Using Falling Weight Deflectometer Data with Mechanistic-Empirical Design and Analysis, Volume II: Case Study Reports

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FOREWORD

This report documents a study conducted to investigate the use of the falling weight deflectometer (FWD) as part of mechanistic-empirical pavement design and rehabilitation procedures incorporated within the *Mechanistic-Empirical Pavement Design Guide (MEPDG)* developed by the National Cooperative Highway Research Program and subsequently adopted by the American Association of State Highway and Transportation Officials. The first volume of this three-volume report documents general pavement deflection-testing procedures and commonly used deflection analysis approaches and a review of backcalculation programs for flexible, rigid, and composite pavement structures. The relevance of the different procedures and approaches to the MEPDG were explored through examination of six case studies evaluated using FWD testing results in the MEPDG, and the findings are presented here in the second volume. Based on the case study findings and information from the literature, best practice guidelines for effective testing of existing pavement structures and interpretation of those results as part of a mechanistic-empirical pavement evaluation and rehabilitation process were developed and are presented in the third volume. This report is intended for use by pavement engineers as well as researchers involved in rehabilitation design and management of agencies' pavements.

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Director, Office of Infrastructure
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16. Abstract

The need to accurately characterize the structural condition of existing pavements has increased with the recent development, release, and ongoing implementation of the *Mechanistic-Empirical Pavement Design Guide* (MEPDG). A number of different material inputs are required in the procedure, and it is important to adequately characterize and define them. The analysis of deflection data collected by the falling weight deflectometer (FWD) provides a quick and reliable way to characterize the properties of the paving layers, as well as to assess the load-carrying capacity of existing pavement structures. With the release of the new MEPDG, there is a pressing need to identify and evaluate the way that FWD testing is integrated into the new design procedure. Moreover, as highway agencies continue to implement the MEPDG, best practices guidance is needed on how to effectively test existing pavement structures and interpret the results as part of a mechanistic-empirical pavement evaluation and rehabilitation process.

This document is part of a three-volume report investigating the use of the FWD as part of mechanistic-empirical pavement design and rehabilitation procedures. In this volume, six case studies—flexible pavement, flexible pavement on rubblized portland cement concrete (PCC), rigid pavement on granular base, rigid pavement on stabilized base, rigid pavement on existing flexible pavement, and composite (hot-mix asphalt (HMA) over PCC pavement)—were used to evaluate how FWD deflection data are used in the rehabilitation portion of the MEPDG. The case studies used data from in-service pavements. Specifically, deflection data and backcalculation results were used to characterize the existing HMA, PCC, stabilized and unstabilized bases, and aggregate and subgrade properties in the MEPDG design program. Laboratory testing results were compared with FWD results, and the final designs were found to be relatively insensitive to the differences in characterization of existing layer inputs.

This is volume II of a three-volume report. The other volumes in the series are FHWA-HRT-16-009, Volume I: *Final Report*, and FHWA-HRT-16-011, Volume III: *Guidelines for Deflection Testing, Interpretation, and Analysis.*

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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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LIST OF ABBREVIATIONS

AADT	annual average daily traffic
AADTT	average annual daily truck traffic
AASHTO	American Association of State Highway and Transportation Officials
COV	coefficient of variation
CTE	coefficient of thermal expansion
FWD	falling weight deflectometer
HMA	hot-mix asphalt
ICM	Integrated Climatic Model
IRI	International Roughness Index
JPCP	jointed portland cement concrete pavement
JRCP	jointed reinforced concrete pavement
LTE	load transfer efficiency
LTPP	Long-Term Pavement Performance
MEPDG	*Mechanistic-Empirical Pavement Design Guide*
PCC	portland cement concrete
PG	performance grade
RMS	root mean square
RPCC	rubblized portland cement concrete
SHRP	Strategic Highway Research Program
SPS	Specific Pavement Study
USGS	United States Geological Survey

CHAPTER 1. CASE STUDY 1: FLEXIBLE PAVEMENT EVALUATION AND OVERLAY DESIGN

PROJECT OVERVIEW

Project 30-0100, located on I-15 near Great Falls, MT, consisted of 12 test sections. Test section 30-0113 was selected as a flexible pavement rehabilitation case study. Test section 30-0113 was a flexible pavement cross section with a hot-mix asphalt (HMA) surface and aggregate base layer over subgrade. This test section is representative of the following selection factors:

- Flexible pavement.
- Dry-freeze climate zone.
- Principal interstate—rural functional class.
- "Fair" pavement condition.
- Sand subgrade classification.
- No (or deep) rigid layer.

The original pavement, nominally a 4-inch (102-mm) HMA surface on 8-inch (203-mm) aggregate base, was constructed in 1997. Rehabilitation and repair work—consisting of crack sealing and an aggregate seal coat—were performed in 2003 and 2004, respectively (construction numbers 2 and 3, respectively).

PAVEMENT CONDITION/PERFORMANCE

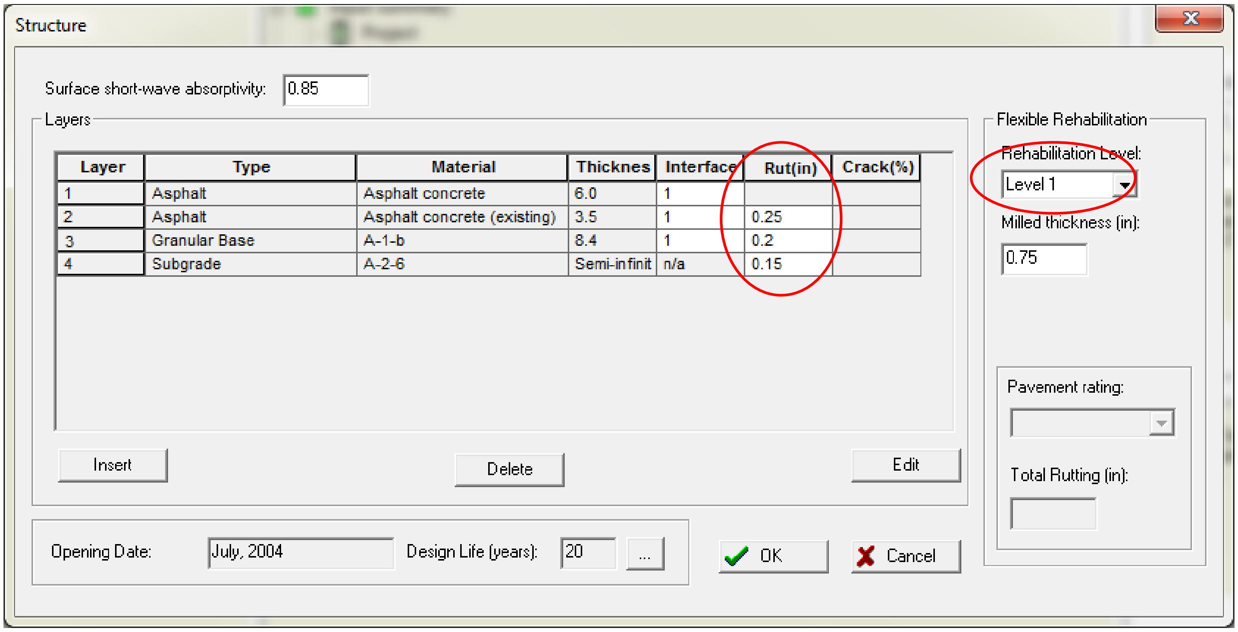
Pavement condition and performance data can be used to customize (or calibrate) the performance models within the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) software for the specific State using the design procedure.(1) However, the calibration of performance models for specific States was beyond the scope of this study. The design inputs after performance model calibration used relatively little distress data. The distress data used for design inputs included the percentage of cracking, rutting depth, and overall condition, depending on design level being used.

The pavement distress data used in the rehabilitation design depend on the design level hierarchy selected. The distress data used for the level 1 analysis are summarized in table 1 and presented in figure 1. Level 1 analysis uses the rutting attributed to each layer of the cross section. Because trenching data were not available, the individual layer rutting contributions were estimated.

Table 1. Summary of existing pavement distress inputs for level 1.

Laboratory Test	Rutting (inches)
Existing HMA	0.25
Granular base	0.2
Subgrade	0.15

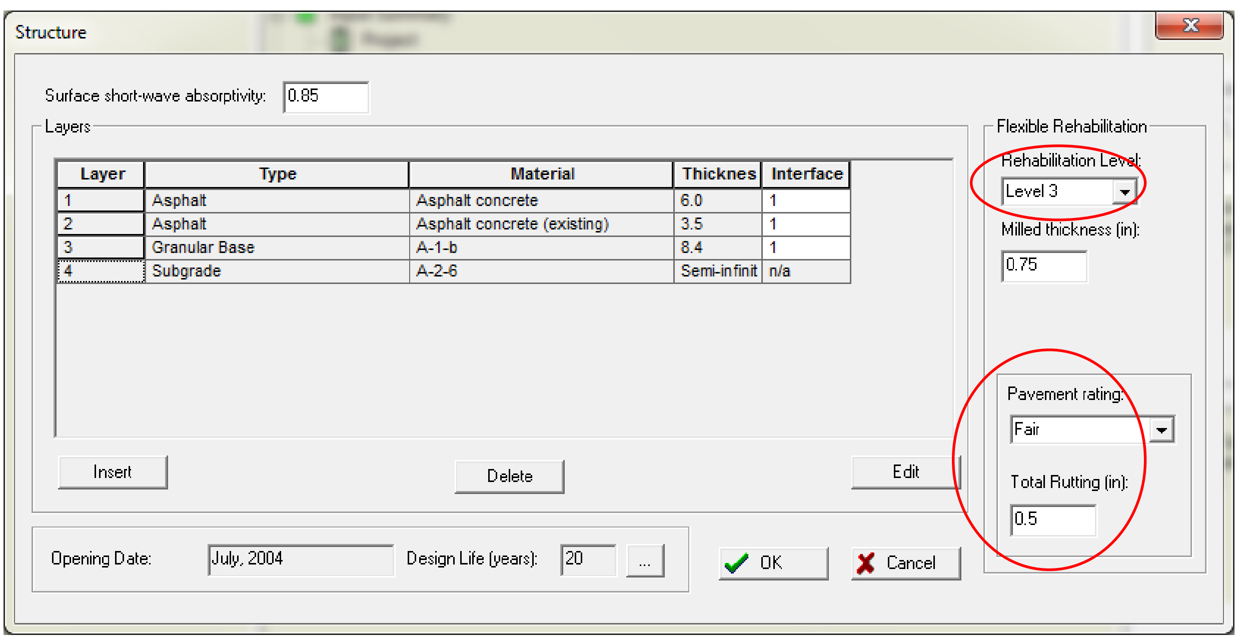
1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 1. Screen Capture. Summary of condition inputs for level 1.

Level 2 design includes the percent cracking in the existing HMA surface in addition to the layer rutting, and level 3 uses the overall rut depth (not individual layer contributions) and the overall pavement rating (good, fair, or poor), as shown in figure 2.



1 inch = 25.4 mm.

Figure 2. Screen Capture. Summary of condition inputs for level 3.

TEST SECTION DATA

The MEPDG design program requires a significant number of inputs, particularly for level 1 analysis. The required design data for the 30-0100 project section were obtained from the Long-Term Pavement Performance (LTPP) Program DataPave database. The required data were not generally complete for any one specific test section within project 30-0100; therefore, results for the entire project were used to obtain the necessary design inputs. Although the test sections had slightly varying cross sections and maintenance histories, the research team concluded that the overall initial data were sufficient for this study.

Deflection-Testing Data

Deflection data for test section 30-0113 were available from the LTPP Program database for several years of testing, including 1998, 1999, 2001, 2003, and 2005. Deflection testing was conducted following the LTPP Program protocols.

Equipment

Deflection testing was conducted with a Dynatest® falling weight deflectometer (FWD) (SN 8002-131 for 1999 through 2005).

Sensor Configuration

Sensors were located at 0, 8, 12, 18, 24, 36, and 60 inches (0, 203, 305, 457, 610, 914, and 1,524 mm) for the 1999 datasets and 0, 8, 12, 18, 24, 36, 48, and -12 inches (0, 203, 305, 457, 610, 914, 1,219, and -305 mm) for data collected after 1999.

Number of Drops and Load Levels

Four load-level targets—6,000, 9,000, 12,000, and 16,000 lb (2,724, 4,086, 5,448, and 7,264 kg)—with four drops at each load level were performed, and data were recorded. Seating drops were also performed, but data were not recorded.

Test Locations/Lanes and Increments

Testing was conducted in the outer wheelpath and mid-lane at 50-ft (15.3-m) intervals, with the outer wheelpath and mid-lane locations at the same stations (i.e., not staggered).

Temperature Measurements

Temperature measurements were taken using drilled holes in the pavement at 1-, 2.2-, and 3.3-inch (25.4-, 55.9-, and 83.8-mm) depths at prescribed time intervals during deflection testing.

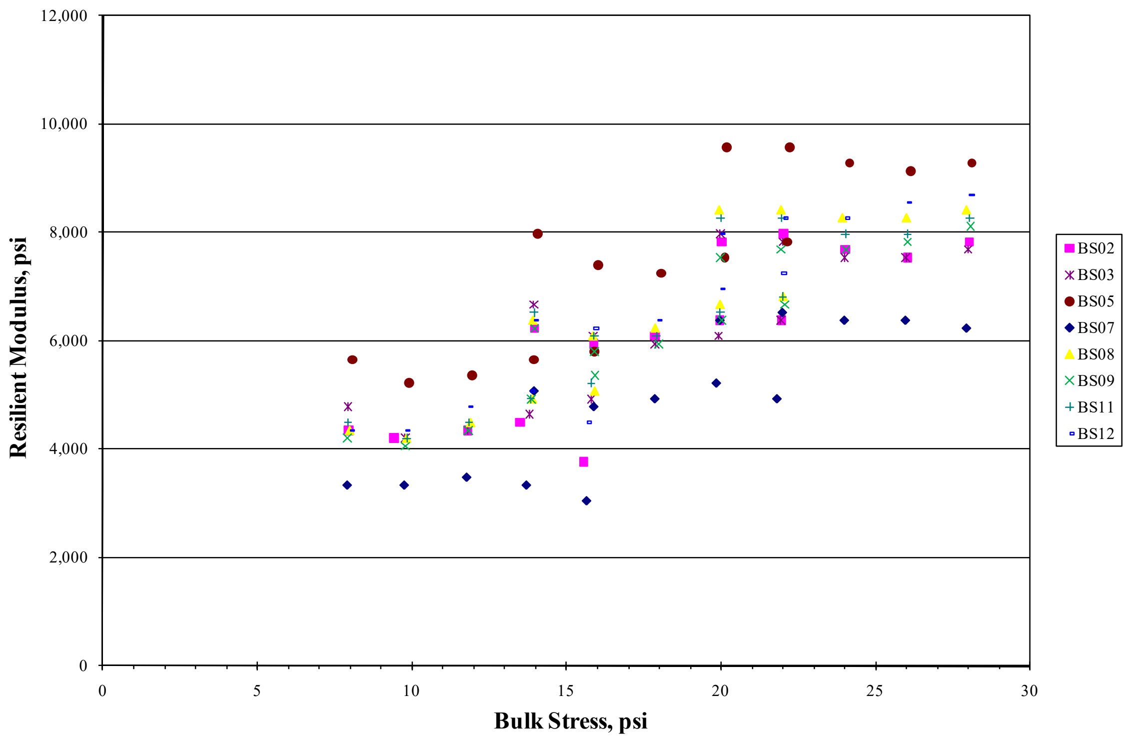
Material Properties Data

Laboratory testing data for unbound materials were available from 2000 and 2002 and for HMA materials from 1999.

Subgrade

Eleven subgrade samples were retrieved from the 30-0100 project location as part of the LTPP Program. The subgrade was classified as A-2-6 under the American Association of State Highway and Transportation Officials (AASHTO) soil classification system.[(2)]

Laboratory resilient modulus testing results were available for several samples of the subgrade materials obtained as part of the LTPP Program data collection. Ten subgrade samples were tested for the project 30-0100 test sections; however, 2 samples were removed from this data analysis because the samples were classified as different soil types than the remaining samples, as discussed previously. Laboratory resilient modulus testing results for the subgrade samples (identified as BS ##) are illustrated in figure 3. Additional subgrade properties, including Atterberg limits and sieve analysis, are summarized in table 2 and table 3.



1 psi = 6.89 kPa.

Figure 3. Graph. Summary of LTPP Program laboratory-measured subgrade resilient modulus from field samples.

Table 2. Summary of subgrade Atterberg limits.

