-2 test sections were included in the phase 1 section of I-76, which was constructed on a new alignment (sections 080217, 080220, 080221, 080222, 080223, and 080224; station 155+90 to station 227+90). The remaining six test sections (sections 080213, 080214, 080215, 080216; 080218, and 080219) were located within the phase 2 section of I-76, which was being reconstructed (station 101+40 to station 155+60).

The 136th Street interchange bisects this SPS-2 site. Sections 080213 through 080216 and 080218 through 080219 are located south of the 136th Street exit. Sections 080221, 080222, and 080223 are located north of the 136th Street entrance ramp. Sections 080217, 080220, and 080224 are located between the 136th Street exit and entrance ramps. No appreciable difference in traffic loadings is expected due to the presence of this interchange.

This SPS-2 site is located in a dry-freeze climate. The subgrade for this project is coarse grained and predominantly consists of sand to clayey sand. The replicate SPS-2 project for this site is located in northern Nevada.

All test sections are on a tangent. The vertical grade averages +1.4 percent in the direction of traffic. Six test sections were located in a cut (sections 080217, 080218, 080219, 080220, 080223, and 080224), while all other sections were located in on fills. The roadway typical for this project consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m.

A weather monitoring station was installed on-site. Construction began on July 1, 1993, and was completed on November 1, 1993. Phase 1 work (new alignment) was completed first and opened to traffic on October 7, 1993. Phase 2 work (I-76 reconstruction) was started after phase 1 was open to traffic. Phase 2 was opened to traffic on January 5, 1994.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 58. Colorado test section pavement design.

Test Section No.	Typical Pavement Design
080259 (Control)	279 mm JPC (4.5 MPa), 3.66 m lane

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The existing subgrade varies from clayey sand to sandy clay, but is classified as coarse grained for this experiment. Profile grade construction in phase 1 was accomplished by constructing embankments in fill from subgrade removed from the mainline in cut areas. Subgrade compaction was accomplished through the weight of construction traffic, which consisted of scrapers and dozers. Compaction was monitored with nuclear density gauges. No moisture was added to the prepared subgrade since the soil exhibited an acceptable level of moisture due to a locally high water table.

The subgrade in section 080222 appeared to be poorly compacted but was not recompacted. The subgrade in section 080220 appeared to have high water content, and pumping was observed in the transition area between this section and section 080224. Two areas in the travel lane in section 080217 required undercutting. Subgrade was removed to a depth of 1.22 to 1.83 m and a width of 1.83 m and replaced with fine sand. Several areas within this test section pumped due to the weight of the end-dump trucks during paving of the LCB. Subgrade compaction with steel-wheeled rollers was performed immediately in front of the trucks, but this action did not solve the pumping problem. LCB paving proceeded with no additional subgrade stabilization being performed. All other test sections in the phase 1 area appeared to be well compacted.

During construction, a temporary access road crossed section 080221. Local traffic and construction traffic used this access road. This resulted in a varying compactive effort for the subgrade in this test section. Additionally, the subgrade was constructed at different times in this test section due to construction sequencing of the access road.

Embankment construction in the phase2 area consisted of removing the existing bituminous and concrete layers of the old roadway, crushing this material to 152-mm-diameter pieces, and recompacting this material along the mainline fill sections in the phase2 area. Approximately 0.61 m of fine sand cover material (obtained onsite) was placed on top of the crushed roadway material. The depth of crushed roadway material ranged from 0.91 to 1.22 m in section 080213 and 080216, to 1.83 m in section 080214, and to 3.05 m in section 080215. Sections 080218 and 080219 were cut with individual cuts ranging from 1.83 to 2.44 m in depth.

Dense-Graded Aggregate Base

The DGAB was a pit run material that conformed to SPS-2 and Colorado Class 5 specifications. Class 5 material has a 38.1-mm top size, 30 to 70 percent of the material passing the No. 4 sieve, and 3 to 15 percent of the material passing the No. 200 sieve. The DGAB was delivered in belly-dump trucks, which were permitted to drive on grade, bladed with a Caterpillar (CAT) 140G, compacted with a CAT CS563 steel-wheeled roller, and trimmed to the desired thickness.

Weather conditions during placement of the DGAB in the phase1 sections were cool and overcast, with light to heavy rain on several occasions. Phase 2 was constructed under better weather conditions, with variations from clear and sunny to cool and overcast with light rain.

Permeable Asphalt-Treated Base

The PATB contained an AC-10 binder at 2.5 percent with no antistrip agent. This layer was delivered in end-dump trucks from an offsite drum plant and placed with a Blaw-Knox track-mounted paver. Grade control was maintained with a stringline. The PATB was placed in 122- to 127-mm lifts at 82 to 93 ºC in 4.11- m widths (except for the last pass, if the required paved width was less than 4.11 m). The PATB was compacted by two passes from a 5-ton steel-wheeled roller. Excessive fines were noticed in sections 080221, 080222, and 080223. The PATB was removed and replaced in section 080221 due to excessive fines. An additional 50.8 mm of PATB were added (day after original paving) in the right shoulder area of section 080221 for 38.1 m when construction inspection personnel discovered that this section was too low.

Lean Concrete

The mix design for this material included 92 kg of Type I/II low-alkali cement, 720 kg crushed No. 57 stone, 698 kg (1550 lb) sand fine aggregate, 27 kg Class F fly ash, and 115 kg water. The water-cement ratio was 0.96. This mix design produced a 7-day compressive strength ranging between 3.8 and 5.2 MPa.

The LCB was hauled in end-dump trucks, which were permitted to back down the subgrade directly in front of the slip-form paver (CAT SF550). The LCB was finished with a wet burlap drag, hand trowels, and steel floats. The LCB was paved at laydown width of 11.58 m in phase 1 sections and 9.75 m in the phase 2 sections. Air temperatures during placement averaged 21 ºC during phase1 and 7.2 ºC during phase 2.

The subgrade was pumping during paving in sections 080217 and 080220. The finished surface appeared rough throughout the entire length of each of these Phase 1 test sections at 0.76 m (2.5 ft) right of the proposed centerline. The LCB mix changed from a 102-mm (4-in) slump to a 50.8-mm (2-in) slump near the end of section 080217 (the first LCB section to be paved). Several transverse cracks, segregated areas, and depressions were noticed in the LCB mat after paving and curing.

Both LCB sections constructed in phase 2 were placed during cool rainy conditions. Both sections had broken edges (shoulders added at a later date) and a muddy finished surface with water stains.

Portland Cement Concrete

All mix designs on this project utilized a Type I/Type II low-alkali cement. The 3.8 MPa mix included Class F fly ash. The 3.8 MPa mix had average flexural strengths of 3.6 MPa at 7 days and 3.9 MPa at 14 days. The 6.2 MPa mix had average flexural strengths of 5.8 MPa at 7 days and 6.2 MPa at 14 days. The water-cement ratio averaged 0.47 for the 3.8 MPa mix and 0.29 for the 6.2 MPa mix.

Side-dumping concrete into a track- mounted Gomaco PS60 spreader completed paving. This machine augered the concrete across the lane, which was consolidated with a track-mounted CAT SF550 slip-form paver. A power screed, wet burlap drag, and a mechanical float were used for intermediate finishing, while hand floats and an astroturf drag were used for final finishing. A transverse tining machine completed the texturing.

Joint sawing was performed within 8 hours of concrete placement. Joints were sawn and sealed according to SPS-2 specifications. The shoulder joint was sawn full-depth along all sections.

Paving of phase1 sections was completed on September 3, 1993 for sections 080221 and 080223 (3.8 MPa mix) and section 080222 (6.2 MPa mix). The air temperature ranged from 13 to 20 ºC during section 080223 paving, 21 to 26 ºC for section 080222 paving, and 24 to 27 ºC for section 080221 paving. At station 186+00 in section 080221, the paving train pulled out a dowel basket. This assembly was not replaced.

Sections 080224, 080220 (6.2 MPa mix), 080217 (3.8 MPa mix), and 080259 (CO DOT 4.5 MPa mix) were paved from September 7 to 9, 1993. Paving in section 080224 was discontinued due to heavy rain. The edge drains and geotextile were damaged during paving but were not repaired. A construction joint was formed in this test section, and paving was resumed on September 8. The belt on the feeder to the spreader broke after completion of this test section. Paving operations were then discontinued for the day. The remaining sections were paved on September 9 and 10, 1993. The air temperature ranged from 13 to 20 ºC during section 080220 paving, 22 to 27 ºC for section 080217 paving, and 13 to 27 ºC for section 080259 paving.

Paving of the phase 2 sections was completed from October 11 through October 22, 1993. Sections 080216 (6.2 MPa mix) and 080213 (3.8 MPa mix) were paved on October 11, 1993. The air temperature ranged from 7.2 to 21 ºC during the section 080216 paving and 21 to 22 ºC for the section 080213 paving. Section 080214 (6.2 MPa mix) was paved at air temperatures of 10 to 18 ºC. Sections 080215, 080218, and 080219 were paved at air temperatures of 18 to 22 ºC, 4 to 13 ºC, and 3.3 to 9 ºC, respectively. Sections 080213 and 080215 (3.8 MPa mix) had average slumps of only 25.4 mm.

Key Observations

Construction Report and Data Evaluation

Subgrade pumping occurred on several phase 1 sections due to rainy weather and a locally high water table. Pumping did not occur on the phase 2 sections. The embankment in these test sections consisted of stable fill material including pulverized concrete and asphalt capped by a fine sand layer.

Several of the PATB sections contained too many fines in the mix. This resulted in removal and replacement of the mat in section 080221.

Due to its high plasticity, the 6.2 MPa concrete mix was harder to work with than the 3.8 MPa mix.

While paving section 080218, equipment problems and concrete delivery problems (muddy haul roads) caused several work stoppages. The dowel bars and basket assembly were torn up at station 141+50 but not replaced.