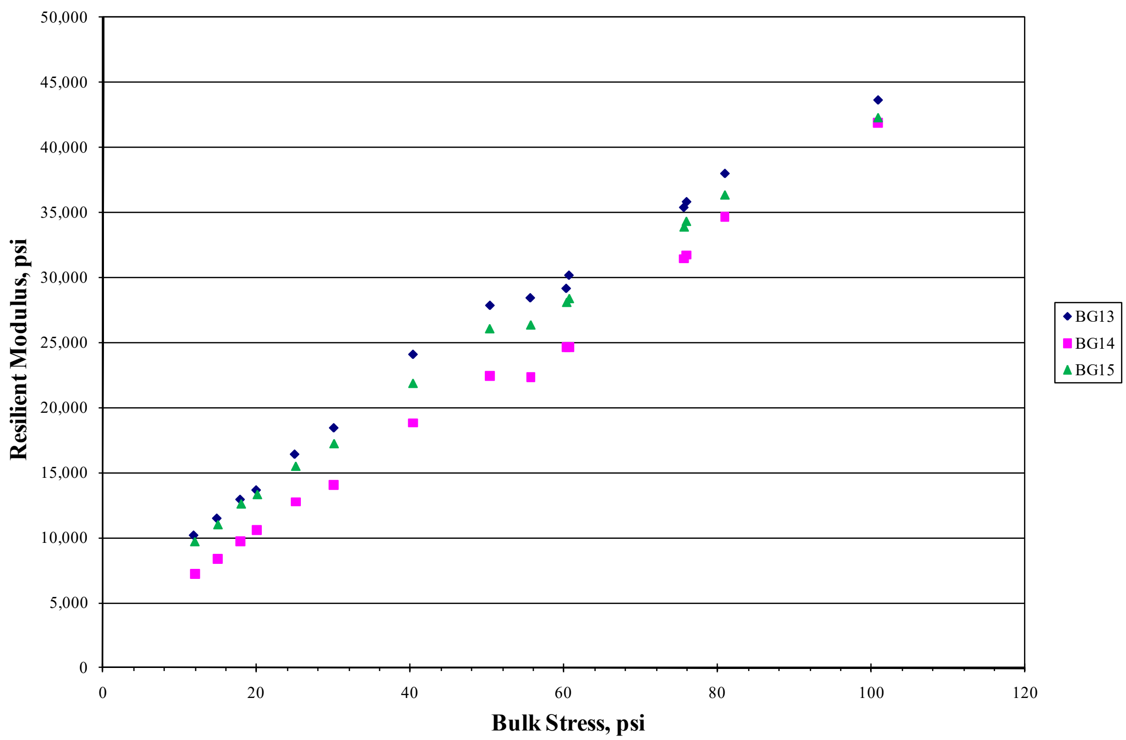
Laboratory Test	Average Test Result (percent)
Liquid limit	33
Plasticity index	16

Table 3. Summary of subgrade sieve analysis.

Sieve Size	Average Percent Passing
No. 4	100
No. 10	99
No. 40	98
No. 80	61
No. 200	27.8

Base Aggregate

Three coarse aggregate samples were obtained as part of the LTPP Program data collection and were classified as AASHTO A-1-b. Laboratory resilient modulus testing results were also available for three samples of coarse aggregate materials in the project 30-0100 test sections. Laboratory resilient modulus testing results for the coarse aggregate samples (identified as BG ##) are illustrated in figure 4.



1 psi = 6.89 kPa.

Figure 4. Graph. Summary of LTPP Program laboratory-measured base aggregate resilient modulus from field samples.

The average thickness of the base from available data was 8.4 inches (213 mm). Additional aggregate properties are summarized in table 4 and table 5.

Table 4. Summary of base aggregate Atterberg limits.

Laboratory Test	Average Test Result (percent)
Liquid limit	11
Plasticity index	1

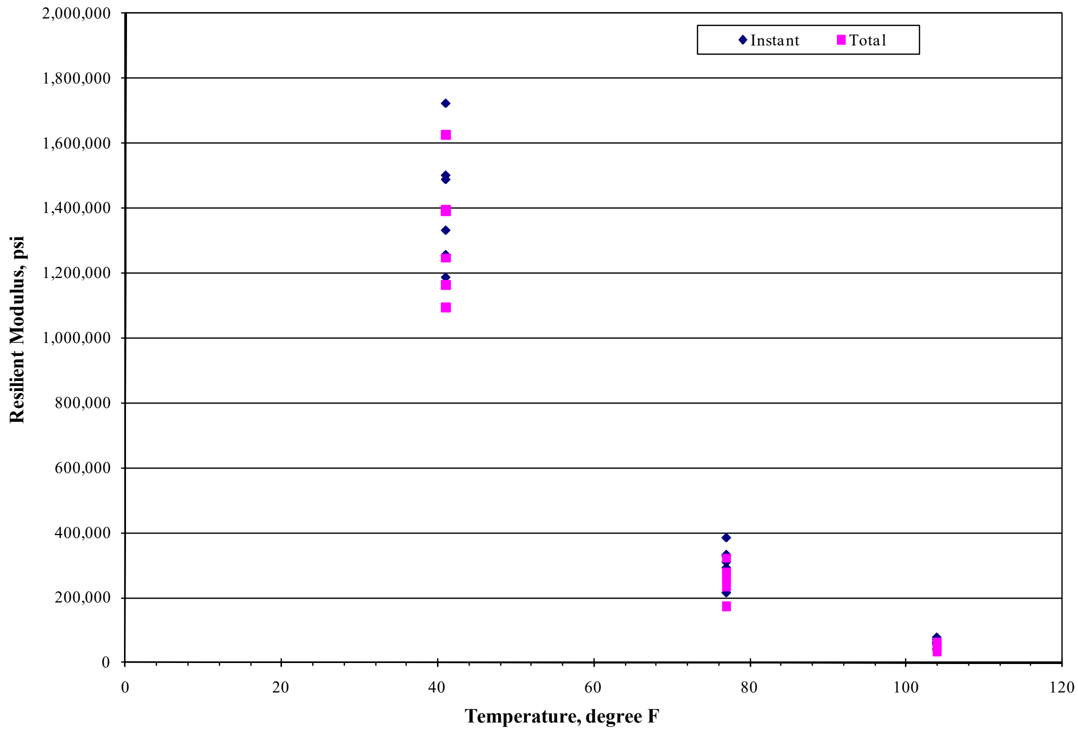
Table 5. Summary of base aggregate sieve analysis.

Sieve Size	Average Percent Passing
1.5 inch	96.7
1.0 inch	93.6
$^{3}/_{4}$ inch	90.8
$^{1}/_{2}$ inch	85.8
$^{3}/_{8}$ inch	82.3
No. 4	74.2
No. 10	64.0
No. 40	37.6
No. 80	20.8
No. 200	13.4

1 inch = 25.4 mm.

HMA Surface

Based on the data for the section, the average HMA thickness was 4.3 inches (109 mm). Construction data indicate the HMA was placed in two approximately equal lifts of the same material. The LTPP Program database contained laboratory resilient modulus testing results for six HMA surface samples from the 30-0100 project section. Dynamic modulus testing data (AASHTO Test Procedure 62) were not available.[3] Testing was performed on 1.5-inch (38-mm)-thick test samples. The average resilient modulus testing results—instantaneous and total—are summarized in figure 5. Additional HMA properties for design are summarized in table 6 and table 7.



1 psi = 6.89 kPa.
°F = 1.8 × °C + 32.

Figure 5. Graph. Summary of LTPP Program laboratory-measured average HMA resilient modulus.

Table 6. Summary of existing HMA material properties.

Variable	Value
Asphalt grading	Performance grade (PG) 70-28
Asphalt content (percent)	9.8
Air voids, percent	7.4
Total unit weight (lb/ft^3)	143.5

1 lb/ft^3 = 0.0160 g/cm^3.

Table 7. Summary of existing HMA aggregate sieve analysis.

Sieve Size	Average Percent Passing
3/4 inch	100
3/8 inch	78
No. 4	50
No. 200	6.5

1 inch = 25.4 mm.

Depth to Rigid Layer/Water Table

The depth to the water table is also required by the MEPDG. Unfortunately, no project-specific data were found for project 30-0100. One record was obtained from the LTPP Program database for test section 30-8129, with measurements obtained during May and June 1997. The depth to the water table at that time averaged approximately 9.4 ft (2.9 m). Historical groundwater levels from the United States Geological Survey (USGS) indicate an average of approximately 12 ft (3.7 m) below land surface.[4] Again, this monitoring location was not at the project 30-0100 site but was in the Great Falls, MT, area.

Climate/Environment Data

Climate data were obtained from the updated climate files on the MEPDG Web site.[5] The weather station at Great Falls, MT, was used for this study. The general weather category for the case study location was dry-freeze.

Traffic Data

Traffic data were obtained from the LTPP Program database. Because evaluating the effect of traffic data on design results was not a primary goal of this study, only basic information was used from the available data (i.e., total volume, growth, and vehicle class distribution), with the remaining inputs (i.e., monthly distribution, hourly distribution, and wheel spacing) kept at their default values in the MEPDG software. A 0.7-percent compounded growth rate was estimated based on traffic data. An average annual daily truck traffic (AADTT) volume of 657 vehicles per day was estimated. The vehicle class distribution obtained from the LTPP Program database is summarized in table 8.

Table 8. Traffic distribution by vehicle class.

Vehicle Class	Traffic Distribution (percent)
4	1.8
5	24.6
6	7.6
7	0.5
8	5.0
9	31.3
10	9.8
11	0.8
12	3.3
13	15.3

ANALYSIS AND INTERPRETATION OF FWD TESTING DATA

This section presents the data checks and backcalculation analysis of the FWD data, as well as a comparison of the backcalculation results with laboratory testing.

Preprocessing Deflection Data

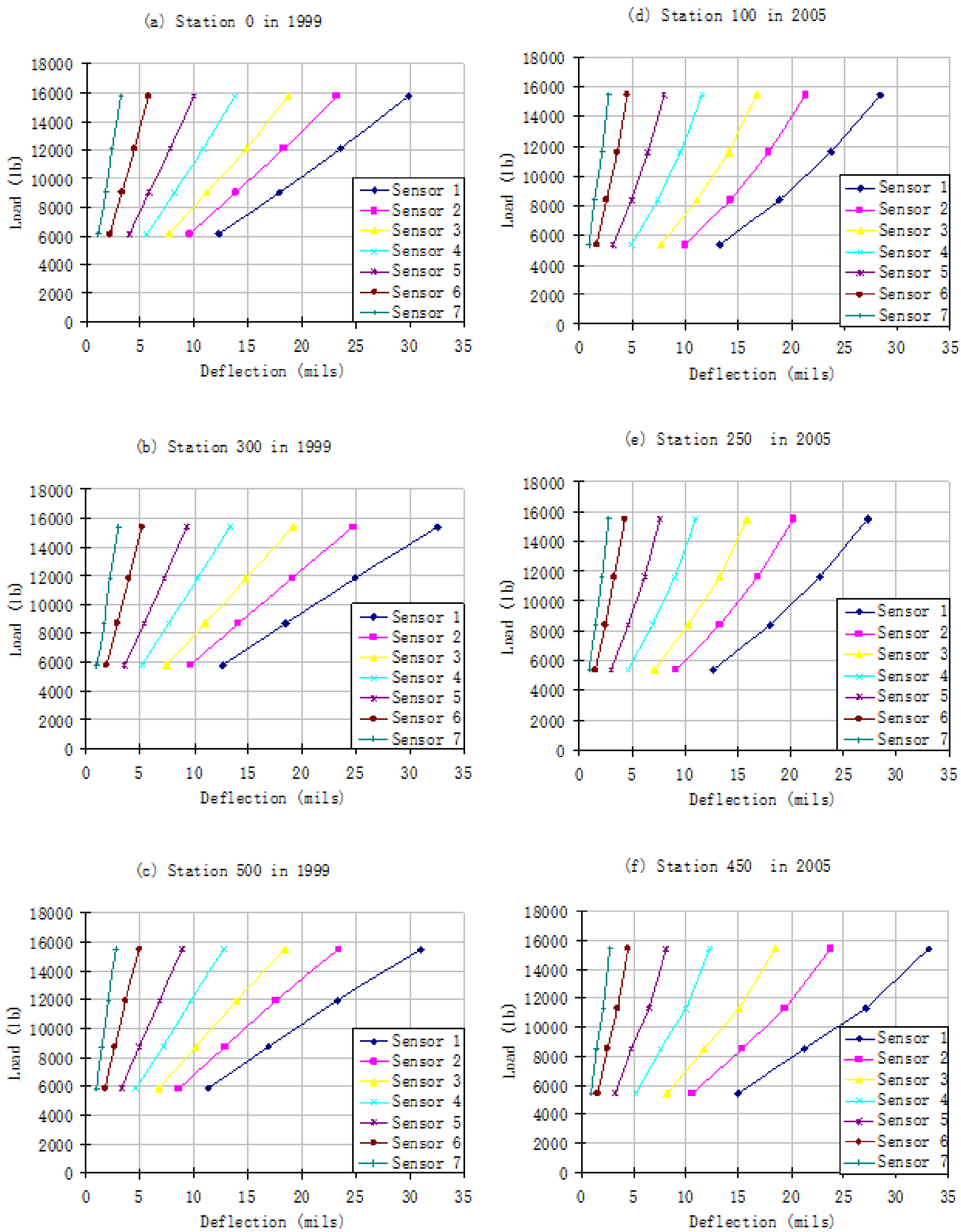
In addition to general data checks (such as non-decreasing data, zeroes, etc.), FWD deflection data were checked for linearity. Figure 6 compares the load versus sensor deflection for a few selected locations within the project. The measurements show slight stress hardening in 2005 as opposed to very slight stress softening behavior in 1999. This may reflect the effect of traffic compaction on unbound layers. Nevertheless, the data were deemed to be acceptable for linear analysis. Depth to bedrock calculations indicated no rigid layer was likely within 7.6 m (25 ft) of the surface.

Backcalculation Analysis

Backcalculation of the test section 30-0113 outer wheelpath deflection data was performed using linear, static layered elastic analysis, as discussed in chapter 4, volume I of this report. Three-layered elastic backcalculation programs were used for this analysis: (1) MODTAG, (2) MICHBACK©, and (3) EVERCALC© to look into the effect of different inverse routines on backcalculated parameters and ultimately on rehabilitation design. In addition, the following layer combinations were used to determine the most realistic design inputs for the MEPDG software:

- Three-layer system (HMA, base, and infinite subgrade).
- Four-layer system (HMA, base, top 2 ft (0.6 m) of compacted subgrade, and infinite subgrade).
- Three-layer system with bedrock at fixed depth (MODTAG analysis).
- Four-layer system with bedrock at fixed depth (MODTAG analysis).

The seed, minimum, and maximum values for layer moduli along with assumed Poisson's ratio values are summarized in table 9.



1 lb = 0.454 kg.
1 mil = 0.0254 mm.

Figure 6. Graphs. Comparison of sample plots of FWD load versus sensor deflection.

Table 9. Input layer modulus and Poisson's ratio values for backcalculation analysis.

Layer	Thickness (inches)	Seed Modulus (psi)	Minimum Modulus, (psi)	Maximum Modulus, (psi)	Poisson's Ratio
HMA	4.3[a]	300,000	50,000	2,500,000	0.35
Base	8.4[a]	30,000	5,000	150,000	0.3
Top 2 ft subgrade	24	15,000	3,000	100,000	0.4
Subgrade	Infinite	7,500	3,000	100,000	0.4

[a]Average thickness determined from LTPP Program inventory data.
1 psi = 6.89 kPa.
1 inch = 25.4 mm.

The backcalculation was performed for each FWD test location using the latest available test data (2005). This allowed investigation of the effect of construction and site variability on backcalculation results. The average layer thicknesses shown in table 9 were used in the MODTAG analysis, while the actual layer thicknesses for each FWD test location, summarized in table 10, were used in the backcalculation analyses using MICHBACK© and EVERCALC©.

Table 10. Input layer thicknesses for different FWD test locations.

Station (ft)	Asphalt Concrete Thickness (inches)[a]	Base Thickness (inches)[a]
0	4.2	9.3
50	4.2	8.1
100	4.2	9.5
150	4.4	8.2
200	5.0	7.5
250	4.0	7.1
300	4.2	8.4
350	4.4	8.8
400	4.2	8.6
450	4.2	8.2
500	4.1	8.4

[a]Obtained from LTPP Program inventory data.
1 inch = 25.4 mm.

Backcalculation Results

Backcalculation results for the various layer models and programs are summarized in table 11 through table 13. Figure 7 illustrates the backcalculation results from MODTAG for the various stations and load levels using several layer combinations, as described previously. The HMA layer moduli were not affected much by the layer combinations for this project (all results are within approximately a 2-percent difference). Introducing a 0.6-m (2-ft) top subgrade layer to the semi-infinite subgrade model led to a lower compacted subgrade modulus value (approximately 10 percent), and a higher base layer value (approximately 15 percent). However, the base layer results tended to be lower than expected for both models.

Table 11. Summary statistics of backcalculation results for three-layer system with semi-infinite subgrade (modulus, psi).

Backcalculation Tool	Statistic	HMA	Base	Subgrade	Error (percent)
MICHBACK©	Mean	595,965	12,181	25,592	1.86
	Standard deviation	88,487	3,760	1,339	0.84
	COV	0.15	0.31	0.05	0.45
	N	40	40	40	—
EVERCALC©	Mean	598,400	12,100	25,600	1.89
	Standard deviation	87,300	3,700	1,400	0.79
	COV	0.15	0.31	0.05	—
	N	40	40	40	40
MODTAG	Mean	608,472	16,727	21,461	9.10
	Standard deviation	133,362	5,595	1,622	4.40
	COV	0.22	0.33	0.08	0.48
	N	36	36	36	—

1 psi = 6.89 kPa.
— Indicates not applicable.
COV = Coefficient of variation.
N = Number of samples.

Table 12. Summary statistics of backcalculation results for four-layer system with semi-infinite subgrade (modulus, psi).

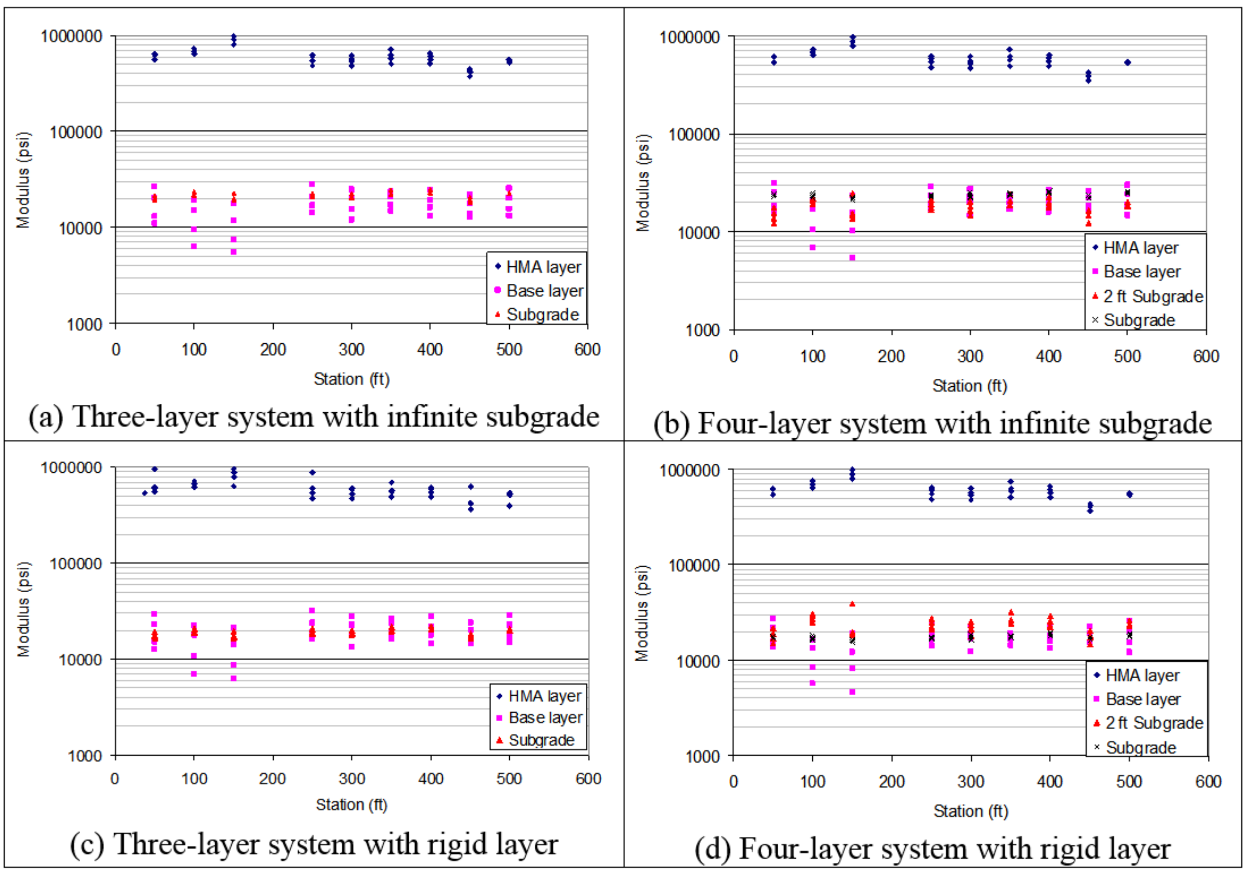
Backcalculation Tool	Statistic	HMA	Base	Top 2-ft Subgrade	Subgrade	Error (percent)
EVERCALC©	Mean	732,278	15,072	21,839	25,272	1.28
	Standard deviation	66,300	2,400	149,400	1,215	0.71
	COV	0.21	0.34	1.05	0.05	0.56
	N	36	36	36	36	—
MODTAG	Mean	591,139	19,652	17,961	23,706	4.31
	Standard deviation	135,146	6,017	3,077	1,188	1.64
	COV	0.23	0.31	0.17	0.05	0.38
	N	36	36	36	36	—

1 psi = 6.89 kPa.
— Indicates not applicable.

Table 13. Summary statistics of MODTAG backcalculation results with rigid layer (Modulus, psi).

Pavement System	Statistic	HMA	Base	Subgrade	Error (percent)
Three-layer system	Mean	592,611	19,051	19,081	6.76
	Standard deviation	132,024	6,292	1,486	1.47
	COV	0.22	0.33	0.08	0.22
	N	36	36	36	—
Four-layer system	Mean	608,083	16,493	23,272[a]	17,439
	Standard deviation	135,993	5,178	5,022[a]	892
	COV	0.22	0.31	0.22[a]	0.05
	N	36	36	36[a]	—

[a]Top 2-ft (610 mm) subgrade.
1 psi = 6.89 kPa.
— Indicates not applicable.



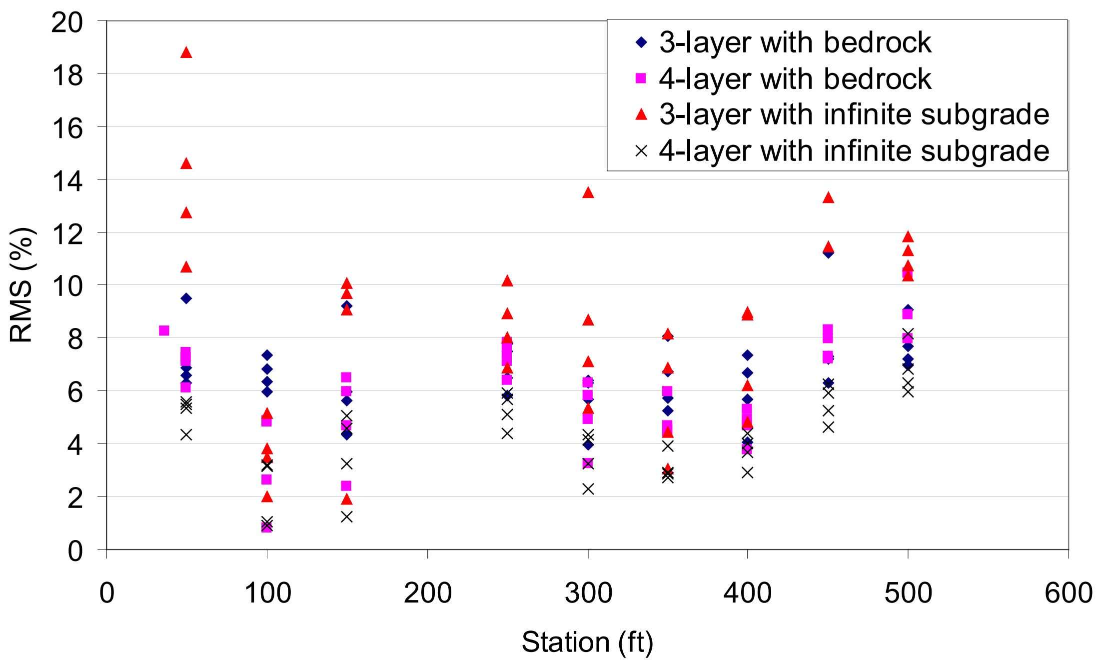
(a) Three-layer system with infinite subgrade
(b) Four-layer system with infinite subgrade
(c) Three-layer system with rigid layer
(d) Four-layer system with rigid layer

1 psi = 6.89 kPa.
1 ft = 0.305 m.

Figure 7. Graphs. Comparison of backcalculated layer moduli (MODTAG) for different layer combinations and load levels along the section.

As shown in table 13, introducing a rigid layer at a fixed depth had some effect on the base and subgrade moduli for the three- and four-layer system, with a more significant effect on the subgrade and base layer moduli for the four-layer system (35- and 19-percent decrease, respectively).

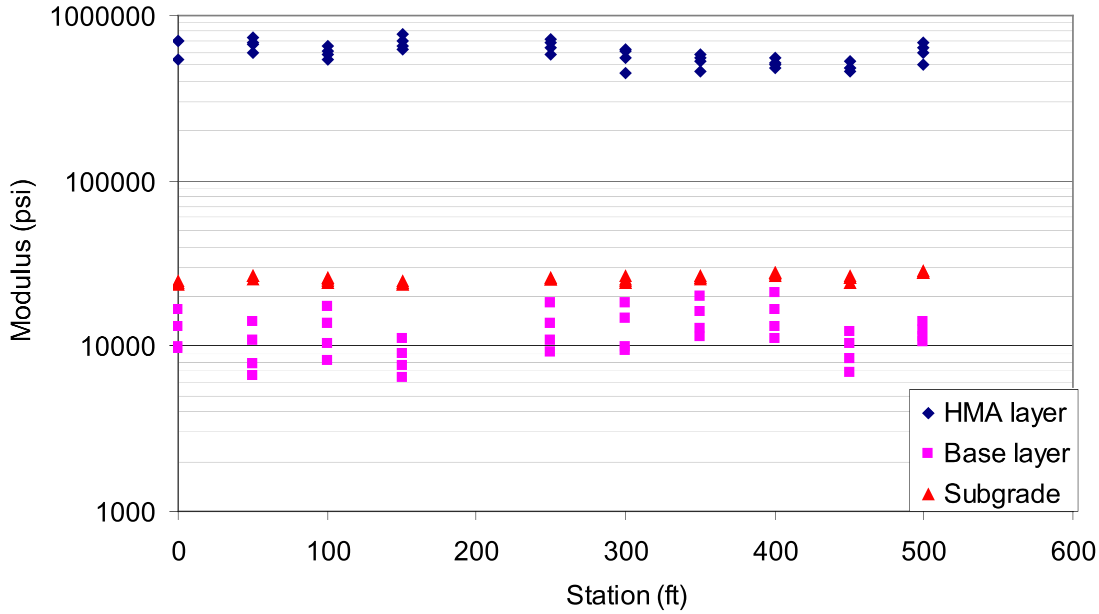
Figure 8 shows the root mean square (RMS) values from MODTAG for the deflection basins at each station along the section using the different layer combinations. The RMS values were observed to be very high (much greater than 2 to 3 percent) and would typically be considered unacceptable for design purposes. The four-layer system with a 2-ft (0.6-m) compacted subgrade layer on top of an infinite halfspace gave the lower RMS values among the four-layer combinations considered. However, the top subgrade modulus was less than the lower subgrade modulus. This may suggest a compensating layer effect, but similar trends were observed when a rigid layer was and was not included in the analysis. It is possible the upper layer was not compacted as well as the lower layer.



1 ft = 0.305 m.

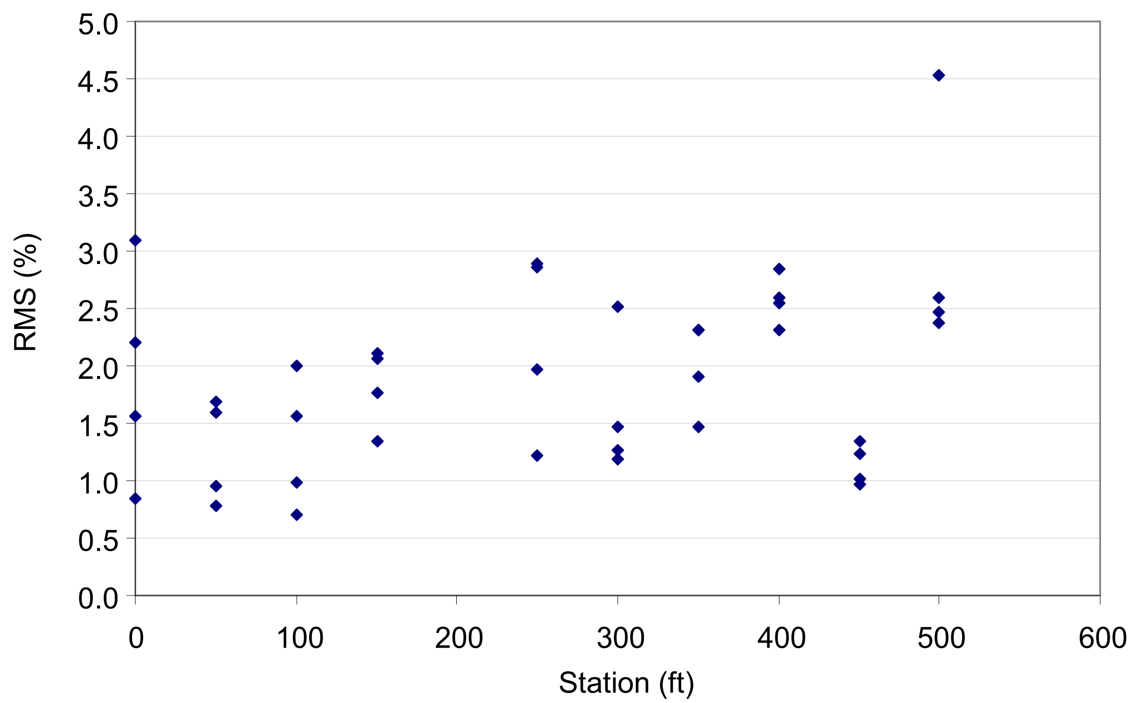
Figure 8. Graph. RMS values (MODTAG) along the section for different layer combinations and load levels.

Figure 9 through figure 12 show the backcalculation results and corresponding RMS values obtained from EVERCALC© and MICHBACK©, respectively, for the various stations and load levels using a three-layer system. It is noted that the two programs produced very similar results, within approximately 1 percent. They also led to lower RMS values (mostly below 3 percent). However, the subgrade modulus was consistently higher than the base modulus (generally about twice as much).



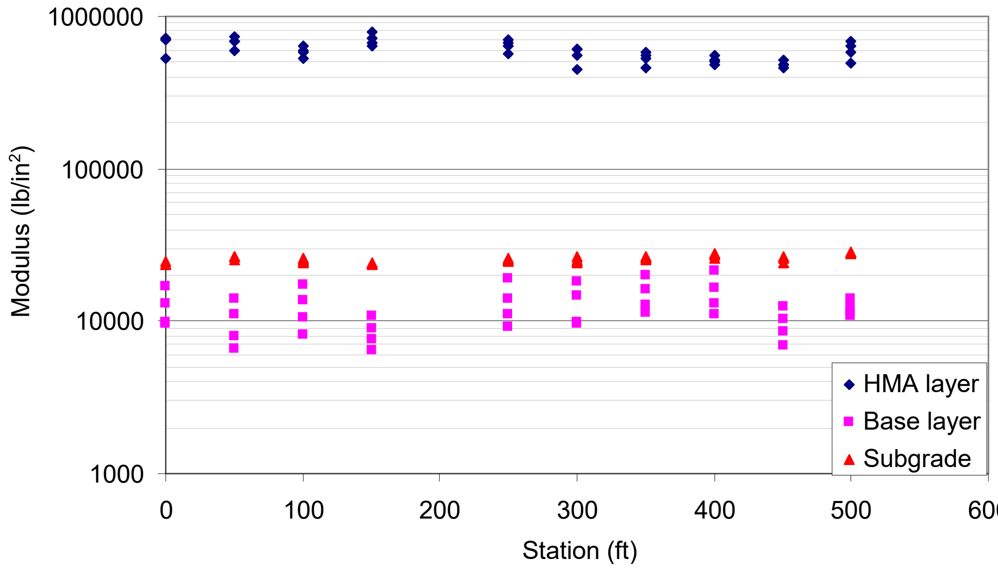
1 psi = 6.89 kPa.
1 ft = 0.305 m.

Figure 9. Graph. EVERCALC© backcalculated layer moduli for a three-layer system and different load levels along the section.



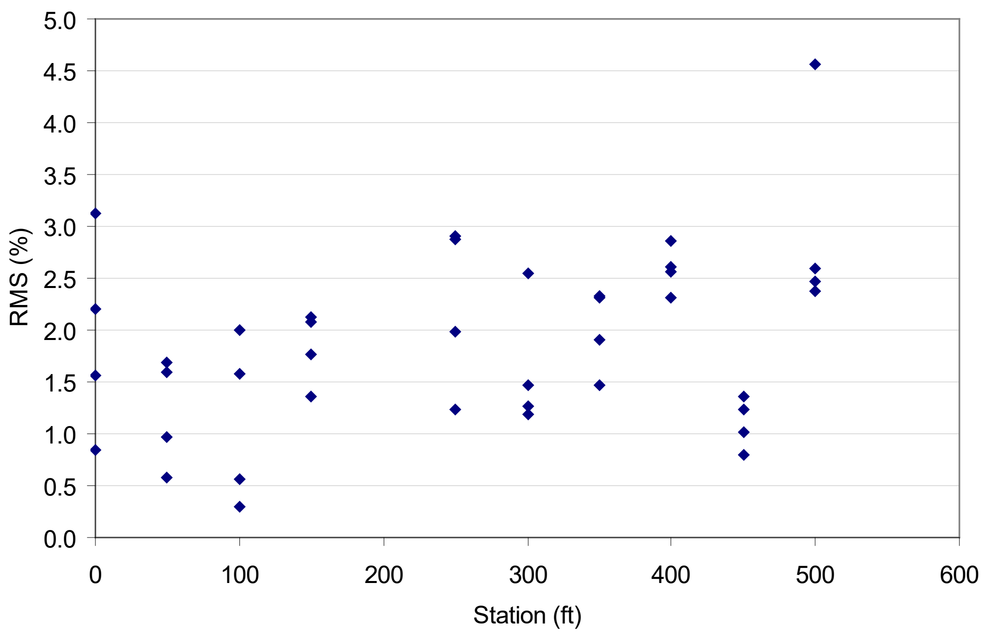
1 psi = 6.89 kPa.
1 ft = 0.305 m.