No major problems occurred during construction of the DGAB and LCB layers.

DELAWARE SPS-2: U.S. 113 SOUTHBOUND, ELLENDALE

Project Description

The Delaware SPS-2 project site is located in the southbound lanes of U.S. 113 in central Delaware, between Milford and Georgetown. U.S. 113 is a rural principal arterial with a 1989 AADT of 10,708 and 10 percent heavy trucks. The design KESALs were calculated to be 3,048,600 for the 15-year design life of the pavement. The SPS-2 project was included in the addition of two southbound lanes to an initial two-lane roadway. The two new southbound lanes were separated by a 7.92- to 12.8-m-wide median from the existing northbound lanes. Route S-625 (State Route) and another access road bisect this SPS-2 site. The traffic from these routes is expected to have little impact on the SPS-2 site.

The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 project site lies within the Atlantic Coastal Plain. The subgrade consists of sand and silty sand. The topography is flat to gently rolling, and bedrock does not exist near the pavement surface. Test sections were constructed on shallow cuts or fills. The cut sections ranged up to 1.52 m in depth. Several wetland areas exist adjacent to the mainline pavement, where the water table is at or near the surface for an extended time period.

This test site is in the wet-freeze zone. The subgrade is classified as coarse grained. A weather monitoring station, WIM, and AVC equipment were installed onsite.

This project was completed and opened to traffic on May 1, 1996.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 59. Delaware test section pavement designs.

Test Section No.	Typical Pavement Design
100259	254 mm JPC (20.7 MPa) on 203 mm DGAB; 3.66-m lane, steel dowels
100260	254 mm JPC (20.7 MPa) on 203 mm DGAB; 3.66-m lane; plastic dowels

Materials and Construction Methods of Individual Pavement Layers

Subgrade

Delaware DOT standards require the top 305 mm of subgrade to be Type A borrow material. This material consists of 95 to 100 percent of the material passing a 76.2-mm sieve and a maximum of 35 percent of the material passing the No. 200 sieve. Existing subgrade material in cut sections that met Type A borrow requirements was left in place. All other areas received at least 305 mm of Type A borrow.

Subgrade and embankment construction started in the spring of 1994 under wet conditions. Wet conditions persisted throughout the summer with little completion of earthwork. Most of the earthwork was completed during the fall of 1994 under dry conditions. Earthwork resumed in April 1995 and was completed by the end of the month.

Dense- Graded Aggregate Base

The DGAB consisted of an igneous diorite (trap rock) crushed stone with approximately 90 percent passing the 25.4- screen. The DGAB was placed in 102-mm lifts with a spreader box and compacted with a Dynapac ™ 105 CA25 single drum vibratory compactor.

Permeable Asphalt-Treated Base

This layer was placed with a material transfer vehicle and a paver. The PATB was rolled with a 10-ton steel-wheeled static roller when the mat temperature was between 66 and 77 ºC. Construction traffic was allowed on the PATB after placement and cooling. Damage to the mat did occur from turning movements of the construction traffic.

Lean Concrete Base

The mix design for the LCB included 144 kg of Type I low-alkali cement, 718 kg fine aggregate, 720 kg coarse aggregate, and 143 liters water. The mix had a water-cement ratio of 0.96. Fourteen-day compressive strengths ranged from 7 to 8 MPa.

The LCB was placed with a Blaw-Knox MC30 mobile conveyor and an ABG Titan™ 511 paver. A roller recompacted the subgrade before LCB placement to avoid rutting of the sandy subgrade. The LCB was laid at a 8.84-m-width for the 3.66-m-wide lane sections and at a 9.45-m-width for the 4.27-m- wide lane sections. A grooving tool was used to form the longitudinal joint in sections 100205, 100206, and 100207, while the longitudinal joint was sawn in section 100208. For sections 100205 and 100208, the longitudinal joint was offset 305 mm into the passing lane. This joint was offset 457 mm into the driving lane on sections 100206 and 100207. High spots in the LCB were milled before placement of the PCC.

Portland Cement Concrete

The SPS-2 3.8 MPa concrete mix utilized on this project contained a Type I low-alkali cement while the 6.2 MPa mix utilized 65 percent Type I cement and 35 percent New Cem™ (slag cement).

The PCC was delivered to the SPS-2 site in side-dump trucks. A Maxon ™ spreader and a Gomaco GP 3500 slip-form paver were used in the paving operations. Concrete paving commenced on June 15, 1999, with the paving of the Delaware control section (section 100260) with a DelDOT Type B mix. Sections 100205, 100201, and 100209 were paved on June 16, 1999, with the SHRP 3.8 MPa mix.

Shrinkage cracks developed in the PCC in sections 100205, 100201, and 100209 placed on June 16, 1995. The cracked PCC in these test sections was removed and repaved with a Delaware DOT Type B mix on October 12, 1995. The DelDOT Type B mix is a 20.7 MPa (compressive strength) mix and has a corresponding flexural strength of approximately 4.5 MPa. Several transverse cracks again developed in these test sections that had been repaved with the DelDOT Type B mix.

Subsequent 3.8 MPa test sections that had not been paved (sections 100211, 100203, and 10207; 4.27-m-wide lane sections) were paved with the DelDOT Type B mix on June 28, 1995. Sections 100212 (PATB), 100208 (LCB), and 100204 (DGAB) are all 6.2 MPa test sections and were paved with a 7.5-bag mix without NewCem™. Section 100212 was paved on July 17, 1995 (6:45 a.m. start). By 9:00 a.m., paving was stopped because the concrete temperature had reached 32 ºC. Paving was restarted at 8:15 p.m. and was completed by 9:45 p.m. Paving continued through section 100208 and reached the midpoint of section 100208 when heavy rain was encountered. The concrete was covered with polyethylene. Paving resumed at 1:15 a.m. on July 18 and reached the end of section 100208 by 4:00 a.m. Rain resumed again at approximately 6:30 a.m. during concrete placement in section 100204. The concrete temperature was measured at 29 to 31 ºC in the transition area between sections 100208 and 100204 while the air temperature was measured at 27 ºC.

The 6.2 MPa SPS-2 mix was placed on June 29, 1999, in sections 100206, 100202, and 100210. Sections 10206 and 100202 developed excessive shrinkage cracks. The concrete was removed and replaced in sections 100202 and 100206 with another 6.2 MPa mix on November 21, 1995, while only patching of the transverse cracks was performed in section 100210.

Several transverse cracks were noticed in these test sections the following day. Longitudinal cracks were also noticed in section 100206, which had an underlying LCB.

DelDOT personnel believed the cracking was due to late sawing. The contractor believed that the joints were sawn as soon as the surface could not be marred. DelDOT personnel decided to remove and replace the concrete in sections 100201, 100205, and 100209 (3.8 MPa mix sections) and sections 100202 and 100206 (6.2 MPa mix sections) due to excessive shrinkage cracking.

Sections 100212, 100208, and 100204 were placed on July 17 and 18, 1995, with the modified 6.2 MPa mix that did not contain NewCem. Paving had to be shut down by 8:53 a.m. on July 17 due to high concrete temperatures. Paving was resumed at 8:15 p.m. Paving on July 18, 1999, had to be stopped twice due to heavy rain.

DelDOT supplemental section 100259 was placed on July 20, 1995. This test section contained the DelDOT 4.5 MPa Type B mix with NewCem.

Sections 100201, 100205, and 100209 were repaved on October 12, 1995, with the DelDOT 4.5 MPa Type B mix with NewCem. The interval between placement and sawing was 12 hours in section 100205. Within 2 weeks, transverse cracks again developed in section 100205. The cracks were predominantly in the driving lane close to the transverse contraction joints. Additional cracks developed during the ensuing winter.

All existing cracks were patched during April 1996. Seventeen cracks were patched in section 100205.

Sections 100206 and 100202 were repaved on November 21, 1995, with the SHRP 6.2 MPa mix containing NewCem.

Key Observations

LTPP SPS Construction Reports

When the paver was stopped during paving of the LCB layer, depressions were formed in the subgrade. Transverse shrinkage cracks also developed in the LCB layer prior to PCC paving, and some of these shrinkage cracks developed in the depression areas.

During removal of the cracked PCC (DelDOT mix), construction personnel noticed that some of the shrinkage cracks in the LCB in section 100205 had reflected through the PCC. Some areas of the LCB had bonded to the PCC; however, the underside of most of the slabs was smooth and clean, which indicates an unbonded condition. The second application of a curing compound immediately before PCC paving appears to have been effective in debonding the PCC, except where surface depressions and irregularities existed in the underlying LCB.

A longitudinal crack had developed by October 13, 1995, in section 100207 at 457 mm from the centerline and parallel to the centerline. This crack was near the underlying construction joint in the LCB. This crack was cored on October 26, 1995, and was found not to extend for the full depth of the concrete pavement. This crack may be attributable to late sawing of the longitudinal joint. This section was paved on June 28, but longitudinal joint sawing was not performed until July 3.

Prior to removal of the concrete in sections 100205, 100206, and 100207, coring of transverse and longitudinal shrinkage cracks was performed. These cracks were found to extend either entirely or partially through the PCC but not into the underlying LCB. No bond was found to occur between the PCC and the underlying LCB.

Removal of some of the DGAB occurred in sections 100201 and 100202 with removal of the cracked JPC. Additional DGAB was added before JPC repaving in the test sections to create a uniform mat. The DGAB was then reshaped and recompacted.

After full-depth patching was completed, several additional cracks developed during the winter of 1995-1996 in section 100205 (LCB). Two additional cracks developed in section 100201 (DGAB), but no additional cracking developed in section 100209 (PATB).

Patching of these cracks was performed from April 18 to 19, 1996. At this time, 17 fine transverse cracks were noticed in various test sections. These cracks occurred at the edge of the pavement and only extended a few feet into the slab panel.

No. 57 stone was used as the edge drain backfill instead of PATB.