Figure 10. Graph. EVERCALC© RMS values for a three-layer system and different load levels along the section.



1 psi = 6.89 kPa.
1 ft = 0.305 m.

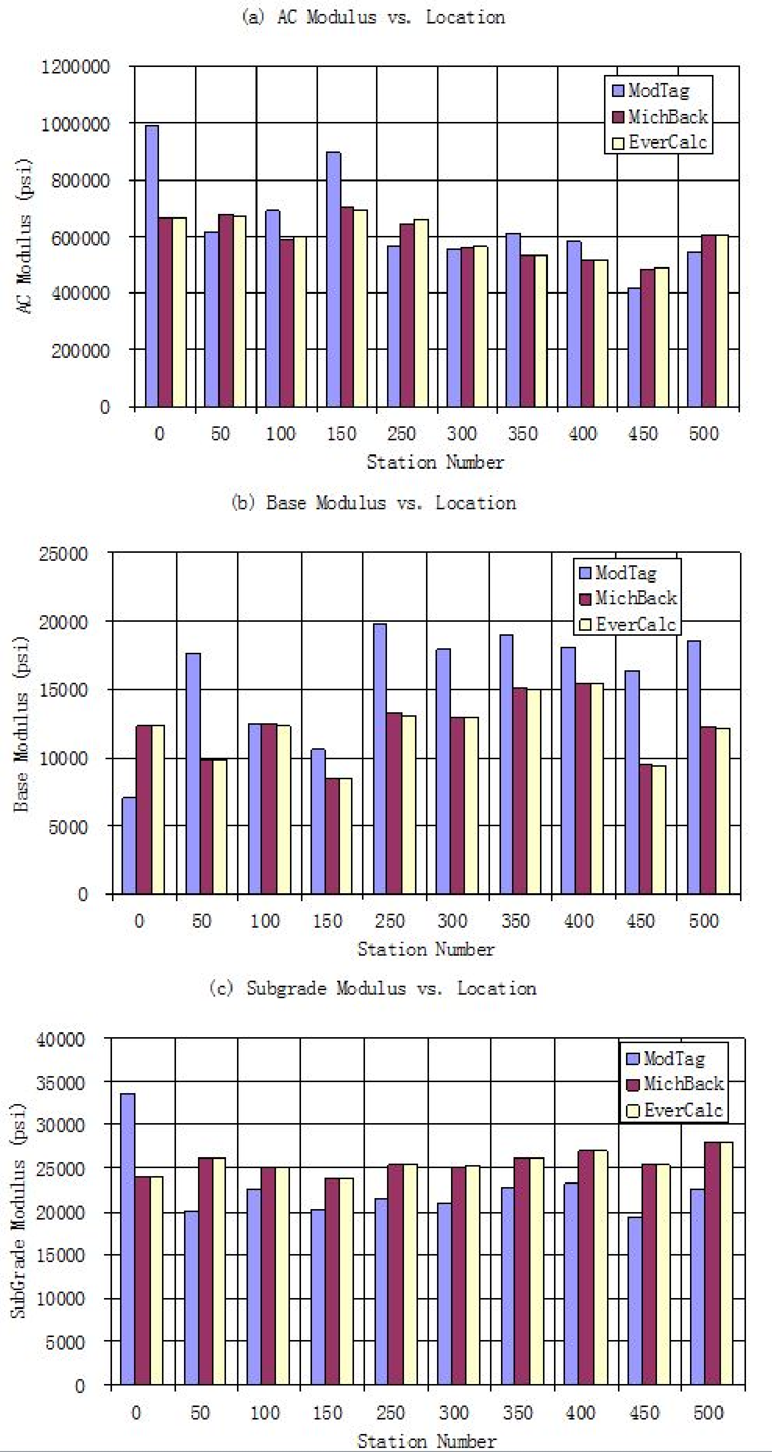
Figure 11. Graph. MICHBACK© backcalculated layer moduli for a three-layer system and different load levels along the section.



1 psi = 6.89 kPa.
1 ft = 0.305 m.

Figure 12. Graph. MICHBACK© RMS values for a three-layer system and different load levels along the section.

Figure 13 summarizes the backcalculated layer moduli (averaged over different load levels) for a three-layer system obtained from the three different programs. The results from MICHBACK© and EVERCALC© were essentially the same for each layer, while those from MODTAG generally differed from them. This could be in part owing to the difference in layer thickness used (average versus location-specific). The base modulus was the most variable along the section, followed by the HMA modulus.



1 psi = 6.89 kPa.

Figure 13. Graphs. Summary of backcalculated results for a three-layer system (average of load levels).

Figure 14 summarizes the backcalculated layer moduli from MODTAG and EVERCALC© for a four-layer system (average of load levels). The MICHBACK© program was unable to converge for this case. The variability in modulus was higher for this case. Also, the compacted subgrade modulus was still higher than the base layer modulus.

Backcalculation Modeling Issues and Recommendations

The RMS values obtained from the three backcalculation programs were generally very high, especially for the MODTAG program. However, the backcalculated layer moduli agreed reasonably well if they were averaged across FWD test locations. Some locations gave unreasonable backcalculation results, the results of which should be excluded from the analysis. The base layer modulus was consistently lower than the subgrade modulus even when using a four-layer system, which included a 2-ft. (0.6-m) compacted subgrade layer on top of the semi-infinite subgrade. The use of correction factors can force the moduli to align along the expected trend of having the subgrade modulus lower than the base modulus. This is addressed in the next section.

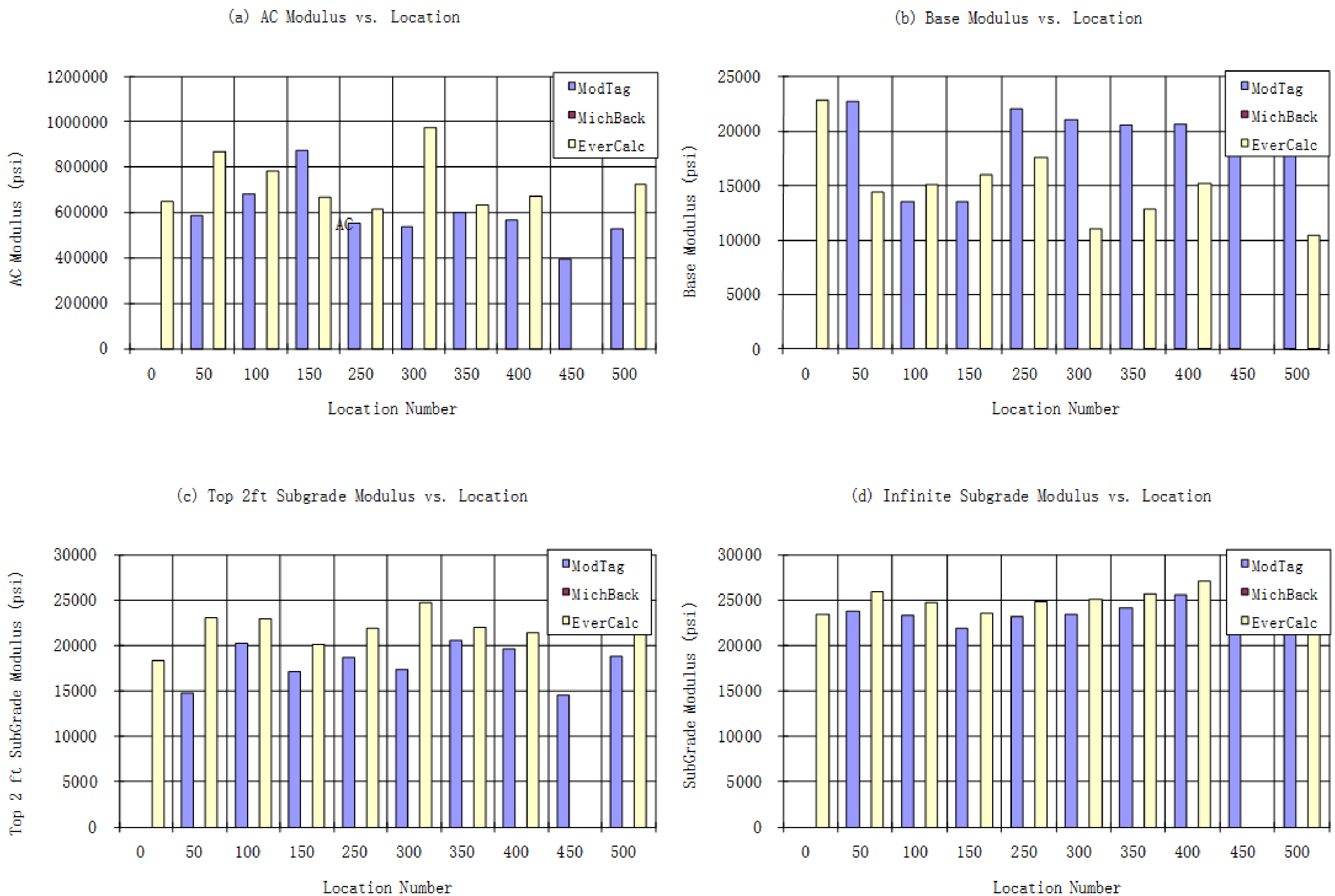
Comparison of Backcalculation and Laboratory Testing Results

To assist in evaluating which layer characteristics were appropriate to use in the MEPDG software, the results of the backcalculation were compared with results obtained from laboratory testing conducted as part of the LTPP Program. While it was outside of the scope of this study to develop new correlations or conversion factors between the two, it was still beneficial to evaluate these relationships and how they may influence MEPDG input selection.

Unbound Materials

The first general argument when comparing field with laboratory results of unbound materials is that the stress state of the material is different for the two conditions. Therefore, correction factors are typically applied to unbound layers. The guidance in the MEPDG, as of the time of this report, was to use the previously established coefficients (summarized in table 3.6.8 of the MEPDG) to adjust backcalculated layer moduli for use in design.(1) These included a factor of 0.35 for subgrade material below flexible pavement with a granular base layer and 0.62 for a granular base below a flexible surface or base layer.

To compare laboratory results to those obtained from FWD testing (or vice versa), the resilient modulus was estimated for the stress conditions at the time of FWD testing. The stress conditions accounted for the overburden pressure of the pavement and the stress due to loading. Load stresses were determined using WinJULEA, the load applied during deflection testing, and initial backcalculated layer properties. Overburden stresses were estimated using layer densities, moisture contents, thicknesses, and at-rest earth pressure coefficients (K_o). A K_o value of 0.5 was used to remain consistent with MEPDG documentation.



1 psi = 6.89 kPa.

Figure 14. Graphs. Summary of backcalculated results for a four-layer system (average of load levels).

To compare the resilient modulus at the stress conditions of FWD testing (assuming a 9,000-lb (4,086-kg) load), the constitutive model in figure 15 (contained in the MEPDG) was used.[6]

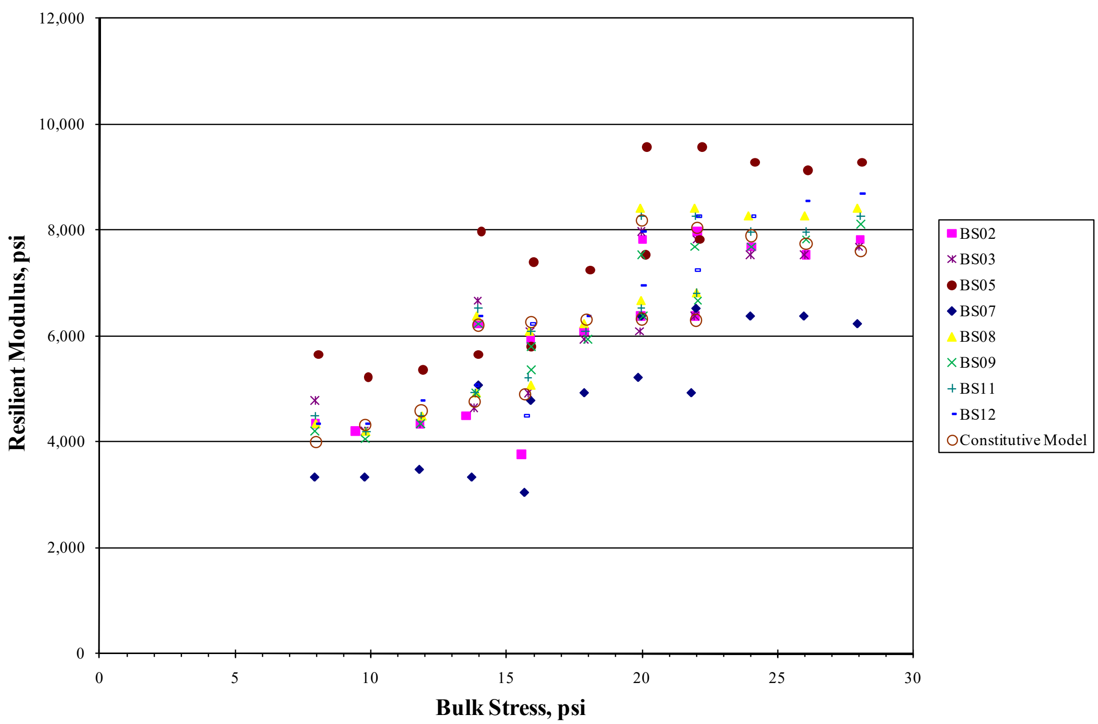
$$M_R = k_1 \times p_a \times \left(\frac{\theta}{p_a}\right)^{k_2} \times \left(\frac{\tau_{Oct}}{p_a} + 1\right)^{k_3}$$

Figure 15. Equation. Constitutive model for determining resilient modulus.

Where:

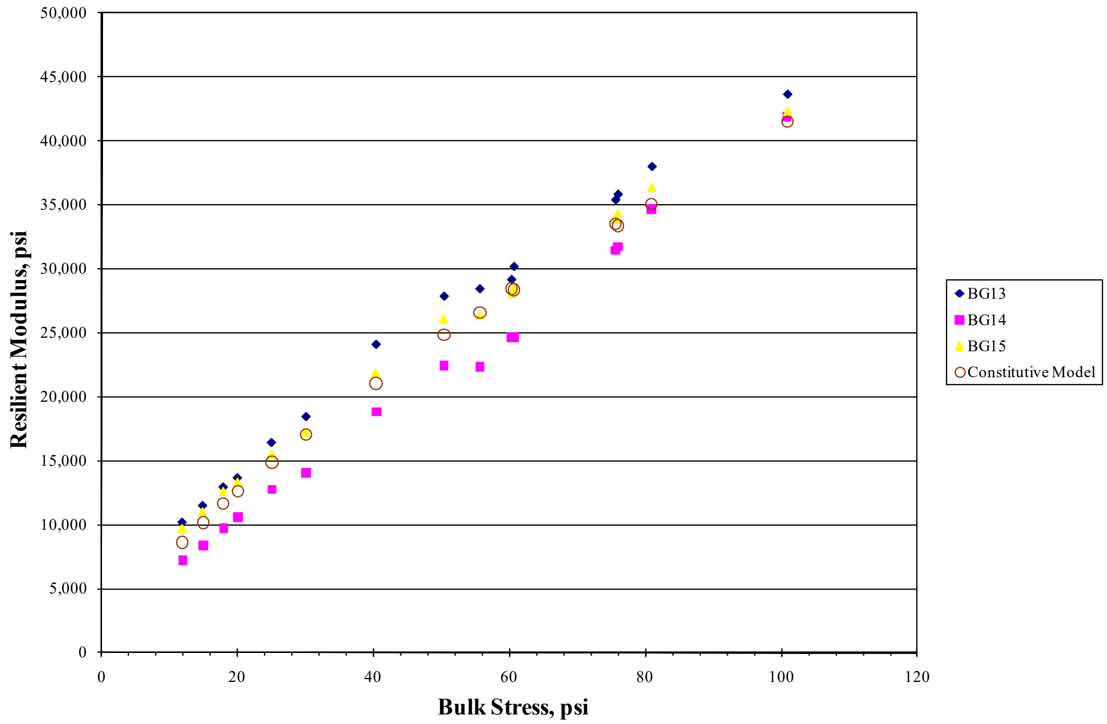
M_R = Resilient modulus, kPa (psi).
θ = Bulk stress, kPa (psi).
τ_{Oct} = Octahedral shear stress, kPa (psi).
p_a = Atmospheric pressure, kPa (psi).
k_1, k_2, k_3 = Regression constants.

Developed constitutive models are illustrated in figure 16 and figure 17 for the subgrade and base materials, respectively. The estimated subgrade resilient modulus based on laboratory testing and using the constitutive model at the stress conditions for a 9,000-lb (4,086-kg) FWD load was 3,900 psi (26,900 kPa). The resulting laboratory-to-field value ratios ranged from approximately 0.15 to 0.22. For this project, this range of values was lower than the typical correction factor (0.35) provided in the MEPDG.[1] Consequently, the equivalent modulus was much lower than the default value used in the MEPDG (20,500 psi (141,340 kPa) for AASHTO A-2-6 subgrade).



1 psi = 6.89 kPa.

Figure 16. Graph. Estimated constitutive model for subgrade resilient modulus.