Transverse joint sealant reservoirs were sawn to a 19-mm width and a 38.1-mm depth, while the longitudinal joints were sawn to a width of 6.4 mm and a depth of 13 mm. The transverse joints in all test sections except sections 100206, 100202, and 100210 were sealed with neoprene seals. The transverse joints in the remaining sections and all longitudinal joints were sealed with hot- pour rubberized asphalt material.

Deviations

Site Location Guidelines Deviations

Eight of the 12 test sections contained partial shallow cuts, but the cut subgrades had to meet Type A borrow specifications. Those cut subgrades that did not meet the Type A specifications were excavated to receive 305 mm of Type A borrow (with prior approval) (sections 100201, 100203, 100204, 100205, 100207, 100208, 100209, and 100211).

Data Collection and Material Sampling and Testing Deviation Comments

A transverse construction joint was placed within section 100212.

The longitudinal joint was sawn 5 days after the concrete placement in sections 100211, 100203, and 100207.

For all sections:

- Bases did not extend the full width of the shoulder (with prior approval).
- Neoprene was used in the transverse joints (hot poured in three sections where the joints were rough) and hot-poured rubberized asphalt in the longitudinal joint.
- No joint sealant was used between the mainline concrete pavement and the asphalt shoulder.
- Joint sealing was done in 1996 and in the second construction season.
- The road was opened to construction traffic before joint sealing.
- Tensile strength testing equipment was not obtained until after July 25, 1995, so cylinders and cores requiring this test before this time were missed.
- Tensile strength testing equipment was not obtained until after July 25, 1995, so cylinders and cores requiring this test prior to this time were missed.
- 365-day cores will not be obtained until the northbound lanes have been rehabilitated and opened to traffic.
- Samples have been sent to the laboratories, but the materials testing data available to date is not complete.

For sections 100212, 100210, 100211, and 100209:

- Edge drains were not located at the outside edges of the shoulder.
- Edge drain outlets were spaced at distances greater than 76 m.

Construction Guidelines Deviations

3.8 MPa flexural strength concrete was not used on sections 100207, 100203, and 100211. 20.7 MPa comp was used instead.

3.8 MPa flexural strength concrete was used on sections 100201, 100205, and 100209. This concrete was removed and replaced with 4.5 MPa flexural strength concrete.

Sections 100202 and 100206 were placed with 6.2 MPa flex 6.5-bag mix. This concrete was later removed and replaced with 6.2 MPa flex 7.5-bag mix.

Profile index for all sections was greater than 158 mm/km. Note that section 100205 is scheduled for diamond grinding.

IOWA SPS-2: U.S. 65 NORTHBOUND, POLK COUNTY

Project Description

The Iowa SPS-2 project site is located in the northbound lanes of U.S. 65 in central Iowa, northeast of Des Moines. U.S. 65 is an urban/principal arterial with a 1994 AADT of 17,400 and 16 percent trucks. The calculated KESALs were 9,870 for the project over the 30-year design life of the pavement. The SPS-2 project was included in the relocation of U.S. 65 in both the northbound and southbound lanes.

The roadway typically consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 test sections were constructed on a portion of U.S. 65 that included both tangent and superelevated sections. All sections were constructed on a tangent except sections 190215 and 190216. These sections were constructed on the high side of a horizontal curve with a superelevation rate of 2.5 percent. Vertical grades throughout the project area range from -2.6 percent to +2.2 percent. Test sections 1902215 through 190220 were constructed on fill sections ranging from near 0 to 11.58 m in height. Sections 190221 through 190224 were constructed on cut sections ranging from 0.91 to 7.01 m.

This test site is in the wet-freeze zone. The subgrade is fine grained.

An onsite weather monitoring station was not installed until 1996.

WIM and AVC equipment was installed in June 1995 on U.S. 65 approximately 1.61 km north of the junction of U.S. 65 with IA-163.

Reconstruction was completed in 1994 during a period of relatively wet weather conditions. The project was opened to traffic on December 1, 1994.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included a control section according to the following specifications:

Table 60. Iowa test section pavement design.

Test Section No.	Typical Pavement Design
190259 (control)	279 mm (11 in) JPC, 4.27- m (14 ft) wide lane

Materials and Construction Methods of Individual Pavement Layers

Subgrade

Subgrade preparation was completed with a motor grader, disk, and a 10-ton sheepsfoot compactor. A CMI™ SP30AST full-width soil profiler completed final grading.

Base Layers

Dense-Graded Aggregate Base

The DGAB was compacted with vibratory and pneumatic-tired rollers. A profiler was used to trim the base to the required thickness.

Permeable Asphalt-Treated Base

This layer was placed with a Cedar Rapids™ CRS61 paver in a 3.96-m pass. The PATB was laid down at approximately 138 ºC and compacted with two passes of a 10-ton tandem steel-wheeled roller. Monolithic construction transversely and into the shoulder was not obtained.

Lean Concrete Base

The LCB mix design included 669 kg of coarse aggregate limestone, 821 kg sand, 72 kg Type II cement, 22 kg Type C fly ash, and 120 kg water. Admixtures included a water reducer and an air-entraining agent. This mix had a water-cement ratio of 1.28, an air content of 6.3 percent, and a 38.1-mm slump. Eight-day compressive strength averaged 4.3 MPa.

The LCB was paved at a 7.92-m width with a CMI SF 450 slip-form paver. Finishing work included machine troweling and hand troweling. The LCB had a water-cement ratio of 1.28 and an average 8-day compressive strength of 4.3 MPa.

Portland Cement Concrete

All mix designs on this project utilized Type II cement. The 3.8 MPa mix design produced an average 7-day flexural strength of 3.9 MPa and an average 14-day flexural strength of 4.2 MPa. The 6.2 MPa mix design produced an average 7-day flexural strength of 5.9 MPa and an average 14-day flexural strength of 6.0 MPa.

Key Observations

LTPP SPS Construction and Deviation Reports

Underground structures were located in 6 of the 13 test sections (190213, 190214, 190215, 190217, 190219, and 190221). These ranged from a 0.61-m diameter concrete pipe at 2.44-m below profile grade to a 2.44-m by 3.05-m concrete pipe at 12.19-m below profile grade.

The contractor removed at least 0.3 m of geotextile from the longitudinal edge drains due to the low permeability of the geotextile.

The boundaries of section 190222 were relocated after construction because dowel bars with the wrong diameter were placed in the initial boundaries of this test section.

Four sections (190215, 190216, 190212, and 190223) had concrete thicknesses in excess of SPS-2 tolerances. These thicknesses ranged from 8 to 23 mm above the desired thickness (203 or 279 mm as applicable).

Deviations

Site Location Guidelines Deviations

During the placement of the PCC pavement in test section 190222, incorrect dowel baskets were placed. This area was removed, and the test section location was shifted to avoid the replaced pavement area. This will shift the location of bulk sampling, nuclear density testing, and coring locations. Some tests will now be located outside and within the test section.

Because of misinterpretation of guidelines, the test section numbers were revised. The correct numbers should be from 13 through 24. This revision was done after most of the sampling, testing, and data collection had been completed.

Construction Guidelines Deviations

The thicknesses of the following test sections deviated from the construction guidelines more than 0.012 m or 12 mm.

Table 61. Iowa test section thickness variations.

Test Section No.	Base Thickness (mm)	PCC Pavement (mm)
190219	+ 3.0 mm (0.12 in)	NA
190220	+ 21 mm (0.82 in)	NA
190215	NA	+ 11 mm
190216	**NA**	+ 8.1 mm
190213	NA	+ 5.6 mm
190214	NA	+ 5.6 mm
190211	NA	+ 2.3 mm
190223	NA	+ 11 mm

Other Deviations

Section 190222 was removed after all sampling and testing and data collection was completed.

KANSAS SPS-2: I-70 WESTBOUND, DICKINSON COUNTY

Project Description

The Kansas SPS-2 project site is located in the westbound lanes of I-70 in central Kansas, east of Abilene. I-70 is a rural interstate with an AADT of 13,750 with 21.4 percent trucks. The yearly ESALs in the design lane are estimated at 1,300,678. The 20-year design ESALs is estimated at 26,013,550. The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05-m, and an inside shoulder width of 1.22-m. The SPS-2 project was included in the reconstruction of I-70. The existing pavement was concrete. The SPS-2 test sections were constructed on a tangent section of I-70 with vertical grades ranging from -2.48 percent to +2.11 percent. All test sections were constructed on fills.

This test site is in the dry-freeze zone. An onsite weather monitoring station had not been installed before completion of the project; however, installation was scheduled to occur by 1994. A Toledo Model 9430 high-speed WIM system was installed onsite.

Construction of this SPS-2 project was completed on July 1, 1992. The project was opened to traffic on August 1, 1992.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections included a control section (I-70 reconstruction typical) as follows:

Table 62. Kansas test section pavement design.

Test Section No.	Typical Pavement Design
200259 (control)	305 mm doweled JPC (4.1 MPa mix) on 152 mm stabilized subbase on 152 mm modified fly ash subgrade, 3.66-m lanes

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The subgrade soil is classified as silty clay. Since the SPS-2 project involved the reconstruction of an existing highway, the top subgrade layer was reworked and recompacted after incorporation of the existing granular subbase and shoulder material. Construction of the SPS-2 project occurred during an abnormally rainy period. KDOT added a Type C fly ash to the underlying subgrade to stabilize and dry this layer.

Base Layers

Permeable Asphalt-Treated Base

The contractor placed this base too thick, but subsequently trimmed the base to the required thickness.

Lean Concrete Base

This mix included 70 percent natural sand and 30 percent crushed limestone. The mix contained 1,256 kg of Type II cement, 976 kg fine aggregate, 418 kg coarse aggregate, and 126 kg water.