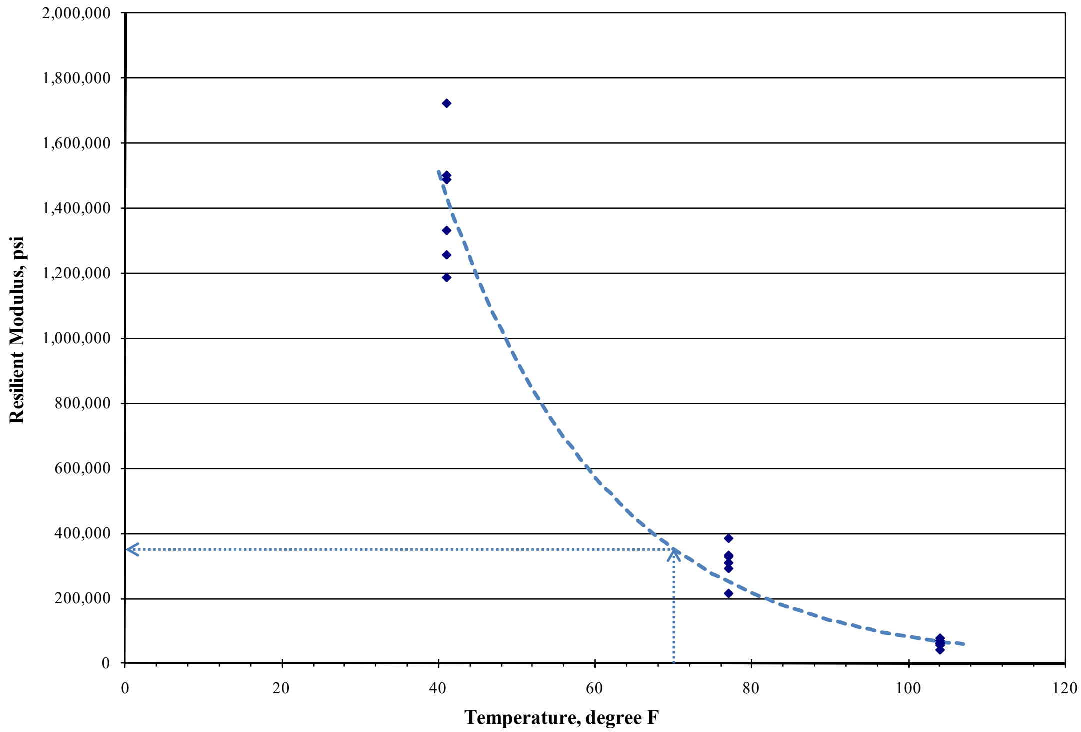
1 psi = 6.89 kPa.

Figure 17. Graph. Estimated constitutive model for base resilient modulus.

The estimated base resilient modulus based on laboratory testing and using the constitutive model at the stress conditions for a 9,000-lb (4,086-kg) FWD load was 13,200 psi (91,000 kPa). The resulting laboratory-to-field value ratios range from approximately 0.67 to 1.09. This range was greater than the typical correction factor (0.62) in this case. The equivalent modulus value was much lower than the default value used in the MEPDG—38,000 psi (262,000 kPa) for AASHTO A-1-b base).

Bound Materials

The laboratory results for the HMA layer modulus for several temperatures are shown in figure 18. The pavement temperature at the time of the FWD testing was recorded as 70 °F (21 °C) based on borehole temperature data. Using figure 18, a corresponding laboratory HMA modulus of about 350,000 psi (2,413,000 kPa) was obtained, compared with average backcalculated moduli ranging from approximately 591,100 psi (4,075,000 kPa) to 732,300 psi (5,049,000 kPa).



1 psi = 6.89 kPa.
°F = 1.8 × °C + 32.

Figure 18. Graph. Approximate HMA resilient modulus based on LTPP Program laboratory testing.

An additional consideration in comparing laboratory and deflection testing is the frequency of loading: laboratory resilient modulus testing was conducted with a 0.1-s load pulse, whereas FWD testing was conducted with a 0.03-s load pulse (approximately). Although dynamic modulus testing data were not available, reviewing the approach for shifting the HMA modulus due to frequency and temperature was worthwhile. Based on the available HMA properties, recommended MEPDG default values, and predictive equations, the estimated dynamic modulus (E^*) master curve using the MEPDG software is illustrated in figure 19.

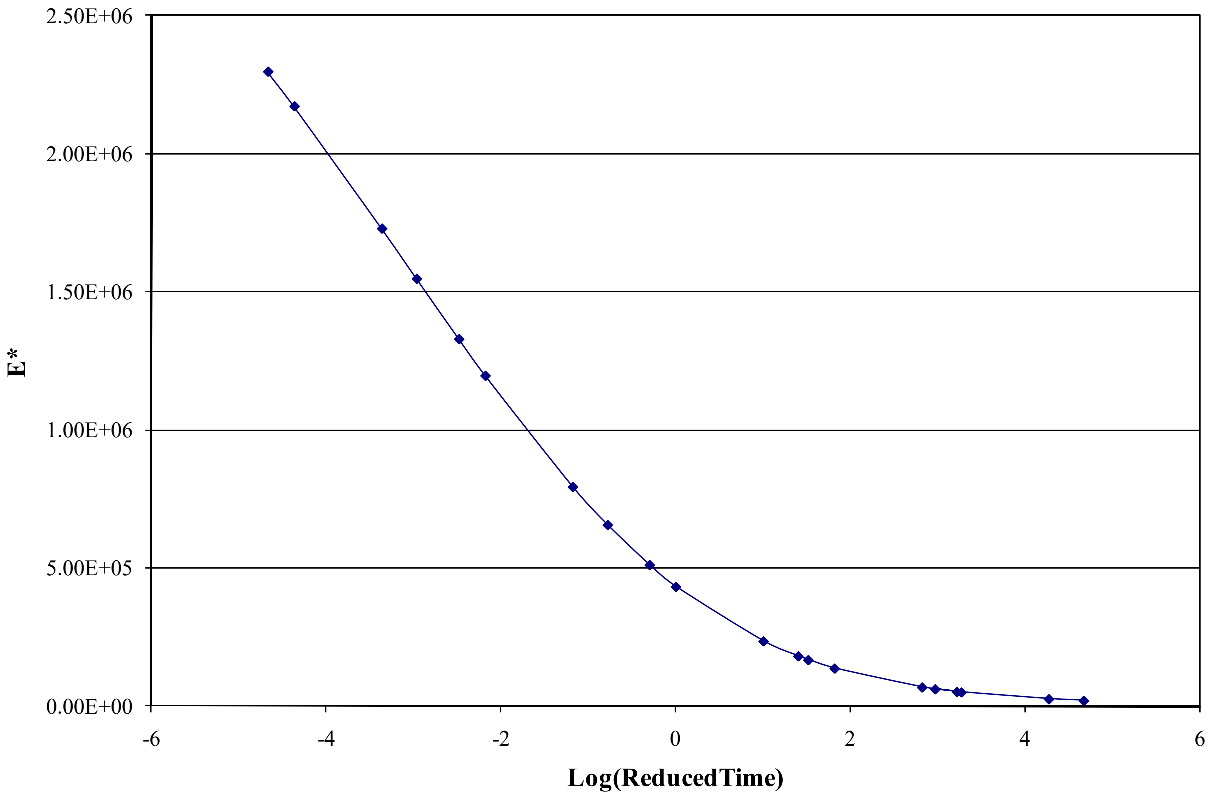


Figure 19. Graph. Illustration of HMA dynamic modulus master curve from MEPDG design software.

Based on the results of the predictive equations incorporated in the MEPDG and case study inputs, a dynamic modulus of 924,000 psi (6,371,000 kPa) was estimated for the conditions during FWD testing (approximately 70 °F (21.1 °C) and 0.03-s loading time). A dynamic modulus of 730,400 psi (5,036,000 kPa) was estimated at 70 °F (21.1 °C) and 0.1-s loading time (resilient modulus testing conditions). In this case, it appears the predictive model results in moduli greater than the resilient modulus testing results and possibly slightly high for HMA material.

REHABILITATION DESIGN RESULTS

An HMA overlay rehabilitation design was selected for this case study to assess design differences between backcalculated inputs and laboratory-based inputs. It was assumed that 0.75 inches (19.1 mm) of the existing HMA overlay would be milled and that additional pre-overlay repairs (patching and crack sealing) would be performed as necessary. The overall design level selected for this rehabilitation (HMA over HMA) was level 1. The required layer inputs used in the MEPDG depend on the overall design level selected for design. When level 1 is selected, all unbound layer inputs need to be level 1, but the HMA layers can be levels 1 through 3. When the overall design levels 2 or 3 are selected, unbound layer inputs can be level 2 or 3, and the HMA layers can be level 1 through 3. Level 3 was used here for the HMA layers.

Design Criteria and General Inputs

The MEPDG calls for determining the HMA overlay thickness by trial and error. A trial thickness is assumed, and the program is executed to predict the future performance in terms of the different performance measures (cracking, rutting, International Roughness Index (IRI), etc.). For a given desired rehabilitation design life, the level of distress at the design life should not exceed a prescribed limiting value, summarized in table 14 for this case study. A reliability level of 90 percent was selected as well as a 20-year design life.

Table 14. Summary of analysis parameter inputs.

Variable	Value
Initial IRI (inches/mi)	63
Terminal IRI (inches/mi)	172
HMA surface down cracking, long cracking (ft/mi)	2,000
HMA bottom up cracking, alligator cracking (percent)	25
HMA thermal fatigue (ft/mi)	1,000
Chemically stabilized layer fatigue fracture (percent)	N/A
Permanent deformation—total pavement (inches)	0.75
Permanent deformation—HMA only (inches)	0.25

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.
1 ft/mi = 0.19 m/km.
N/A = Not applicable.

For new HMA materials, level 1 analysis requires conducting *E** (complex modulus) laboratory testing (ASTM D3496) at loading frequencies and temperatures of interest for the given mixture.[7] Level 2 analysis does not require *E** laboratory testing; instead, the user can input asphalt mix properties (gradation parameters) and laboratory binder test data (from *G** testing or other conventional binder tests). The MEPDG software calculates the corresponding asphalt viscosity values; it then uses the modified Witczak equation to predict *E** and develops the master curve for the HMA mixture. The same procedure is used for level 3 analysis to estimate the HMA dynamic modulus except that no laboratory test data are required for the binder, and typical values for the selected binder grade are used. The properties used for the new HMA overlay are summarized in table 15 and table 16.

Table 15. Summary of new HMA material properties.

Variable	Value
Asphalt grading	PG 70-28
Asphalt content (percent)	12.5
Air voids (percent)	4.0
Total unit weight (lb/ft^3)	148

1 lb/ft^3 = 0.0160 g/cm^3

Table 16. Summary of new HMA aggregate sieve analysis.

Sieve Size	Average Percent Passing
$^3/_4$ inch	100
$^3/_8$ inch	78
No. 4	50
No. 200	6.5

1 inch = 25.4 mm.

Incorporation of Backcalculation Results With the MEPDG Software

Several analyses were executed with the MEPDG design software (summarized in table 17) to evaluate the influence of the unbound and bound layer inputs, as discussed in the following sections. Version 1.003 of the software was used, as well as the nationally calibrated performance models.

Table 17. Input base and subgrade moduli for five MEPDG runs.

Analysis Run Number	Subgrade Modulus (psi)	Base Modulus (psi)	HMA Modulus (psi)
1[a]	14,500	38,000	N/A
2[b]	21,500	16,700	608,500
3[c]	7,500	10,400	608,500
4[d]	9,000	7,500	597,000
5[e]	3,900	13,200	350,000
6[f]	6,300 (24 inches) 8,300 (infinite)	12,200	591,000

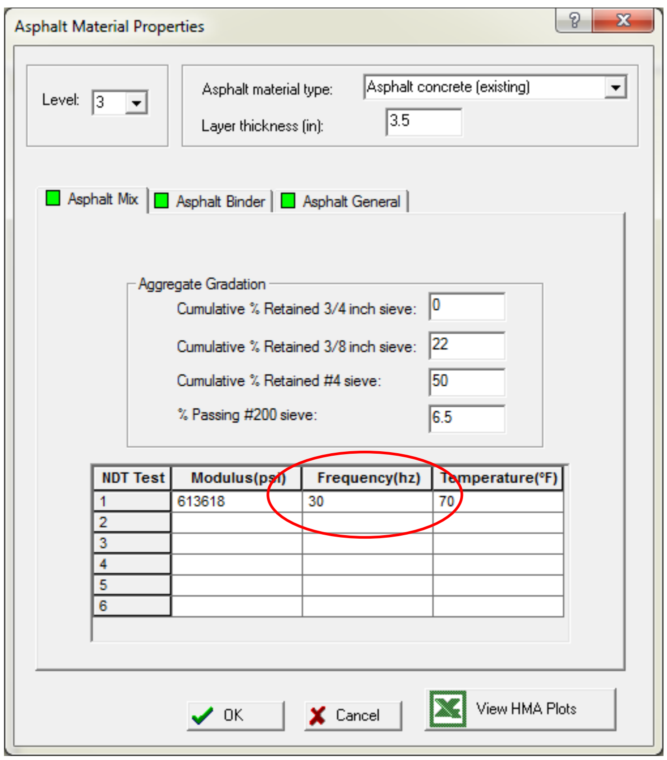
[a]Analysis 1: Default layer properties; HMA modulus determined internally based on material properties.
[b]Analysis 2: Uncorrected backcalculation values based on MODTAG results for three-layer system.
[c]Analysis 3: Corrected backcalculation values based on MODTAG results for three-layer system.
[d]Analysis 4: Corrected backcalculation values based on EVERCALC© and MICHBACK© for three-layer system.
[e]Analysis 5: Based on LTPP Program laboratory testing results.
[f]Analysis 6: Corrected backcalculation values based on MODTAG results for four-layer system.
1 psi = 6.89 kPa.
1 inch = 25.4 mm.
N/A = Not applicable.

The matrix of design analyses incorporated the use of default layer properties and internally calculated HMA moduli based on mixture properties (analysis 1), backcalculation-based values (analyses 2–4), and LTPP Program laboratory testing results (analysis 5).

HMA Layers

For rehabilitation design, the determination of the existing HMA layer dynamic modulus follows the same general concepts described for new HMA, except that the software allows a modified procedure to account for damage incurred in the HMA layer during the life of the existing pavement. Therefore, the procedure determines a field-damaged dynamic modulus master curve, as discussed in chapter 4, volume I, of this report. When the overall design hierarchy is level 1, FWD data can be used at material layer levels 1 through 3. For the overall design levels 2 and 3, FWD data are not used, regardless of the layer input level.

Level 1 analysis requires entering the backcalculated HMA modulus with the corresponding temperature at the time of testing and an equivalent frequency for the FWD pulse, as shown in figure 20. As previously indicated, the pavement temperature at the time of the FWD testing was 70 °F (21 °C) based on borehole temperature data. Based on the HMA over HMA rehabilitation example in the MEPDG appendix, an equivalent frequency of 30 Hz was used for FWD-based inputs. This was calculated as 1/(FWD pulse duration) = 1/0.033 s = 30 Hz. Although this formula technically does not yield frequency (based on sinusoidal wave), it is compatible with the equivalent frequency used for calculating E^* in the MEPDG, which is calculated as equivalent frequency = 1/equivalent time. The equivalent time is calculated using the width of the tire, an equivalent depth based on the Odemark concept of equivalent thickness and a 1:1 stress dissipation slope.



1 inch =25.4 mm.

Figure 20. Screen Capture. HMA input based on FWD testing and backcalculation for overall design level 1 and layer input level 3.

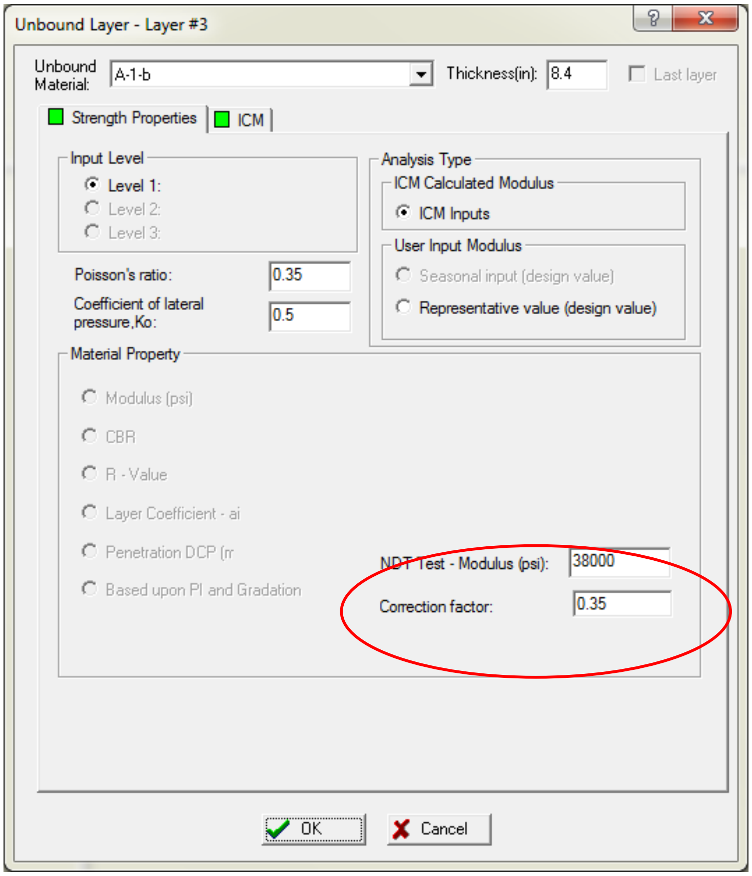
Unbound Layers

For unbound materials (and bedrock, if used), only level 1 analysis calls for FWD testing in rehabilitation and reconstruction designs. The resilient modulus, M_R, for each unbound layer (including the subgrade) can be either determined in the laboratory using cyclic triaxial tests or backcalculated using standard backcalculation procedures. Although the MEPDG does allow use of the generalized nonlinear, stress-dependent model in the design procedure, this approach is not recommended at this time because the performance models in the software have not been calibrated for nonlinear conditions. Therefore, the option of backcalculating the k_1, k_2, and k_3 parameters in the nonlinear model is not discussed in this report. Consequently, the discussion includes only the backcalculation and use of "effective" moduli that would account for any stress-sensitivity, cracks or any other anomalies in any layer within the existing pavement.