Portland Cement Concrete

All mix designs utilized on this project contained 70 percent natural sand and 30 percent crushed stone. Crushed limestone was used in the 3.8 MPa concrete, but calcite-cemented sandstone was utilized in the 6.2 MPa concrete mix in part to help increase the flexural strength. Type II cement was used for each mix design.

The 14-day flexural strength averaged 4.2 MPa (0.34 MPa standard deviation) for the 3.8 MPa mix and 5.8 MPa (0.34 MPa standard deviation) for the 6.2 MPa mix.

Key Observations

Deviation and Construction Reports

The project construction report indicates that the PATB was difficult to place. The contractor placed this material too thick in several of the test sections. The excess was removed with a trimmer. During initial construction operations, the PATB deformed when compacted. This problem was resolved as the contractor gained experience with this material.

The thickness of the PCC did not meet SPS-2 tolerances for the following test sections:

Table 63. Kansas test section PCC thicknesses.

Test Section No.	Design Thickness, mm	Actual Thickness, mm
200209	203	216
200210	203	211
200211	279	254
200212	279	231
200207	279	257
200204	279	290

Underground structures were present in sections 200204, 200208, 200209, 200210, 200211, and 200212. Median drains were present in several test sections; however, these drains were at least 1.52 m below the pavement surface.

Deviations

Data Collection and Materials Sampling and Testing Deviations

- Weather station was not installed until 1996 (4 years after construction was complete).
- The DOT staff experienced many problems with the sampling and testing requirements.
    - An insufficient number of cores was specified in the sampling and testing plan.
    - Field cores of the PATB could not be collected. Therefore, it was impossible to conduct tests on samples CA 01, 02, 03, 05, 47, 48, 51, and 54.
- Traffic monitoring data were only submitted for 1993 (78-day period).
- The first distress survey was not performed until April 1993.

Site Location Guideline Deviations

- Vertical curves (-2.48 to +2.11 percent grade) exist within the limits of the test sections.
- Several underground structures exit within the limits of the test sections.
  - Many of the sections contain 457-mm median drains. These drains are located >1.5 m below the surface of the pavement.
  - Test sections 200204, 200208, 200209, and 200211 have box culverts located within their limits.
  - Test section 200210 contains a transverse drain for the PATB.
  - Test section 200211 contains a median drain ±1.2 m below the surface of the pavement.

Construction Guideline Deviations

- Several sections have concrete pavement thicknesses that exceed the allowable tolerance of ±6.4 mm (200209 = +13 mm; 200210 = 7.6 mm; 200211 = -25.4 mm; 200212 = -48 mm; and 200204 = +10 mm).
- Construction was delayed due to an extremely wet and rainy season.
- The contractor experienced many problems while trying to place the PATB. Trimming was often required to obtain the desired thickness.
- Type C fly ash was used to help dry and stabilize the subgrade.

Other Deviations

- Test sections 200201 and 200204 received rehabilitation in 1995.
  - Test section 200201 required one full-depth patch and two partial-depth patches.
  - Test section 200204 required two partial-depth patches.

MICHIGAN SPS-2: U.S. 23 NORTHBOUND AND SOUTHBOUND, MONROE COUNTY

Project Description

The Michigan SPS-2 project site is located in the northbound and southbound lanes of U.S. 23 in southeastern Michigan, approximately 16 km west of Toledo. U.S. 23 is a rural principal arterial with a 1989 AADT of 35,000 and 22 percent heavy trucks. Twenty-six million ESALs were calculated for the design lane over the 20-year design life of the pavement. The SPS-2 project was included in the reconstruction of 9.7 km (6.021 mi) of U.S. 23 in both the northbound and southbound lanes. Consear Road, a low-volume county road, bisects this SPS-2 site. Traffic counts taken in the northbound lanes reveal that traffic south of this interchange is only 7 percent higher than traffic north of this interchange (AVC data only).

The roadway typically consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 test sections were constructed on a portion of U.S. 23 that is relatively straight and flat. Vertical grades throughout the project area range from 0.00 percent to +0.55 percent. Most of the test sections were constructed on shallow fills; however, sections 260214, 260218, and 260219 were constructed on deeper fills. All sections except 260214, 260218, and 260219 were constructed in tangent sections. Section 260214 was constructed in a superelevation transition area, while sections 260218 and 260219 were constructed on a full superelevation of 0.037 m/m.

This test site is in the wet-freeze zone. The subgrade is fine grained. An onsite weather monitoring station was not installed until the winter of 1995-1996.

WIM and AVC equipment were installed on U.S. 23 south of Consear Road. Only AVC equipment was installed north of Consear Road.

Reconstruction of U.S. 23 began in April 1993 with removal of the existing pavement layers. Construction of the subgrade progressed from mid-May through mid-June, and placement and compaction of the embankment was completed by mid-June. Undercuts were completed in sections 260216, 260022, and 260223 due to unstable soil conditions revealed during proofrolling. These undercuts were 11 m wide and 0.3 m deep, but only extended for a partial length of each test section. The undercuts were backfilled with embankment borrow clay. Base and subbase layer construction began by mid-June and was completed by mid-September 1993. Concrete paving commenced on September 13, 1993 (excluding control section) and was completed on September 21, 1993. The project was opened to traffic in November 1993.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 64. Michigan test section pavement design.

Test Section No.	Typical Pavement Design
260259 (Control)	267 mm JPC on 102 mm open- graded, asphalt- stabilized base on 76 mm aggregate base with tied concrete shoulders, neoprene seals in transverse joints, and hot-poured rubberized asphalt in longitudinal joints

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The existing sand subbase was removed before the embankment clay. It was then graded and rolled with a 14-ton single-drum static roller and proofrolled with a 25-ton pneumatic tired device on a rigid frame. Embankment clay was then spread on grade in 229-mm lifts and compacted with a 13-ton sheepsfoot roller. The embankment was fine graded with a CMI autograder and proofrolled again.

Base Layers

Dense-Graded Aggregate

The DGAB was end dumped on grade, leveled by a bulldozer and grader, and compacted with a 14-ton single-drum vibratory roller. A CMI autograder trimmed and fine graded the DGAB.

Permeable Asphalt-Treated Base

This layer was placed with a Barber Greene (BG)™ 225B paver. The laydown width for the paver varied between 2.74 and 4.88 m. The PATB was compacted with six passes of an 8-ton static roller. Monolithic construction transversely and into the shoulder was not obtained.

Lean Concrete Base

The mix design for the LCB included 722 kg coarse aggregate, 617 kg fine aggregate, 74 kg cement, and 128 kg water. The mix incorporated a water reducer and an air-entraining agent.

The LCB was delivered in end-dump trucks from adjacent haul roads and conveyed to a CMI SF 250 slip-form paver through a CMI MTP00 transport vehicle. Machine troweling and hand troweling were performed. A membrane-curing compound was applied to the LCB after placement and was reapplied before placement of the PCC. No data on placement widths of the LCB was provided.

Transverse shrinkage cracks developed in each LCB test section shortly after paving. Shrinkage crack development may have been augmented by very hot and dry conditions during LCB paving, as well as by dry embankment clay, as evidenced by the desiccation cracks.

Portland Cement Concrete

All mix designs on this project utilized Type I cement.

The PCC was delivered to the SPS-2 site in end-dump trucks on haul roads adjacent to the mainline, placed on the belts of a CMI belt placer, and the first layer thickness was spread equal to 75 percent of the final depth. The final layer was again delivered via adjacent haul roads and deposited on belts of a CMI 450 slip-form paver. Dowel baskets were placed at the transverse joints. Finishing consisted of machine and hand troweling. The surface was burlap dragged and then tine textured. A membrane-curing compound was applied to the surface. Transverse contraction joints were sawcut as soon as the surface could not be marred.

Cracking underneath the sawcuts had not occurred under all joints for several weeks. This was attributed to the time of year the pavement was constructed. Little change in temperature occurred from day to night. The transverse joints were sealed with low-modulus silicone.

Key Observations

Construction and Deviation Reports

The project construction and deviation reports indicate that some deviations occurred due to site conditions (unequal traffic volumes, culvert within test section, unequal fill height geometry).

For sections 260213 through 260220, the moisture content of the subgrade was not maintained within 85 to 120 percent of the optimum moisture content. Moderate-to-severe desiccation cracks (up to 50.8 mm in width and 254 mm in depth) developed in the subgrade compacted dry of optimum since the completed embankment was exposed to hot and dry weather conditions before construction of the overlying base or subbase layers. This occurred on all sections except those constructed with PATB. Cracking did not occur on PATB sections because the DGAB was placed soon after the completion of the embankment.

Michigan DOT required the contractor to scarify the desiccated subgrade sections and recompact severely desiccated subgrade to SPS-2 requirements.

The following are some specific observations:

- The DGAB in section 260221 segregated, but was reworked in the worst areas to minimize segregation.
- The DGAB was not kept uniformly wet before paving in sections 260213 through 260216.
- Rutting occurred in the PATB in the outside shoulder area due to construction traffic. This also caused deformation of the edge drains.
- Rutting occurred in the PATB in the outside shoulder area due to construction traffic. This also caused deformation of the edge drains.
- Cracking of the LCB occurred in the outside shoulder area of sections 260217 and 260220 due to construction traffic.

- Bonding of the LCB to the PCC was noticed in two of the sampling cores. This was not evident in other LCB/PCC cores.
- The LCB in sections 260218 through 260220 had a slump less than 25.4 mm.
- The LCB was milled between inside and outside lane placements in section 260218. The surface grooves were filled with grout and the spray cure was reapplied.
- The PCC in sections 260214, 260219, and 260220 had an air content less than 5 percent.
- The PCC did not meet SHRP requirements for 3.8 MPa and 6.2 MPa flexural strengths at 14 days; however, flexural testing indicated that the 3.8 MPa mix and the 6.2 MPa mix had similar flexural strengths at 365 days after placement. The flexural strengths at one 1 year averaged 6.1 MPa for the 3.8 MPa design mix and 6.6 MPa for the 6.2 MPa design mixes. The 14-day strength was too high in sections 260213, 260214, 260215, and 260219, and it was too low in sections 260220 and 260224.
- Several pavement layers were out of specifications with respect to thickness tolerances.
- Elevation measurements were not taken on all embankment layers.

Distress Surveys

Longitudinal joint seal damage at the lane-shoulder joint occurred in several test sections by 1994. The entire length of this joint in all test sections (except control section) failed by 1995. No damage was evident in the control section, which was constructed with tied concrete shoulders.

Pumping at the longitudinal joint and transverse joints was observed in most of the sections constructed with a DGAB and all of the sections constructed with a LCB (undrained). No pumping was observed in PATB (drained) sections.

Low-severity transverse joint sealant damage occurred in several test sections by 1995.

Structural distresses—including pumping, transverse joint faulting, transverse cracking, longitudinal cracking, and corner breaks—had occurred in section 260218 (203 mm PCC on 152 mm LCB).

Deviations

Data Collection and Materials Sampling and Testing Deviations

- Early in the project, elevation measurements were not taken at the required embankment layer locations.
- Elevation measurements have only a fair to poor correlation with the measured pavement thickness.
- Fresh concrete samples of section 260259 were not obtained within the limits of that test section.

- Fresh concrete samples of section 260259 were not obtained within the limits of that test section.
- The AWS was not installed until 1996. Until then, climatic data was obtained from the Toledo, OH airport, which is ±16 km away.
- Splitspoon samples were used in place of shelby tubes, due to the hardness of the subgrade and the presence of gravel and cobblestone.
- The strength of the concrete in section 260219 is well below that of the other sections constructed with 3.8 MPa concrete.
- The profile measurements were not performed until October 1994.

Site Location Guideline Deviations

- Test section 260259 (control section) has tied concrete shoulders, neoprene transverse joint seals, and hot-poured rubberized asphalt longitudinal joint seals.
- A low-volume road intersects the test sections near the middle of the experiment site, which causes a minor difference in traffic volumes and loading across the test sections. To help monitor this difference, the WIM was located to the south of the interchange and an AVC was placed on each side of the interchange.
- Test sections 260214, 260218, and 260219 are located on deep fills and on a superelevated horizontal curve.
- Vertical curves, with grades ranging from -0.81 to +0.55 percent, exist within the test section limits.
- A 762-mm concrete culvert exists ±267 m below the top of the pavement surface in section 260224.

Construction Guideline Deviations

- The moisture content of the compacted subgrade was not within the range of 85 to 120 percent of optimum for sections 260213, 260214, 260215, 260216, 260217, 260218, 260219, and 260221. This resulted in severe desiccation cracking of the subgrade that the contractor had to rework.
- The DGAB layer in section 2602121 segregated. The contractor reworked and improved the area, but some segregation still existed.
- The surface of the DGAB was not kept uniformly moist in sections 260213, 260214, 260215, and 260216.
- The underdrain filter fabric did not extend the minimum of 0.305 m under the pavement.
- Traffic was allowed on the outside shoulder of the PATB, which resulted in rutting of 13 to 44.5 mm.
- A transverse construction joint in the LCB was located within the test section limits.
- The paving equipment was allowed to operate on the outside shoulder area of the LCB, which resulted in longitudinal cracking in sections 260217 and 260220.
- Fresh LCB samples revealed a slump lower than the 25.4 mm limit for sections 260218, 260219, and 260220.

- Cores of the LCB in section 260218 did not satisfy the thickness tolerance of design ±13 mm.
- Cores of the LCB in section 260218 did not satisfy the thickness tolerance of design ±13 mm.
- Fresh concrete samples revealed a slump lower than the 25.4 mm limit in sections 260215 and 260219, and air contents lower than the 5 percent limit in sections 260214, 260219, and 260220.
- The 14-day flexural strength requirements were not satisfied.
- Cores of the concrete in sections 260213, 260214, 260217, 260218, 260222, and 260259 did not satisfy the tolerance of design ±6.4 mm.
- Test sections 260216, 260222, and 260223 had to be undercut because of unstable subgrade material.
- Test sections 260221 and 260224 had areas of unstable subgrade but were not undercut.
- Large quantities of PATB were rejected in sections 260222 and 260223 because the material was not adequately coated with asphalt.
- The contractor had problems maintaining the proper elevation for the PATB because the paver was not using a stringline.
- Test section 260213 was diamond ground to remove a "must-grind" bump.

Other Deviations

None.

NEVADA SPS-2: I-80 IN HUMBOLT AND LANDER COUNTIES, NEVADA

Project Description

The Nevada SPS-2 project site is located in north central Nevada, approximately 8 km west of Battle Mountain, in the outer eastbound lane of interstate I-80. The SPS-2 sections extend from station 1596+65 to station 64+50 (milepost 223.7).

The construction work on this segment of I-80 consisted of removing the existing AC surfacing, CTB, DGAB, and embankment. The original subgrade was stabilized with lime and the embankment was replaced. The SHRP structural sections were then placed on top of the embankment.

The terrain surrounding the test sections is generally flat with minimal ground cover.

The location of the test site is in the dry-freeze zone. Based upon climatic information collected at a Battle Mountain weather station from 1961 to 1990, the average yearly high temperature was 39 ºC, the average yearly low temperature was -26 ºC, and the average yearly precipitation was 209 mm.

The soil in this area varied throughout the project. The Nevada SPS-2 project site fills the dry-freeze, coarse subgrade categories.

Nevada SPS-2 Construction

For this project, one supplemental State section was included. This section is designated as 320259 and consisted of a 38.1-mm leveling course over the existing AC and a 267-mm PCC surface layer. This was the Nevada standard design for the remainder of the project.

Materials

This project was constructed over an existing section of highway; removal of the existing AC layer was necessary. Upon this removal, there were problems that are described in the following sections. To correct these problems, a layer of lime- stabilized soil was placed and topped with a layer of granular material to produce a suitable subbase for the test sections.

Based on laboratory testing, the natural subgrade was primarily sandy silt. The percentage of clay ranged from 4.5 to 13.9.

Summary—Nevada SPS-2 Construction

Section 320212 had severe shrinkage cracking following paving and was removed on August 4, 1995. The PATB was also torn out, and 127 mm of CTB was placed in the excavation. The CTB was followed by a 267-mm lift of Nevada DOT PCC mix.

Key Observations

This project was constructed over an existing section of highway, and removal of the existing pavement structure was required. When this was performed, the subgrade (which was sandy silt) was found to be out of specifications for NDOT subgrade material. This required the lime stabilization of the top 0.3 m of subgrade material.

After this stabilization, embankment material was placed and compacted. FWD testing on the embankment showed that sections 320201, 320205, 320207, and 320209 had significantly higher deflections than the other sections.

The DGAB was placed on 8 of the 12 sections. The material was placed in either one or two lifts, depending on the design thickness. Sections 320201 and 320209 were found to have high variations in deflections during FWD testing, and section 320203 had deflections in the first 38.1 m, while the other five sections were more consistent.

As per the SPS-2 experiment design, four sections received a 102-mm PATB. Edge drains were constructed on these sections utilizing a geotextile and open graded rock placed in trenches.

Also according to the SPS-2 experiment design, four sections had a 152-mm LCB placed directly on the embankment. The LCB was placed in one 12.19-m-wide pass and there were no joints sawed. All sections except 320206 exhibited extensive cracking within two 2 weeks of paving.

The PCC consisted of three different mixes. Section 320259 was the State standard mix, while the other 12 sections had 6 sections of a 3.3 MPa mix and 6 sections of a 5.2 MPa mix. The typical SPS-2 project has six 3.8 MPa and six 6.2 MPa mixes, but it wasn't possible to reach the 6.2 MPa target using local materials, so the target strengths were revised.

A number of problems were encountered during PCC paving. Section 320201 had sections areas that needed to be hand-finished, and shrinkage cracks appeared shortly after paving. Section 320202 had several areas of tearing in the last 91 m. Within a day of paving section 320203, shrinkage cracks appeared. There was tearing of the PCC in the areas around the dowel bar inserter (DBI) on section 320204. Transverse tie bars had to be pounded in by hand for the first 88 m of section 320205 due to an equipment failure, and there were a number of areas that required hand finishing. Shrinkage cracks also developed in sections 320205 and 320208. In section 320209 had to have, approximately every other tie bar was pounded in with a hand mallet due to equipment problems. It rained for about 15 minutes, which resulted in dimples in about 61 m of the surface of section 320209. After the fifth load of PCC on section 3202011, the 19-mm aggregate was reduced by 2 percent and the fines were increased by 2 percent. Section 320212 exhibited such severe cracking after paving that it was removed and replaced with nonconforming materials, thereby removing it from the study.

The majority of the problems with the PCC paving came as a result of the mixes being significantly different from those typically used by the paving crew. This was especially true for the 5.2 MPa mix. Proof of this fact is that section 320259, which was the State standard mix, had none of the problems with shrinkage cracks and tearing that were so common for the majority of the project. The primary conclusion that can be drawn is that trying to perform nonstandard construction can cause significant problems. It is highly unlikely that the majority of the test sections will last anywhere close to their design lives.

NORTH CAROLINA SPS-2: U.S. 52 SOUTHBOUND, DAVIDSON COUNTY

Project Description

The North Carolina SPS-2 project site is located in the southbound lanes of U.S. 23 near Lexington, NC. U.S. 52 is a rural principal arterial with an AADT of 23,500 to 26,100 (1994) and 13 percent heavy trucks. The 18-kip design KESALs was calculated as 10,784,326 for the 20-year design life of the pavement. The SPS-2 project was included in the construction of 7.8 km of U.S. 52 in both the northbound and southbound lanes. U.S. 64 bisects this SPS-2 site. All test sections except section 370204 are located north of the U.S. 64 interchange. This section will be monitored with AVC equipment to determine if the traffic south of U.S. 64 is different than traffic to the north of U.S. 64.