For level 2 analysis, correlations with strength test data were used. For level 3, the MEDPG lists typical modulus values based on soil classification but warns that they are very approximate and strongly recommends some form of testing, especially using FWD testing and backcalculation (level 1). The guide notes that the reason for caution is related to using the wrong assumptions: either a fairly strong subgrade material may be erroneously assumed to be semi-infinite while it may actually be less than 3 ft (1 m) thick (e.g., as part of an embankment), or conversely, a weak subgrade soil may be assumed to be semi-infinite while it may, in reality, be overlying a stronger soil or bedrock.

The MEDPG also notes that for granular materials, moduli values that matched FWD backcalculated results were 50 to 70 percent higher than the typical laboratory-tested values, while for subgrade soils, they were two to three times the typical laboratory determined values. The guidance in the MEPDG as of the time of this report is to use the previously established coefficients (summarized in table 3.6.8 of the MEPDG) to adjust backcalculated layer moduli for use in design.(1)

The MEPDG allows for entering the backcalculated base and subgrade layer moduli directly, together with a correction factor to account for the difference between laboratory-determined and backcalculated moduli, as shown in figure 21. However, the version of the MEPDG used in the analysis (version 1.003) did not allow the base modulus to be changed for the level 1 rehabilitation module (i.e., it reverted to the default values). Therefore, a suitable correction factor had to be used to obtain the desired input value. For example, the default modulus for AASHTO A-1-b material was 38,000 psi (262,000 kPa). To obtain an input of 10,400 psi (71,700 kPa) (16,700 psi × 0.62 (115,100 kPa × 0.62)), a factor of 0.27 (or 0.62 × 0.44) was used. The same was done for the subgrade modulus.



1 psi = 6.89 kPa.
1 inch = 25.4 mm.

Figure 21. Screen Capture. Base layer input for overall design level 1.

Unbound layers can be entered as "Representative value," or the user can select the "Integrated Climatic Model (ICM)" analysis. Because backcalculated values were not available throughout the year for this case study location, the ICM analysis was used. ICM inputs (gradation, Atterberg limits, etc.) were obtained from the LTPP Program database, as summarized earlier.

Evaluation of Design Results

As noted previously, the analyses summarized in table 17 were run with the MEPDG software (version 1.003) to evaluate the influence of varying the backcalculation-based inputs; the nationally calibrated performance models were used. The HMA overlay thickness requirement was determined for each set of inputs, and the change in distress predictions and reliabilities were compared.

The required HMA overlay thickness for all analyses was 3 inches (76 mm), except for analysis 4, which was 3.5 inches (89 mm), to achieve a 90-percent reliability level. The required HMA overlay thickness was controlled primarily by top-down cracking (summarized in table 18 through table 20 and illustrated in figure 22), with other distress predictions showing minimal differences between the analyses.

Analysis 4 had the greatest predicted top-down cracking. While analysis 4 had the weakest base layer modulus, it had one of the higher subgrade moduli. The existing HMA modulus for analysis 4 was also nearly the same as analyses 2 and 3. Even though analysis 5 had the lowest subgrade modulus, the top-down cracking was lower than analyses 2 and 3 (uncorrected and corrected backcalculation-based, respectively). However, the permanent deformation for this analysis increased compared with analysis 3, which was partly attributed to the lower HMA modulus. Analysis 6 (four-layer system) was comparable to the three-layer systems.

Table 18. Summary of design results for 76-mm (3-inch) HMA overlay (analyses 1 and 2).

Pavement Performance Measure	Default Values Analysis 1		Uncorrected Backcalculation-Based Analysis 2	
	Quantity	Reliability	Quantity	Reliability
Terminal IRI (inches/mi)	109.3	98.06	97	99.67
AC surface down cracking (longitudinal cracking) (ft/mi)	5.2	99.91	62.7	91.65
AC bottom-up cracking (alligator cracking) (percent)	0	99.999	0	99.999
AC thermal fracture (transverse cracking) (ft/mi)	1	99.999	1	99.999
Chemically stabilized layer (fatigue fracture)	N/A	N/A	N/A	N/A
Permanent deformation (AC only) (inches)	0.13	99.43	0.08	99.999
Permanent deformation (total pavement) (inches)	0.38	99.999	0.08	99.999

1 inch/mi = 0.0158 m/km.
1 ft/mi = 0.19 m/km.
1 inch = 25.4 mm.
N/A = Not applicable.

Table 19. Summary of design results for 76-mm (3-inch) HMA overlay (analyses 3 and 4).

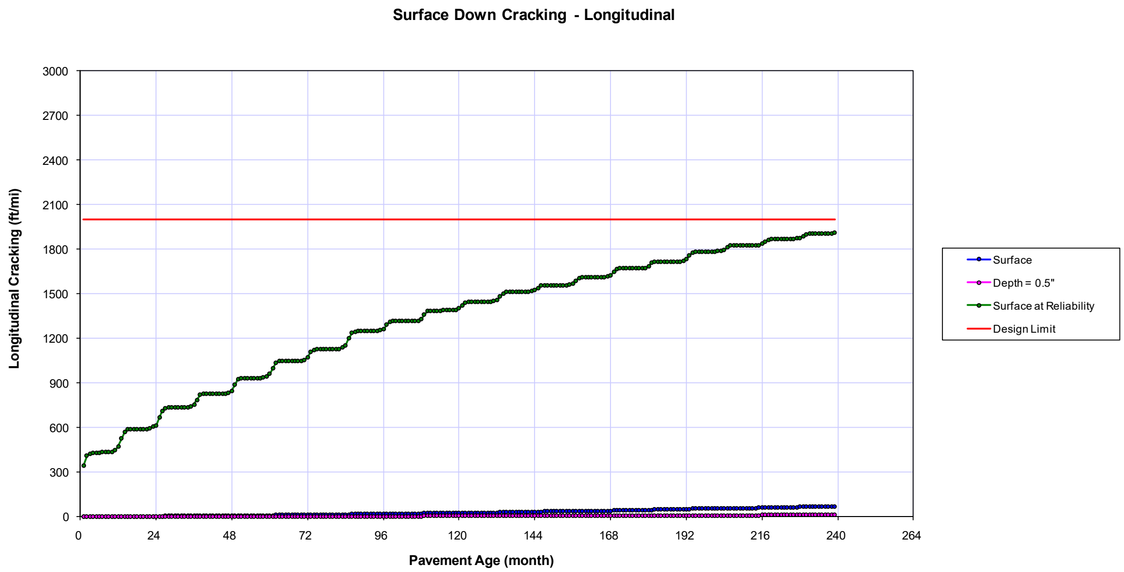
Pavement Performance Measure	Corrected Backcalculation-Based Analysis 3		Corrected Backcalculation-Based Analysis 4	
	Quantity	Reliability	Quantity	Reliability
Terminal IRI (inches/mi)	97	99.67	97.1	99.67
AC surface down cracking (longitudinal cracking) (ft/mi)	69.5	91.01	165	85.5
AC bottom up-cracking (alligator cracking) (percent)	0	99.999	0	99.999
AC thermal fracture (transverse cracking) (ft/mi)	1	99.999	1	99.999
Chemically stabilized layer (fatigue fracture)	N/A	N/A	N/A	N/A
Permanent deformation (AC only) (inches)	0.07	99.999	0.08	99.999
Permanent deformation (total pavement) (inches)	0.08	99.999	0.08	99.999

AC = Asphalt concrete.
N/A = Not applicable.
1 inch/mi = 0.0158 m/km.
1ft/mi = 0.19 m/km.
1 inch = 25.4 mm.

Table 20. Summary of design results for 76-mm (3-inch) HMA overlay (analyses 5 and 6).

Pavement Performance Measure	Laboratory-Based Analysis 5		Corrected Backcalculation-Based Analysis 6	
	Quantity	Reliability	Quantity	Reliability
Terminal IRI (inches/mi)	100.7	99.39	97.9	99.62
AC surface down cracking (longitudinal cracking) (ft/mi)	49.4	93.07	50.4	92.95
AC bottom-up cracking (alligator cracking) (percent)	0.3	99.999	0	99.999
AC thermal fracture (transverse cracking) (ft/mi)	1	99.999	1	99.999
chemically stabilized layer (fatigue fracture)	N/A	N/A	N/A	N/A
Permanent deformation (AC only) (inches)	0.12	99.72	0.08	99.999
Permanent deformation (total pavement) (inches)	0.16	99.999	0.12	99.999

AC = Asphalt concrete.
N/A = Not applicable.
1 inch/mi = 0.0158 m/km.
1ft/mi = 0.19 m/km.
1 inch = 25.4 mm.



1ft/mi = 0.19 m/km.

Figure 22. Graph. Top-down cracking distress prediction from MEPDG design program for analysis 3.

SUMMARY

Three backcalculation programs (MODTAG, MICHBACK©, and EVERCALC©) were used to analyze FWD deflection data from various test locations (stations) within the project. The deflection data showed considerable variability within the project. The RMS values obtained from the three backcalculation programs were also generally very high. It is recommended that one should conduct FWD testing at multiple locations and use the average backcalculated layer moduli. This should provide consistent recommended overlay designs.

For this case study, the MEPDG results indicated that surface-down cracking was critical in rehabilitation design of HMA overlay over existing HMA pavements; although this was minimal even considering a 3-inch (76-mm) HMA overlay was satisfactory for nearly all of the input combinations. Within the ranges identified, the selection of inputs was more critical as one approached lower values for any layer.

In addition, the following procedures are recommended:

- Until ongoing studies are completed, the correction factors developed by Von Quintus and Killingsworth should be applied to the backcalculation results.(8)

- While this procedure needs to be verified, for the time being, an equivalent frequency of 30 Hz should be used. This is calculated as 1/(FWD pulse duration) = 1/0.033 s = 30 Hz. While this formula is technically incorrect, it is compatible with the equivalent frequency used for calculating E^* to be used in MEPDG.

CHAPTER 2. CASE STUDY 2: FLEXIBLE PAVEMENT INCLUDING RUBBLIZED CONCRETE PAVEMENT EVALUATION AND OVERLAY DESIGN

PROJECT OVERVIEW

Project 17-0600, located on I-57 near Pesotum, IL, consisted of 14 test sections. Test section 17-0600 was selected as an HMA surface on a rubblized portland cement concrete (RPCC) layer rehabilitation case study. The original underlying pavement was jointed reinforced concrete pavement (JRCP) with an aggregate base layer. The JRCP was rubblized, and a HMA overlay was placed in 1990. This section is representative of the following selection factors:

- Rubblized concrete pavement.
- Wet-freeze climate zone.
- Rural principal arterial—interstate functional class.
- “Fair” pavement condition
- Fine-grained soil subgrade classification.
- No (or deep) rigid layer.

The underlying concrete pavement was originally constructed in 1964 (construction number 1), with rehabilitation/repair work performed in 1990 and 1997 (construction numbers 2–4, respectively). Based on LTPP Program core data, the average original pavement cross section consisted of 10 inches (254 mm) of portland cement concrete (PCC) (JRCP) and 7 inches (178 mm) of aggregate base. The HMA overlay placed in 1990 consisted of a 1.5-inches (38-mm) HMA surface and a 6.5-inches (165-mm) HMA binder course.

PAVEMENT CONDITION/PERFORMANCE

Pavement condition and performance data can be used to customize (or calibrate) the performance models within the MEPDG software for the specific State using the design procedure.[1] However, the calibration of performance models was beyond the scope of this study. The design inputs after performance model calibration used relatively little distress data, and the design of HMA over RPCC did not use any pavement distress data.

TEST SECTION DATA

The MEPDG design program requires a significant number of inputs, particularly for level 1 analysis. The required design data for the 17-0600 project section were obtained from the LTPP Program’s DataPave database. The required data were not generally complete for any one specific test section within 17-0663; therefore, results for the entire project, and during multiple years, were used to obtain the necessary design inputs. Although the test sections had varying cross sections and maintenance histories, the research team concluded that overall data were sufficient for this study.

Deflection-Testing Data

Deflection data for test section 17-0663 were available from the LTPP Program database for several years of testing, including 1990–1993, 1995, 1997–1998, and 2004. To compare backcalculation results at the time of rehabilitation, deflection data from 1990 were retrieved from the LTPP Program database. Deflection testing was conducted following the LTPP Program protocols.

Equipment

Deflection testing was conducted with two FWD devices. For the 1990–1993, 1995, and 1997–1998 data, a Dynatest® FWD (SN 8002-060) was used; for 2004, a different Dynatest® FWD (SN 8002-130) was used.

Sensor Configuration

Sensors were located at 0, 8, 12, 18, 24, 36, and 60 inches (0, 203.2, 304.8, 457.2, 609.6, 914.4, 1,524 mm) from the center of the load plate for 1990–1993, 1995, 1997–1998 datasets and at 0, 8, 12, 18, 24, 36, 48, 60, and -12 inches (0, 203.2, 304.8, 457.2, 609.6, 914.4, 1,219.2, 1,524, -304.8 mm) from the center of the load plate for data collected in 2004.

Number of Drops and Load Levels

Four load level targets—6,000, 9,000, 12,000, and 16,000 lb (2,724, 4,086, 5,448, and 7,264 kg) with four drops at each load level were performed, and data were recorded. Seating drops were also performed, but data were not recorded.

Test Locations/Lanes and Increments

In 1990, immediately before rubblization (construction number 2), FWD testing was conducted on the JRCP at the mid-lane at mid-panel and in the outer wheelpath at both approach and leave joints (for load transfer). For the year after overlay construction (CN 3), FWD testing was conducted in the outer wheelpath and mid-lane of the flexible pavement at 50-ft (15.25-m) intervals along the length of the project.

Temperature Measurements

Temperature measurements were taken using drilled holes in the pavement at depths 1, 2.2, and 3.3 inches (25.4, 55.88, and 83.82 mm) at prescribed time intervals during deflection testing.

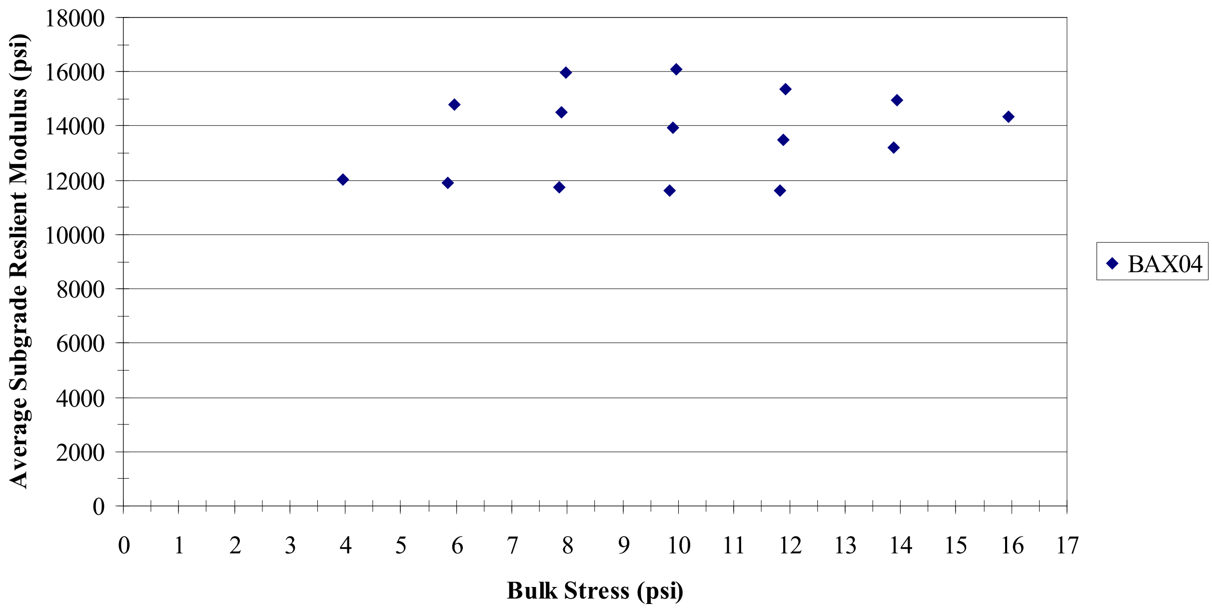
Material Properties Data

This section summarizes the data obtained from LTPP database for test section 17-0600 regarding its subgrade, base, HMA and PCC material properties, traffic, climate, and depth of water table or stiff layer.