The roadway typically consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m (10 ft), and an inside width of 1.22 m. The shoulders were constructed with econocrete instead of the SPS-2 required flexible bituminous material. The majority of SPS-2 test sections were constructed on tangent sections with slight grades. Five test sections were completely located within horizontal curves, and one section was partially located within a horizontal curve. Sections that include a 203-mm PCC slab were constructed as add-on lanes adjacent to the mainline travel lane. This parallel roadway section was constructed through some deep cuts and high embankments.

This test site is in the wet no-freeze zone. The subgrade consists of fine-grained soils. An onsite weather monitoring station was installed in August 1994.

No information is available on the installation of WIM and AVC equipment.

Seasonal monitoring sensors, strain gauges, and linear variable differential transducers were installed on several test sections.

Reconstruction began in 1992 with earthwork grading.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 65. North Carolina test section pavement design.

Test Section No.	Typical Pavement Design
370259 (Control)	254 mm JPC on 102 mm PATB on 25.4 mm AC on 203 mm lime-stabilized subgrade
370260	279 mm JPC on 25.4 mm AC on 127 mm bituminous base on 203 mm cement- treated subgrade.

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The existing subgrade is silt, which retained a large percentage of moisture and lost its bearing capacity when wet. Soil stabilization was performed with lime slurry in all test sections except three (320204, 320207, and 320260), which were treated with cement. The subgrade was ripped to a depth of 203 mm, mixed with the applied lime, and recompacted with vibratory sheepsfoot and vibratory steel-wheeled rollers. This process was repeated the following day, and the soil was then fine-graded to the required profile grade. The top 76 mm of the stabilized subgrade in sections 370209 and 370210 (add-on lanes) had to be removed to meet the required profile grade.

Sections 370204, 370207, and 370260 were stabilized to a depth of 178 mm with cement and granular base material. Mixing and recompaction was similar to that performed on the other test sections. The subgrade in all test sections was sealed with a CRS-1 emulsion.

Base Layers

Dense-Graded Aggregate Base

The dense graded aggregate base in sections 370209 and 370210 was not treated with a prime coat before the construction of the overlying PATB.

Permeable Asphalt-Treated Base

This layer was placed with a track mounted Blaw-Knox spreader and was rolled (one pass) with a 10-ton steel wheeled tandem roller when the mat temperature was between 66 and 77 ºC. Construction traffic was allowed on the PATB after placement and cooling. Damage to the mat occurred from turning movements of the construction traffic.

Lean Concrete Base

The mix design for the LCB included 222 kg Type I cement, 67 kg Class 1 fly ash, 553 kg fine aggregate, 866 kg coarse aggregate, and 115 liters water per cubic meter. Admixtures included an air-entraining agent and a water reducer. The LCB had a 14-day compressive strength of 24 MPa. The LCB was delivered to the SPS-2 site in side-dump trucks. A Maxon spreader and a Gomaco GP 3500 slip-form paver were used in the paving operations. The LCB was paved at widths up to 7.92 m.

Portland Cement Concrete

All concrete mixes utilized on this project contained a Type I cement. The 14-day flexural strength of the 3.8 MPa mix ranged from 4.0 to 5.1 MPa, while the 6.2 MPa mix had a 14-day flexural strength of 6.1 MPa.

The PCC was delivered to the SPS-2 site in side-dump trucks. The concrete was dumped on grade ahead of the paver and spread with a front-end loader. A Maxon spreader and a Gomaco GP 3500 slip-form paver were used in the paving operations. Paving of the test sections located in the add-on lanes was accomplished by dumping the concrete from the previously constructed adjacent mainline lanes. Concrete paving commenced on October 24, 1994, and was completed on November 26, 1994. Construction joints were formed in sections 370204 and 370260. Air temperatures during concrete placement ranged from 3.3 to 20 ºC.

Key Observations

LTPP SPS Construction Reports

Edge drains were located at a 0.61-m offset from the pavement edge, rather than the SPS-2 required 2.4-m offset. Stone was used instead of PATB as trench backfill.

Econocrete shoulders were approved for use instead of asphalt shoulders.

The DGAB extended only 0.61 m into the shoulder from the pavement edge.

Dowel bars (25.4 mm in diameter) were utilized on sections that included a 203-mm thick PCC. The LCB was constructed to extend only 0.61 m into the shoulder from the pavement edge.

Cracks developed in the LCB layer in several sections before construction of the PCC. These cracks were covered with tar paper prior to PCC paving. Several of these cracks reflected through the PCC. Consequently, some of these slabs were repaired.

NORTH DAKOTA SPS-2: I-94 EASTBOUND, CASS COUNTY

Project Description

The North Dakota SPS-2 project site is located in the eastbound lanes of I-94 in eastern North Dakota, west of Fargo. I-94 is a rural interstate with a 1996 AADT of 8,310 and 12 percent trucks. The yearly ESALs in the design lane are estimated at 900,000. The design ESALs (30-year design life) is estimated at 2,150,000. The roadway typically consists of two 3.66 m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m. The SPS-2 project was included in the reconstruction of a concrete pavement that included 229 mm of concrete on 76 mm of aggregate base on 152 to 229 mm of aggregate subbase. The SPS-2 test sections were constructed on a portion of I-94 that is very flat and relatively straight. All sections except North Dakota supplemental sections 380260 and 360261 were constructed on tangent sections.

This test site is in the dry-freeze zone. An onsite weather monitoring station was installed in 1994.

WIM and AVC equipment were installed onsite.

Several delays were encountered during subgrade preparation due to the presence of extremely wet clayey soils. Construction of individual test sections was completed on October 1, 1994, and the pavements were opened to traffic on November 1, 1994.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 66. North Dakota test section pavement designs.

Test Section No.	Typical Pavement Design
380259	254 mm doweled JPC (ND mix) on 203 mm salve with skewed joints (4.57 m spacing), 3.66-m lanes
380260	279 mm doweled JPC (ND mix) on DGAB with skewed joints (4.57 m spacing), 4.27-m lanes
380261	279 mm undoweled JPC (3.8 MPa MR) on DGAB with skewed joints, 3.66-m lanes
380262	279 mm undoweled JPC on LCB with skewed joints (various lengths), 4.27-m lanes
380263	279 mm undoweled JPC on permeable asphalt base with skewed joints (various lengths), 3.66-m lanes
380264	279 mm undoweled JPC on permeable asphalt base with skewed joints (4.57 m lengths), 4.27-m lanes

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The subgrade soil can be classified as a fine-grained clay. The project is located on the remnants of glaciated Lake Agassiz. Since the SPS-2 project involved the reconstruction of an existing highway, the top 457 mm were reworked and recompacted after removal of the existing concrete pavement, base layers, and top 305 mm of subgrade. The subgrade was then loosened with a plow and recompacted with sheepsfoot rollers. Sensors were placed in unbound base layers and in the subgrade to monitor moisture conditions after construction.

Base Layers

Dense-Graded Aggregate Base

The DGAB was placed with a motor grader, rolled with an 18-ton pneumatic-tired roller, and trimmed with a CMI profiler.

Permeable Asphalt-Treated Base

This layer was drum plant mixed, paved with a Barber Greene 146 paver, and compacted with a 10-ton double-drum vibratory roller. The PATB was hard to roll and lost its form and shape once rolled. This layer was rolled at a temperature of 93 ºC. NDDOT typically places PATB beneath concrete interstate pavements and has not experienced deformation of this layer (NDDOT specifications) during construction.

Lean Concrete Base

The LCB mix contained 36.7 kg Type C fly ash, 85.3 kg Type I cement, 128.3 kg (water, 1017 kg fine aggregate, and 448 kg coarse aggregate. The LCB was paved with a REX Town and Country™ slip-form paver and a Curb Master™ paver. Initial construction problems associated with the LCB included sloughing of the edges. This was attributed to too many fines in the mix and migration of mix water to the outside edge of the LCB during finishing. This problem was corrected by adjusting the mix design to a strength higher than that specified by SPS-2.

Portland Cement Concrete

All mix designs utilized on this project contained gravel coarse aggregate. Type C fly ash was utilized in each mix design instead of Type F, as required by the LTPP protocol.

Key Observations

Deviation and Construction Reports

The project deviation report indicates that the LCB was difficult to place until the mix design was changed to increase the strength of this layer. The thickness tolerances on four core SPS-2 sections were not met (sections 380217, 380218, 380219, and 380220). Transverse cracks developed in section 380217. These cracks reflected through the 203-mm PCC within 5 days after construction of the PCC.

The PATB deformed when compacted.

The subgrade in section 380218 was unstable and should have been undercut. This caused some initial frost heave, but the condition has corrected itself.

Deviations

Data Collection and Materials Sampling and Testing Deviations

None known due to the involvement of Steve Pflipson with his computerized tracking approach.

Construction Guidelines Deviations

The layer thickness for the following sections contained deviations.

- 380217—LCB not within the 0.012 m from design. Based only on road and level.

- 380218 and 380220—LCB not within the 0.012 m from design. Based on both road and level and core results.
- 380219—LCB not within the 0.012 m from design based on core results.

LCB was difficult to place, so the mix was designed stronger than the guidelines.

PATB was difficult to roll due to its fluid-like characteristics and its short lengths required.

In test section 380217, the transverse cracks in the LCB reflected through to the 203 mm of PCC pavement.

Site Location Guidelines Deviations

The first two test sections—380260 and 380261—were built on slight superelevations just after the on-ramp from Casselton.

Other Deviations

None known.

OHIO SPS-2: U.S. 23 NORTHBOUND, DELAWARE COUNTY

Project Description

The Ohio SPS-2 project site is located in the northbound lanes of U.S. 23 in central Ohio, approximately 48 km north of Columbus. U.S. 23 is a rural principal arterial with a 1994 AADT of 20,210 with 12 percent trucks. The roadway typically consists of two 3.66-m-wide lanes, an outside asphalt shoulder width of 3.05 m (10 ft), and an inside asphalt shoulder width of 1.22 m. The SPS-2 test sections were constructed on a portion of U.S. 23 that is relatively straight and flat.

This site is in the wet-freeze zone. The subgrade is fine grained. An onsite weather monitoring station was installed in 1995.

Permanent WIM equipment consisting of weigh plates was mounted in each lane of U.S. 23. Additional instrumentation was installed in the SPS-2 experiment area to collect environmental data, including temperatures of individual pavement layers and moisture freeze/thaw conditions of the subbase and subgrade layers. Load-response monitoring instrumentation installed included strain, deflection, and pressure gauges.

Construction started in the fall of 1994 with the subgrade preparation. Individual test sections were completed by October 1995, and the project was open to traffic on October 1, 1996.

Test Sections Constructed

All required core sections were constructed. Supplemental State test sections constructed included the following:

Table 67. Ohio test section pavement designs.

Test Section No.	Typical Pavement Design
390259	279 mm JPC (3.8 MPa MR) on 152 mm DGAB
390260	279 mm JPC (3.8 MPa MR) on 102 mm PATB on 102 mm DGAB
390261	279 mm JPC (3.8 MPa MR) on 102 mm CTPB on 102 mm DGAB
390262	279 mm JPC on 102 mm CTPB on 102 mm DGAB
390263	279 mm JPC on 152 mm DGAB
390264	279 mm JPC on 152 mm DGAB
390265	279 mm JPC (3.8 MPa MR) on 102 mm PATB on 102 mm DGAB

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The subgrade was compacted in 305-mm lifts with a 22.1-ton sheepsfoot roller. No stabilizing agents were used.

Base Layers

Dense-Graded Aggregate Base

The DGAB was placed in 229-mm lifts and was compacted with a 16.5-ton single-drum vibratory roller to obtain a 152-mm layer thickness. The 152-mm lift thicknesses were similarly compacted to a 102-mm layer thickness. A CMI trimming machine was used to trim excess material; however, final constructed DGAB layer thicknesses were generally in excess of design thicknesses.

Permeable Asphalt-Treated Base–

This layer was placed at a 127- to 152-mm lift thickness with a Blaw-Knox PH200B paver and compacted to 102 mm with 15 passes from a 7-ton steel-wheeled tandem roller. The single-pass laydown width for this paver was 3.66 m; therefore; monolithic construction into the shoulder was not obtained.

Lean Concrete Base

The mix design for the LCB base utilized a water reducer but no fly ash. This material was paved with a Gomaco GP2500 slip-form paver and spreader, which had a laydown width of 7.92 m. Finishing included screeding and the application of a membrane-curing compound to the LCB. Only partial monolithic construction into the shoulder area occurred.

Cement-Treated Permeable Base

The mix design for this material included 1136 kg of coarse aggregate, 113 kg of cement, and 38 kg of water. This material was paved with a CMI slip-form paver with a laydown width of 9.45 m. Finishing was performed by screeding, and white polyethylene was applied to the surface.

Portland Cement Concrete

All mix designs utilized on this project contained Type I cement, 100 percent crushed stone for the coarse aggregate, and 100 percent manufactured sand for the fine aggregate. Type C fly ash was utilized in each mix design instead of Type F, as required by the LTPP protocol.

Dowel baskets were placed at the transverse joints. Finishing was performed by screeding and the surface was tine textured. A membrane-curing compound was applied to the surface. Transverse joint sawcut depths averaged 64 mm for the 203-mm JPC and 89 mm for the 279-mm JPC. Surface profiles were corrected by diamond grinding. An additional sawcut produced a transverse joint sealant reservoir 12.7 mm wide by 25.4 mm in depth. The transverse joints were sealed with low-modulus silicone.

Key Observations

Deviation Report

The LTPP SPS project deviation report indicates that some of the DGAB cracked during compaction (sections 390259 and 390204). Contaminated PATB was removed and replaced due to an oil spill in section 390260.

Construction Report and Data Evaluation

Monolithic construction of base layers would have ensured that a layer of uniform thickness and material quality was constructed transversely across the typical pavement section. This would have resulted in the highest support conditions at the pavement edge, which is often the most critical stress area (edge stresses and positive curling stresses) for a doweled JPCP. Only the CTPB width can be considered monolithic.

Individual pavement layer thicknesses are often in excess of LTPP tolerances. Variability of a single layer depth occurs both within an individual test section and from section to section for those test sections that have common layer depth requirements. The constructed depth of the JPC may have the largest effect on pavement performance if subgrade strength and subsurface drainage quality are relatively uniform throughout all test sections. The allowable KESALs were developed from the *AASHTO Supplemental Design Guide for Rigid Pavements* and a computer program with environmental data from the nearest SHRP LTPP data site, design thickness, and strength values from SPS-2 protocols (average material values were used when these values are not defined in the SHRP SPS-2 protocol).

Deviations

Construction Guidelines Deviation Comments

Sections 390259 and 390204: Some surface aggregate cracked due to compaction.

Section 390260: Oil spilled on PATB. Contaminated sections were removed and replaced.

All sections: Unbound aggregate base layers cut to grade using a CMI trimming machine.

Other Deviations

None known.

Site Location Guidelines Deviations

None known.

Data Collection and Materials Sampling and Testing Deviations

Several test locations were moved due to obstructions. These are noted on testing log sheets.

Several other minor sampling deviations are noted on sampling data sheets.

WASHINGTON SPS-2: S.R. 395 NORTHBOUND, ADAMS COUNTY

Project Description

The Washington SPS-2 project site is located in the northbound lanes of S.R. 395 in eastern Washington, 4.8 km south of Ritzville. S.R. 395 is an urban principal arterial with a 1993 AADT of 18,000. The designs ESALs for this project are 35 million for a 40-year design life. The SPS-2 project includes construction of two new northbound lanes and the upgrade of S.R. 395 to a four-lane divided highway. The new lanes were constructed uphill from the existing lanes. Two sections were located in a cut (section 530203 and 530259), while all other test sections were located on fills. The roadway design for this project consists of two 3.66-m-wide lanes, an outside shoulder width of 3.05 m, and an inside shoulder width of 1.22 m.

The initial SPS-2 test sections were constructed on a horizontal curve to the left from the beginning of the SPS-2 project to station 2050+00. Section 530201 is partially located within a horizontal curve and partially located within a superelevation runout area. Sections 530205, 530206, 530207, and 530208 are on tangent, while the remainder of the test sections were constructed on a curve to the left. The maximum superelevation rate for this curve is 3 percent. Vertical grades range from 0.14 percent to 3 percent.

This test site is in the dry-freeze zone. The average high temperature is 37 ºC, while the average low temperature is –14.6 ºC. It is not known if an onsite weather monitoring station was ever installed.

Construction of the test sections started in June 1993 with the removal of the existing pavement. Construction of individual test sections was completed by November 1, 1995.

Test Sections Constructed

All required core sections were constructed. The supplemental State test sections constructed included the following:

Table 68. Washington test section pavement design.

Test Section Number	Typical Pavement Design
530259 (Control)	Undoweled 254 mm JPC (4.5 MPa) on 76 mm ATB on 50.8 mm crushed surfacing base course; 4.27-m lane

Materials and Construction Methods of Individual Pavement Layers

Subgrade

The existing subgrade is classified as fine-grained sandy silt. Borrow excavation for embankment construction came from three sources within the project area. All three sources were sandy silt of very low plasticity. Completed embankment depths ranged from 0.84 to 1.75 m in depth.

The existing ground from station 2004+00 to the end of the SPS-2 project was saturated and required removal before subgrade and embankment construction. Undercutting of the existing subgrade was performed from station 2004+00 to station 2085+50. After overexcavation was completed, the contractor refilled these areas with shot rock. This material was a volcanic stone with a 457-mm top size. It was placed to act as a drainage layer and to provide a stable platform for roadway construction. Borrow excavation consisted of silt and was constructed on top of the shot rock to the finished subgrade elevation. Compaction of the embankment was completed with a vibrating steel-wheeled roller and a 17-ton steel-wheeled roller. This layer was compacted at 100 percent compaction, but at a moisture level of 5.89 percent below optimum. The embankment was trimmed to final grade with a laser-controlled grader.

Base Layers

Dense-Graded Aggregate Base

The DGAB consisted of a crushed stone with a 31.75-mm top size. A 7-ton vibrating steel-wheeled roller was used to compact the DGAB. Construction traffic was allowed on the completed DGAB, and no significant damage was noticed. DGAB layers, which would be overlaid with PATB, were prime-coated. Construction traffic was again allowed to run on the prime-coated DGAB. Minor tracking and bleeding of the prime coat occurred. Average compaction achieved on the DGAB layer was 97.7 percent of the optimum density.

Permeable Asphalt-Treated Base

The PATB contained an AR400W binder at 4.5 percent and an antistrip agent. An average of 87 percent of the crushed basalt utilized for the PATB passed the 0.5-inch sieve, while an average of 7.5 percent passed the No. 200 sieve. The PATB had a density of 2.56 g/cm^3 (159.5 pcf) and 7.5 percent voids. This layer was placed at 77 to 82 ºC with a Blaw-Knox PF150 paver. Profile grade control was maintained with a wire guide line and/or paving skis. The PATB was compacted at 66 ºC by two passes each of a 17-ton steel-wheeled vibratory roller and a 10.5-ton static steel-wheeled roller. Compacted densities averaged 84 percent of maximum density (82 to 87 percent ranges). No appreciable construction traffic was allowed on the PATB.

Lean Concrete Base

The mix design for this material included 10.1 kg Type II cement, 1.13 kg Type F fly ash, 77.5 kg coarse aggregate (crushed basalt), 81.1 kg fine aggregate, 11.6 kg water, a water reducer, and an air-entraining agent. The water-cement ratio was 1.03. This mix design produced an average 7-day compressive strength of 4.1 MPa. Cored 14-day compressive strengths varied between 3.7 and 6.4 MPa, while companion cylinder breaks at 14 days varied between 2.0 and 6.4 MPa.