Subgrade

Six subgrade samples were retrieved from the 17-0600 project location as part of the LTPP Program. However, the soil classification data were only available for five of the samples. The subgrade soil was generally classified as A-6 under the AASHTO soil classification system: three of the subgrade samples were classified as AASHTO A-6, while the other two were classified as AASHTO A-4.(2)

Laboratory resilient modulus testing results were available for only one sample of the subgrade materials obtained as part of the LTPP Program data collection. Laboratory resilient modulus testing results for the subgrade sample (BAX04) are illustrated in figure 23. Additional subgrade properties, including Atterberg limits and sieve analysis, are summarized in table 21 and table 22, respectively. Average moisture content was 11.6 percent.



1 psi = 6.89 kPa.

Figure 23. Graph. Summary of LTPP Program laboratory-measured subgrade resilient modulus.

Table 21. Summary of subgrade Atterberg limits.

Laboratory Test	Average Test Result (percent)
Liquid limit	23
Plasticity index	13

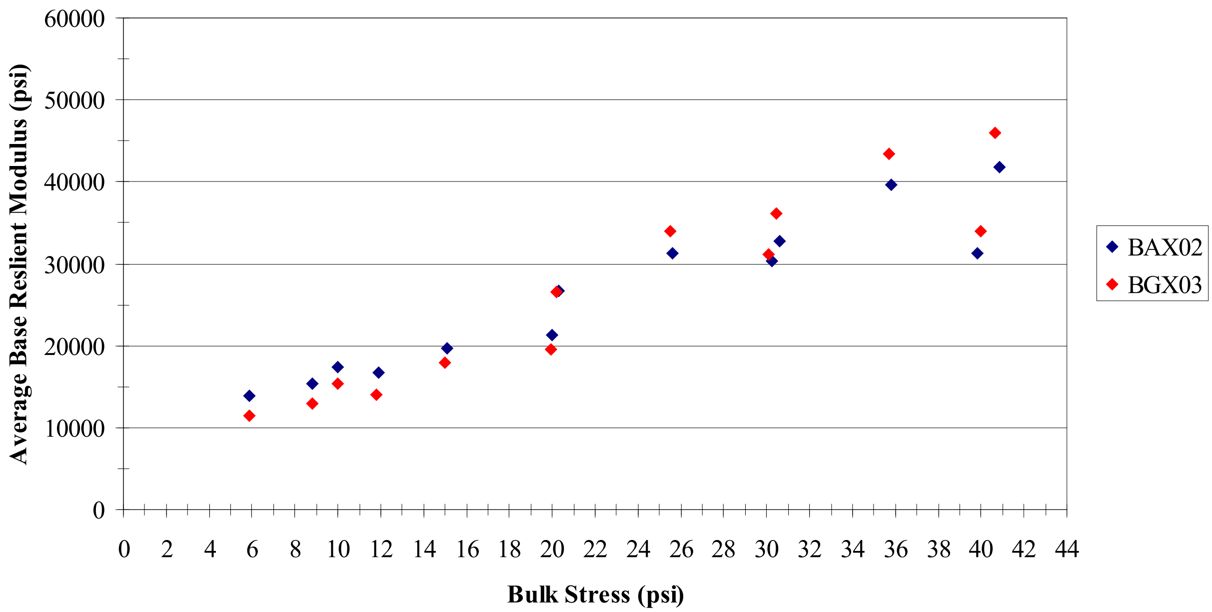
Table 22. Summary of subgrade sieve analysis.

Sieve Size	Average Percent Passing
1 inch	100
$^3/_4$ inch	99
$^1/_2$ inch	99
$^3/_8$ inch	98
No. 4	96
No. 10	94
No. 40	81
No. 80	67
No. 200	58.5

1 inch = 25.4 mm.

Base Aggregate

Seven coarse aggregate samples were obtained as part of the LTPP Program data collection, and the materials were classified as uncrushed gravel. Laboratory resilient modulus testing results were available for two samples of coarse aggregate materials in the Project 17-0600 sections and are illustrated in figure 24. Additional aggregate properties are summarized in table 23 and table 24.



1 psi = 6.89 kPa.

Figure 24. Graph. Summary of LTPP Program laboratory-measured base aggregate resilient modulus.

Table 23. Summary of base aggregate Atterberg limits.

Laboratory Test	Average Test Result (percent)
Liquid limit	6
Plasticity index	1

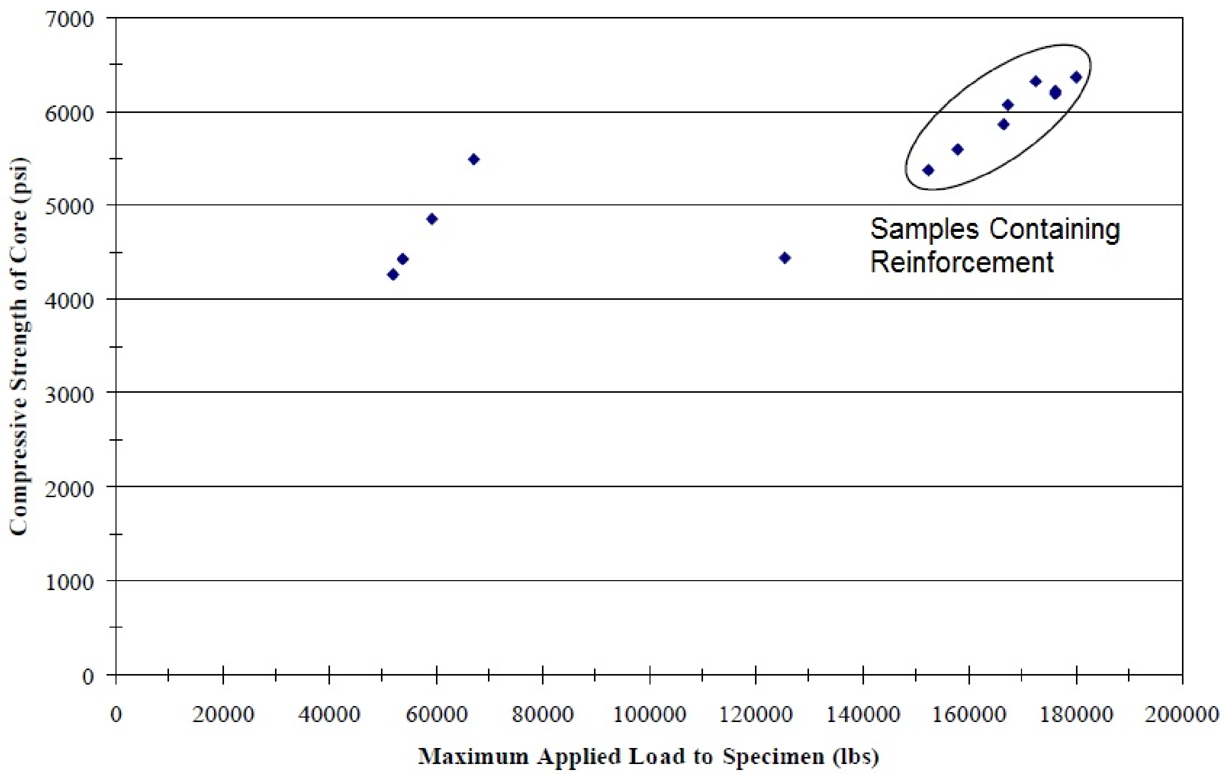
Table 24. Summary of base aggregate sieve analysis.

Sieve Size	Average Percent Passing
1.0 inch	100
$^3/_4$ inch	99.6
$^1/_2$ inch	84.6
$^3/_8$ inch	73.9
No. 4	52.0
No. 10	41.3
No. 40	24.8
No. 80	19.2
No. 200	16.0

1 inch = 25.4 mm.

PCC Layer

The LTPP Program database contained laboratory compressive strength testing results for 13 PCC surface samples from the 17-0600 project section. The compressive strength testing results of cores are summarized in figure 25. Eight of the PCC samples were noted to contain steel reinforcement. The overall average compressive strength was 5,490 psi (37,850 kPa); the average compressive strength of the samples without reinforcement was 4,690 psi (32,340 kPa). Average tensile strength testing results were 725 psi (5,000 kPa).



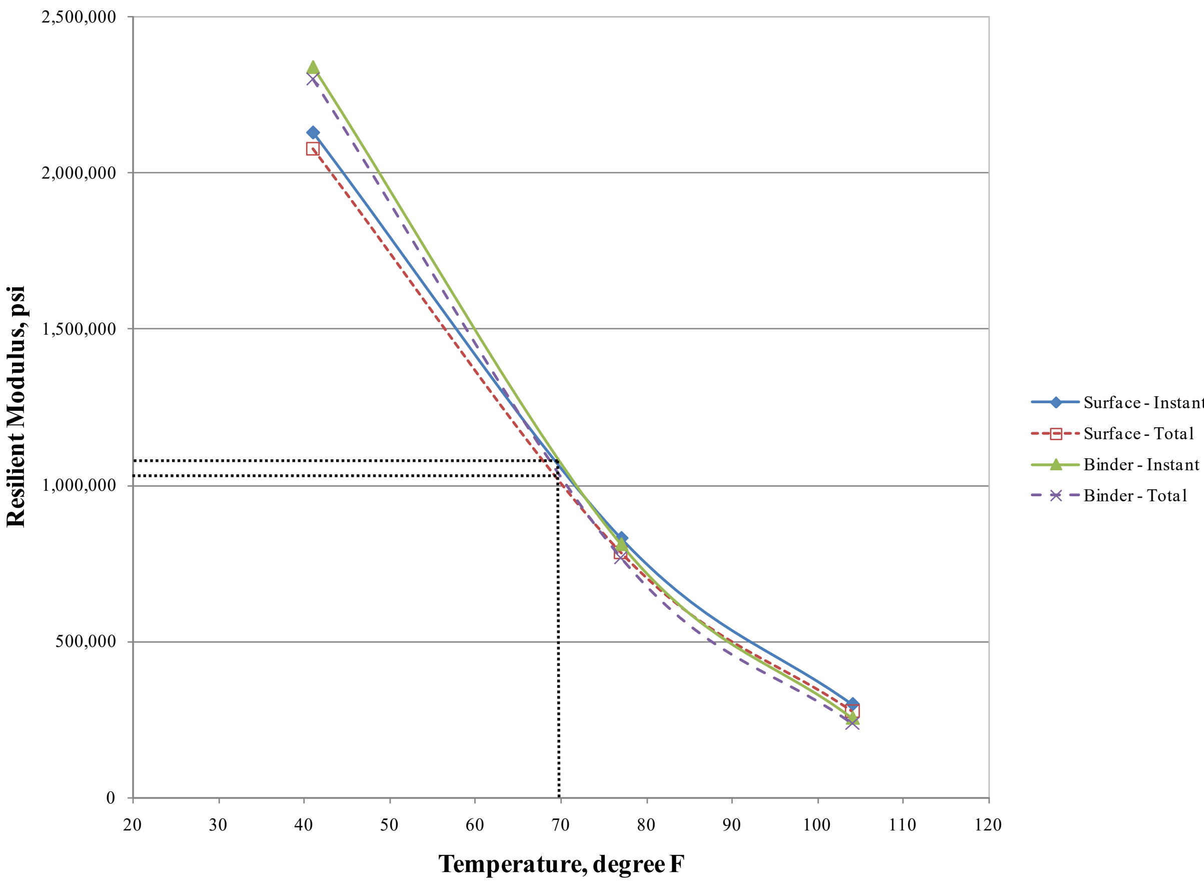
1 psi = 6.89 kPa.
1 lb = 0.454 kg.

Figure 25. Graph. Summary of LTPP Program laboratory-measured compressive strength.

In 1990, the PCC layer was rubblized with a self-propelled resonant frequency breaker. The resulting particle size was noted to range from sand to 152 mm (6 inches) maximum.

HMA Layer

The HMA overlay placed in 1990 consisted of a 6.5-inch (165-mm) binder course and a 1.5-inch (38-mm) surface course. Laboratory testing was conducted for both layers. The resilient modulus testing for each layer is summarized in figure 26. The instantaneous resilient modulus for both layers at 70 °F (21 °C) was approximately 1.1 million psi (7,584,233 kPa). Additional HMA properties for design for the binder course are summarized in table 25 and table 26. It was assumed that the surface layer would be mostly milled off during rehabilitation, so the binder layer properties were more influential.



1 psi = 6.89 kPa.
°F = 1.8 × °C + 32.

Figure 26. Graph. Summary of HMA resilient modulus data from LTPP Program.

Table 25. Summary of existing HMA binder material properties.

Variable	Value
Asphalt grading	PG 64-28
Effective asphalt content, percent	7.75
Air voids, percent	4.8
Total unit weight, lb/ft^3	147

$1\ lb/ft^3 = 0.0160\ g/cm^3$.

Table 26. Summary of existing HMA binder aggregate sieve analysis.

Sieve Size	Average Percent Passing
$^3/_4$ inch	99.6
$^3/_8$ inch	87.0
No. 4	56.4
No. 200	6.8

1 inch = 25.4 mm.

Depth to Rigid Layer/Water Table

The depth to the water table is also required for the MEPDG. However, no data were found for Project 17-0600. In fact, no water table depth data had been reported for the State of Illinois. An average value of 10 ft (3.05 m), considering the mean depth of the water table in a neighboring State, Iowa, was assumed for use in the MEPDG.

Climate/Environment Data

Climate data were obtained from the updated climate files on the MEPDG Web site.[5] The weather station at Champaign, IL, was used for this study. The general climatic category for the case study location is wet-freeze.

Traffic Data

Traffic data were available from the LTPP Program database. Because evaluating the effect of traffic data on design results was not a primary goal of this study, only basic information was used from the available data (total volume, growth, and vehicle class distribution), with the remaining inputs (such as monthly distribution, hourly distribution, and wheel spacing) kept at their default values in the MEPDG software. An AADTT volume of 5,400 vehicles per day was estimated.

ANALYSIS AND INTERPRETATION OF FWD TESTING DATA

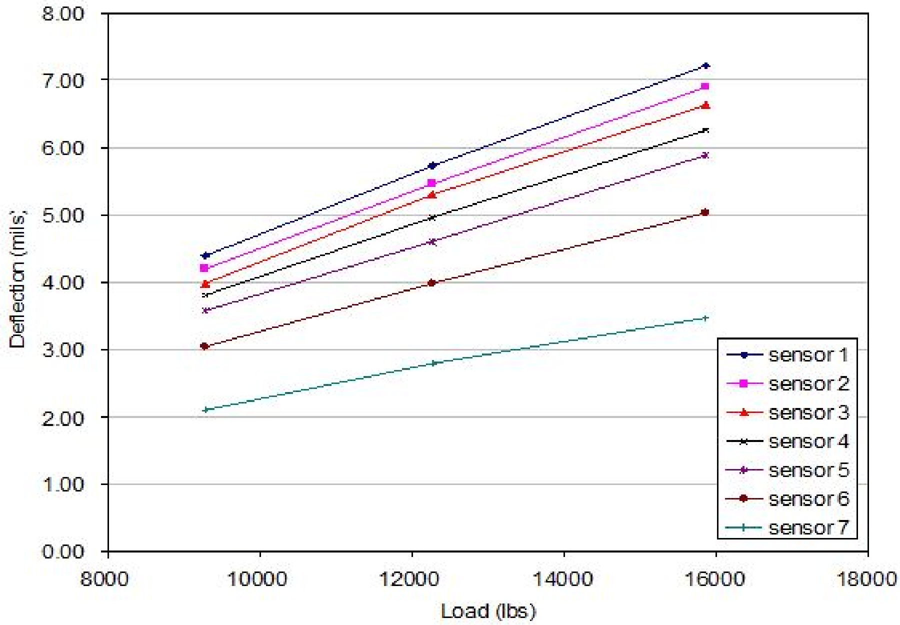
This section presents the data checks and backcalculation analysis of the FWD data, as well as a comparison of the backcalculation results with laboratory testing.

Preprocessing Deflection Data

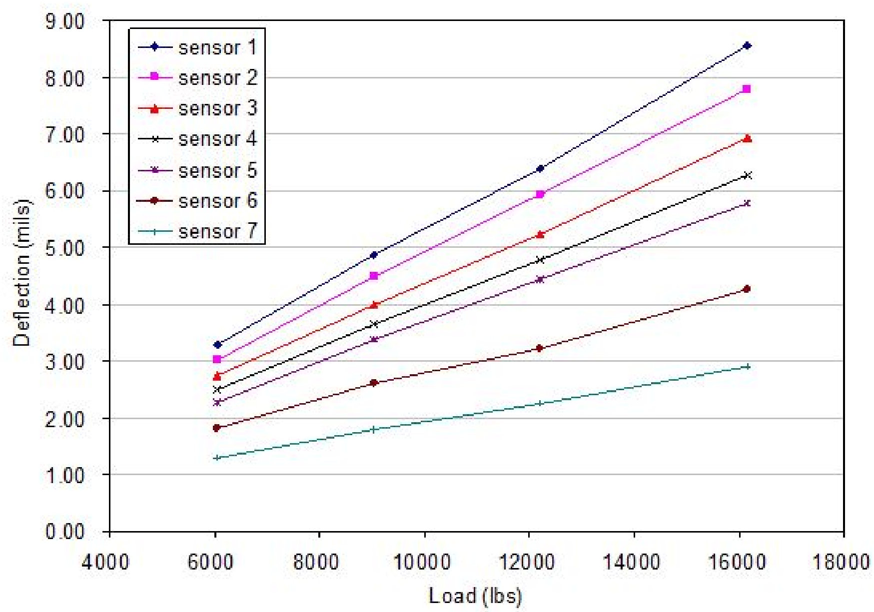
FWD deflection data were checked for linearity. Figure 27 compares the load versus sensor deflection for a single location before rubblization and after rubblization. The measurements show higher deflections and a slightly more irregular pattern after rubblization. The data were deemed to be acceptable for linear analysis.