The LCB was paved with a Guntert Zimmerman™ slip-form paver with a hydraulic spreader and trowel. Laydown widths varied between 5.08 and 6.50 m. The LCB construction joint was located 1.62 m to the right of the PCC joint for both 4.27-m- and 3.66-m-wide lanes. Minor transverse and longitudinal cracking occurred, with most of the longitudinal cracking occurring in the outside shoulder.

Portland Cement Concrete

The coarse aggregate for both SPS-2 mixes was a crushed basalt, and the fine aggregate consisted of 88 percent crushed basalt and 12 percent natural sand. All mix designs on this project utilized Type II cement. The 3.8 MPa mix included a Class F fly ash.

The PCC paving train consisted of a track-mounted Guntert Zimmerman slip-form paver, a track-mounted spreader with side-loading conveyor belts, a float finisher, tining machine, and a curing machine. Joint sawing was performed at least 12 hours after concrete placement.

JPC paving began on September 25, 1995, and was completed on October 3, 1995. The project was opened to traffic on November 1, 1995. Average daily temperatures ranged between 5.3 and 19.9 ºC, and the relative humidity ranged from 30.8 to 95.9 percent. The maximum relative humidity exceeded 90 percent in the morning for each day of paving, but decreased to less than 42 percent by 5:00 p.m. for each day except September 28. The maximum relative humidity on this day stayed above 75 percent, while average temperatures ranged between 9.4 and 26. 2 ºC.

The water-cement ratio averaged 0.455 for the 3.8 MPa mix and 0.286 for the 6.2 MPa mix.

The 3.8 MPa mix had an average 14-day flexural strength of 3.3 MPa (0.38 standard deviation) and a 28-day flexural strength of 4.3 MPa (0.52 standard deviation). The 6.2 MPa mix had an average 14-day flexural strength of 5.7 MPa (0.24 standard deviation) and a 28-day flexural strength of 6.5 MPa (0.59 standard deviation).

For each SPS mix design (3.8 MPa or 6.2 MPa), beam flexural strengths, cylinder and core compressive strength, and splitting tensile strengths showed consistency with each design strength.

Key Observations

Construction Report and Data Evaluation

Construction traffic helped to further consolidate the DGAB, as evidenced by an average density of 2,106 kg/m^3 for those DGAB sections receiving construction traffic and an average density for the control section (section 530259, which did not receive construction traffic) of 1,867 kg/m^3.

Six of the eight test sections constructed with DGAB had average thicknesses between 10 and 23 mm greater than SPS-2 specifications.

The average ATB thickness was 66 mm with a 10-mm standard deviation. The SPS-2 specified thickness was 76 mm, ±6.4 mm.

The average LCB thickness was either 155 or 157 mm for each test section paved with LCB. The SPS-2 specified thickness was 152 mm, ±6.4 mm.

The 203-mm PCC test sections had average thicknesses ranging from 211 to 206 mm. Test sections 530201, 530206, and 530209 had thicknesses of 221, 218, and 216 mm, respectively.

All 279-mm PCC test sections had PCC thicknesses within 7.6 mm of the specified depth.

The 14-day core compressive strengths for three of the four LCB test sections were within SHRP tolerances of 3.4 to 5.2 MPa. Section 530207 had compressive strengths up to 2.5 times as high as other LCB test sections. This was attributed to a water-cement ratio lower than the mix design.

All but one PATB test section had an average thickness of either 97 or 99 mm. Section 530212 had an average PATB thickness of 89 mm.

The 3.8 MPa mix had hairline cracks below the sawn transverse joint and 6.4-mm joint widths several days after paving for the DGAB and LCB sections. The 203-mm PCC on LCB (section 530205) had not cracked at the transverse joints by October 2, 1995.

The 6.2 MPa mix had larger cracked joint widths than the 3.8 MPa mix for corresponding sections.

The 3.8 MPa mix had cracked joints up to 7.9 mm in the PATB sections, while the 6.2 MPa mix had transverse joint crack widths averaging 13 mm on PATB sections.

Section 530206 developed shrinkage cracks from 1.6 to 3.2 mm in width. All but 1 slab was cracked, and 19 of the 32 slabs had more than 5 cracks per slab.

Transverse and longitudinal joints were sealed with a hot- poured material.

FWD testing revealed that those sections constructed in cut areas had the most variability in support (0.4 to 1.4 mm), while those test sections constructed on embankments had more uniform support.

WISCONSIN SPS-2: S.H. 29, WESTBOUND, MARATHON COUNTY

Project Description

The Wisconsin SPS-2 project site is located on the westbound and eastbound Wisconsin State Highway 29 (S.H. 29) in Marathon County, WI. This site is roughly 5.6 km east of Hatley, WI. The site is located on a 0.3 percent downgrade with four curves in between. The maximum curve does not exceed 2 degrees with a superelevation equal to 0.055 l/l.

The lanes are 3.66 m and 4.27 m wide, with an outside shoulder of 3.05 m and an inside shoulder of 1.83 m.

The project is located on a four-lane section of S.H. 29, which is classified as a rural arterial. As of 1995, the current average daily traffic was 6,650 vehicles with a truck distribution of 29.5 percent. This site had a design life of 20 years.

This SPS-2 project was planned for the wet-freeze environmental zone and on a coarse-grained subgrade. An AWS unit was installed in June 1997. A WIM system was installed on August 29, 1997. The WIM equipment used was a DAW-1000 bending plate unit.

The subgrade preparation for this project began in early June 1997, and paving operations were completed by mid-October 1997.

Test Sections Constructed

All required core sections were constructed. This SPS-2 project also incorporates eight Wisconsin DOT supplemental sections. Their designs are described below:

Table 69. Wisconsin test section pavement designs.

Test Section Number	Typical Pavement Design
550259	279 mm JPC (3.8 MPa MR) on 152 mm DGAB
550260	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with alternate dowel bar placement
550261	203 mm JPC (3.8 MPa MR) on 102 mm OGDB on 102 mm DGAB
550262	203 mm JPC (6.3 MPa MR) on 152 mm DGAB, with tied concrete shoulder
550263	203-279 mm JPC (3.8 MPa MR) on 152 mm DGAB, variable pavement thickness
550264	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with composite dowels
550265	279 mm JPC (3.8 MPa MR) on 152 mm DGAB, with stainless steel dowels
550266	Information not available

Materials and Construction Methods of Individual Pavement Layers

Subgrade

Scrapers, bulldozers, and pushcarts were used to compact the subgrade. The lift thickness was typically 203 mm. Remnants of old PCC pavement were found in the subgrade when sampling using shelby tubes. The PCC slabs were removed, and the subgrade was reworked to bring it back to the required elevation.

Base Layers

Dense-Graded Aggregate Base

The DGAB and CSOGB (cement stabilized open graded base) thicknesses were 102 and 152 mm (4 and 6 in), respectively. Compaction was achieved using scrapers, bulldozers, and pushcarts. Typically, a 203-mm lift thickness was used for 152-mm layers, and a 152-mm lift was used for 102-mm layers. This procedure frequently resulted in a layer that was too thick. Therefore, a CMI trimming machine was used to achieve the proper layer thickness.

Permeable Asphalt-Treated Base

All PATB base layers were 102 mm thick. A Rex R28 was used for paving. The paver had a single- pass laydown width of 3.66 m and, typically, a first- lift placement thickness of 127 to 152 mm. The asphalt was obtained from a local plant located 3.2 km from the test site with a hauling time of 10 minutes.

Lean Concrete Base

The LCB were paved with a Rex R28 slip-form paver. This machine has a 7.3-m-wide laydown width. The concrete was obtained from a local concrete plant located 1.61 km from the test sections. Vibrating screeds were used to consolidate materials. Finishing was done by screeding, and a membrane-curing compound was placed on the LCB.

Mix Designs and Concrete Paving

Two different mix designs were used in this SPS-2 project. All used a La Farge™ Type II cement. The coarse aggregate was made of 100 percent crushed aggregate, and the fine aggregate was composed of 100 percent manufactured sand.

A Rex R28 slip-form paver paved the PCC layer. The width paved in one pass varied from 6.1 to 7.9 m. The cement mixture was consolidated using internal vibrators. Vibrators were placed 152 mm below the surface approximately 610 mm apart. Finishing was done by screeding, and a membrane-curing compound was used. The surface was textured using a tine.

Deviations

Construction Guidelines Deviation Comments

All sections: Unbound aggregate base layers were cut to grade using a CMI trimming machine.

Site Location Guidelines Deviations

None known.

Data Collection and Materials Sampling and Testing Deviations

During the splitspoon testing, a number of areas were found to have existing concrete slabs beneath the old pavement structure. These areas of concrete were removed and fill was placed in these areas.

Because of the process used to remove the existing pavement, it was not possible to obtain undisturbed samples of the existing base or subbase material.

Soil boring records were provided that made it unnecessary to perform shoulder probes. The depth to rigid layer exceeded 6.1 m.

Several other minor sampling deviations are noted on sampling data sheets.

Other Deviations

None known.

References

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3. *Specific Pavement Studies Material Sampling and Testing Requirements for Experiment SPS-2 Strategic Study of Structural Factors for Rigid Pavements*, Unpublished Internal LTPP Documents, Federal Highway Administration, McLean, VA, January 1994.

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5. LTPP Directive D-05, *Measurement Frequency and Priorities of Manual Distress Surveys*, Unpublished Internal LTPP Documents, Federal Highway Administration, McLean, VA, March 1995.

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10. LTPP Directive GO-21, *LTPP Test Section Monitoring Adjustments*, Unpublished Internal LTPP Documents, Federal Highway Administration, McLean, VA, September 1999.

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