The shape of the deflection basins before and after rubblization was also checked. Figure 28 shows typical deflection basins before and after rubblization. The shape of the deflection basin after rubblization is more irregular. This was due to the variability of the rubblized material created during the rubblization process. The irregular shape of the deflection basin after rubblization led to significantly higher RMS values.

Figure 29 illustrates the normalized (9,000-lb (4,086-kg)) deflections along the section for data just before rubblization and shortly after placement of the HMA overlay. As the figure illustrates, the deflections were greater after rubblization, and there was an increase in deflections at the beginning of the section. The data point after rubblization at approximately 300 ft (91 m) had lower normalized deflections than the adjacent points. This may be because of variability in the rubblization process, or it could be variability in one of the other pavement layers.



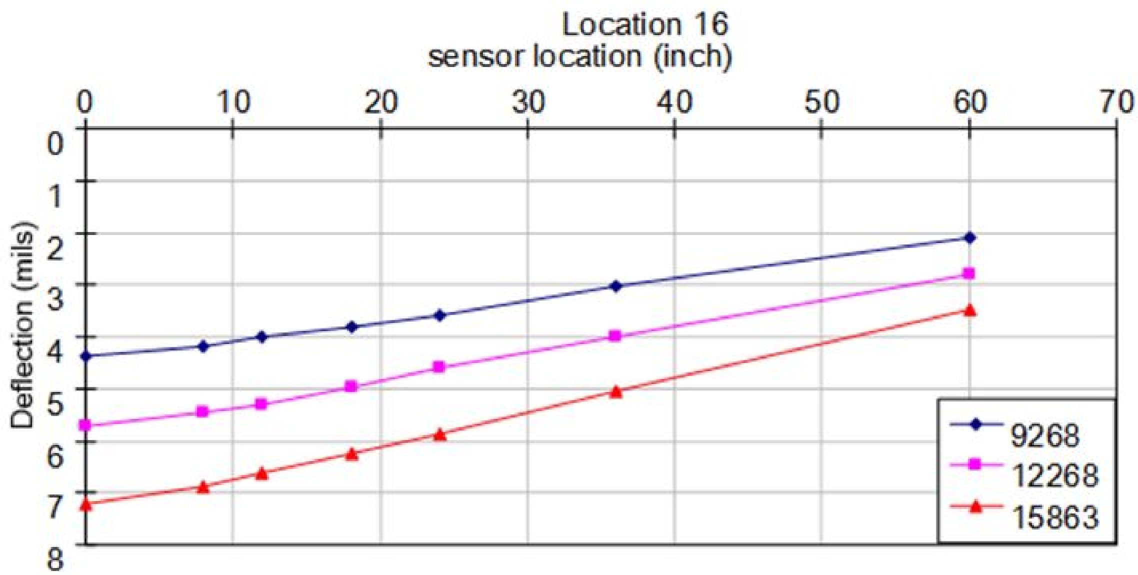
Before Rubblization



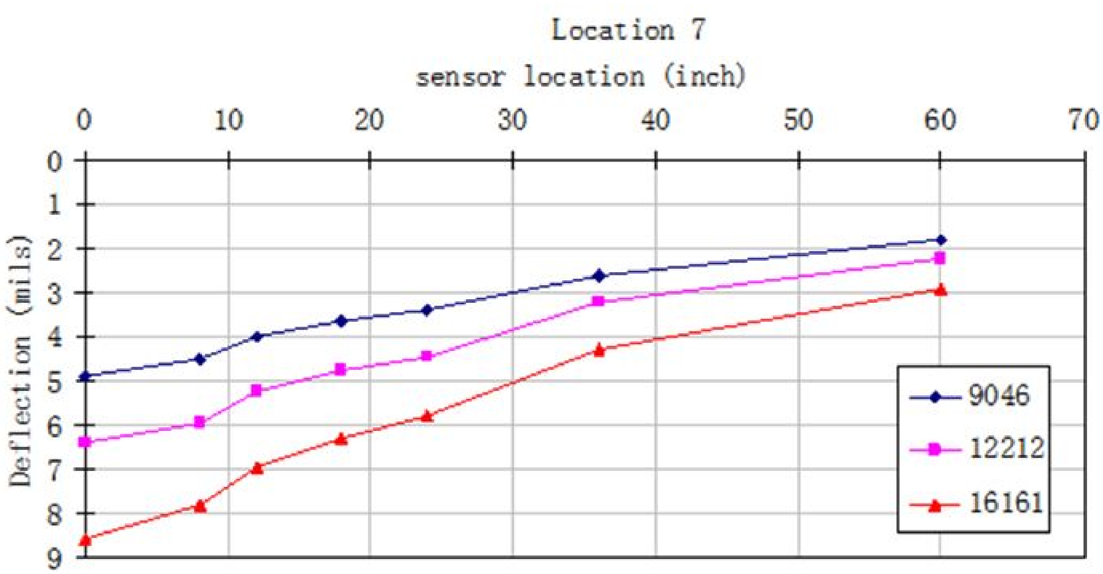
After Rubblization

1 lb = 0.454 kg.
1 mil = 0.0254 mm.

Figure 27. Graphs. Comparison of sample plots for FWD load versus sensor deflection before (top) and after (bottom) rubblization.



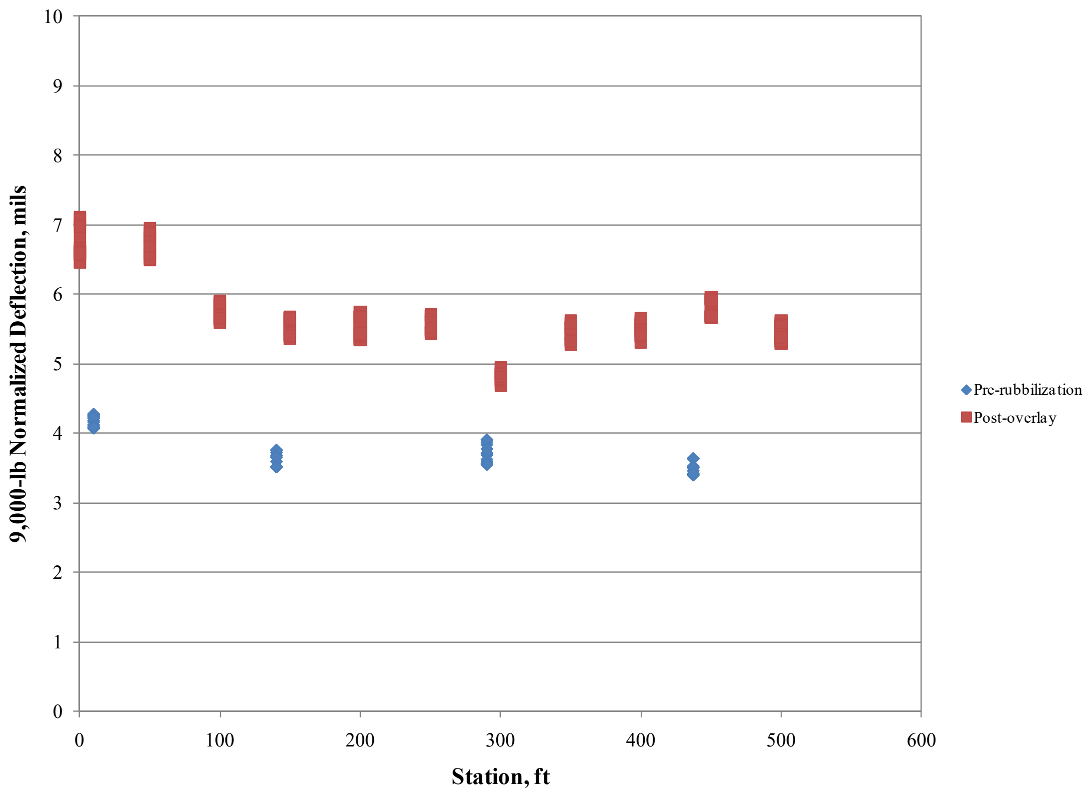
(a) prior to rubblization



(b) after rubblization

1 mil = 0.0254 mm.
1 inch = 25.4 mm.

Figure 28. Graphs. Comparison of typical deflection basins before (top) and after (bottom) rubblization.



1 mil = 0.0254 mm.
1 ft = 0.305 m.

Figure 29. Graph. Normalized deflections along section.

Backcalculation Analysis

The backcalculation of the rubblized cross section was conducted using layered elastic methods. This is because the rubblized PCC no longer acted as a plate and was assumed to behave similar to a granular layer. Three backcalculation programs were used for this backcalculation analysis: (1) MODCOMP©, (2) MICHBACK©, and (3) EVERCALC© to look into the effect of different inverse routines on backcalculated parameters and ultimately on rehabilitation design. In addition, the following layer combinations were used during the backcalculation analysis to determine the most realistic design inputs for the MEPDG software:

- **Case I**: Four-layer system with HMA, RPCC layer, base, and infinite subgrade.
- **Case II**: Four-layer system with HMA, combined RPCC layer and base, top 2 ft (0.6 m) of compacted subgrade, and infinite subgrade.
- **Case III**: Three-layer system with HMA, combined RPCC layer and base, and infinite subgrade.

The seed, minimum, and maximum values for layer moduli for the three- and four-layer backcalculation analyses are shown in table 27 and table 28, respectively.

Table 27. Seed, minimum, and maximum layer moduli for three-layer backcalculation analysis.

Layer	Thickness (inches)	Seed Modulus (psi)	Minimum Modulus (psi)	Maximum Modulus (psi)
HMA	7	500,000	100,000	5,000,000
RPCC + base	17	100,000	30,000	300,000
Subgrade	Infinite	7,500	1,000	50,000

1 psi = 6.89 kPa.
1 inch = 25.4 mm.

Table 28. Seed, minimum, and maximum layer moduli for four-layer backcalculation analysis.

Layer	Thickness (inches)	Seed Modulus (psi)	Minimum Modulus (psi)	Maximum Modulus (psi)
HMA	0	500,000	100,000	5,000,000
RPCC	10	100,000	30,000	300,000
Base	7	30,000	3,000	100,000
Subgrade	Infinite	7,500	1,000	50,000

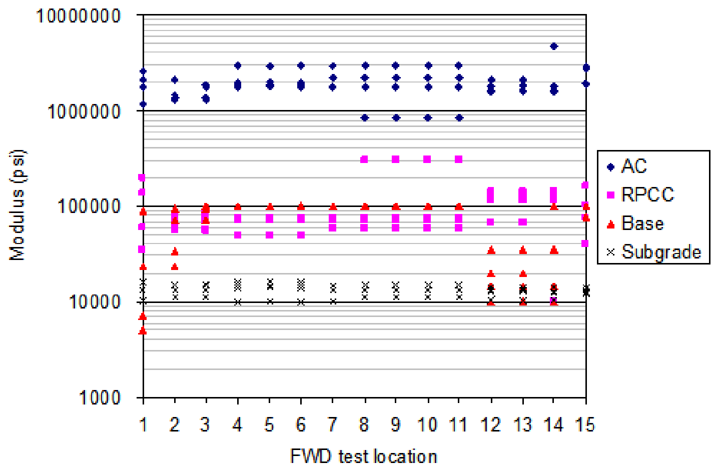
1 psi = 6.89 kPa.
1 inch = 25.4 mm.

The backcalculation was performed for each FWD test location for the available test data most closely following rubblization (1990). This allowed for investigation of the effect of construction variability on backcalculation results. The average layer thicknesses shown in table 27 were used in the analysis.

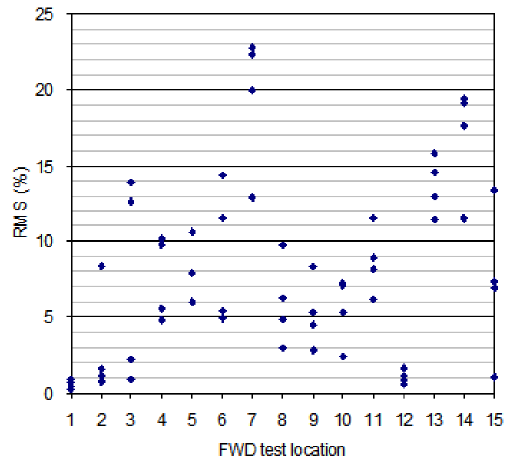
Backcalculation Results

For case I, MODCOMP© results gave unrealistically low granular base layer moduli (average of 11,376 KPa (1,650 psi)). Therefore, MODCOMP© was not used for this case. Figure 30 illustrates the backcalculation results from MICHBACK© and EVERCALC© for the various stations and load levels using a four-layer combination, with separate RPCC and base layers, as described in case I. The results show high variability, especially in the modulus of the rubblized and base layers. A number of locations had a higher modulus for the aggregate base than the rubblized layer. Other locations had base or rubblized layer moduli that reached their maximum limit. The backcalculation results for all these locations had to be excluded from further analysis. The RMS values from MICHBACK© were much higher than those from EVERCALC©.

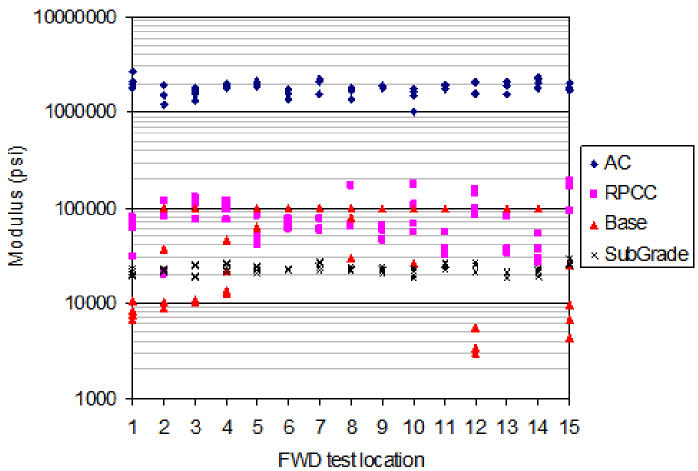
Figure 31 shows the backcalculation results from all three programs for the various stations and load levels using a four-layer combination, with combined RPCC and base layer, as described in case II. The results again showed very high variability, especially in the modulus of the top 2 ft (0.6 m) of compacted subgrade. On the other hand, the HMA layer moduli were fairly consistent, and the modulus of the combined RPCC and base layer was higher than that of the subgrade layer, which was a desired outcome. Locations that had a higher modulus for the subgrade than the combined aggregate base and rubblized layer were excluded from further analysis. The RMS values from MICHBACK© were the highest, followed by MODCOMP©, and then EVERCALC©.



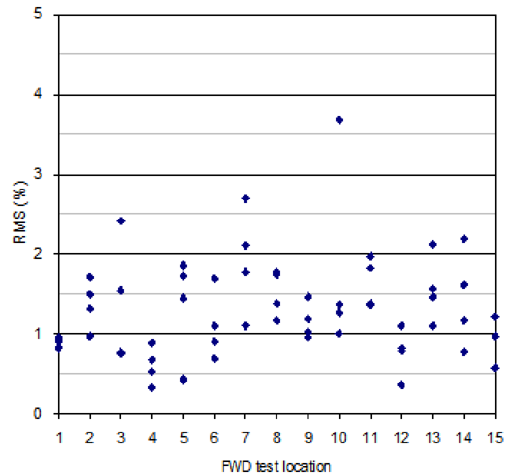
(a) MICHBACK Moduli



(c) MICHBACK RMS



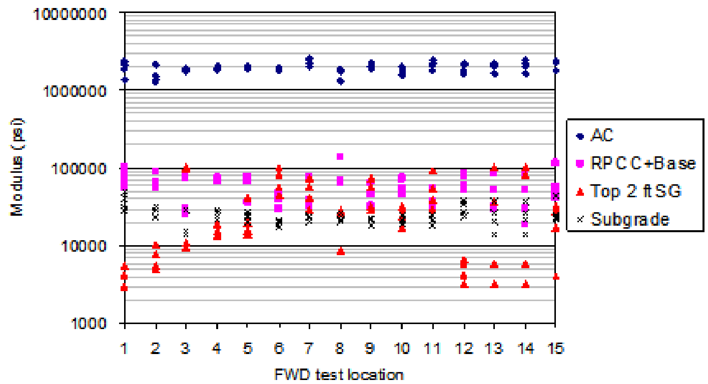
(b) EVERCALC Moduli



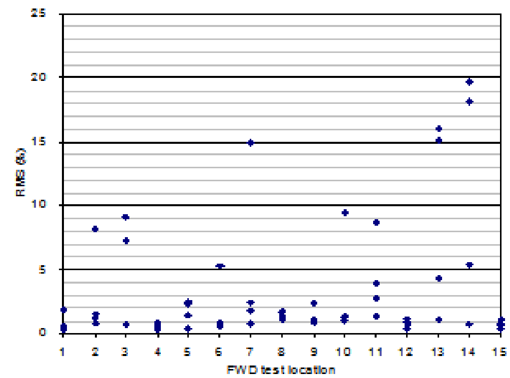
(d) EVERCALC RMS

1 psi = 6.89 kPa.

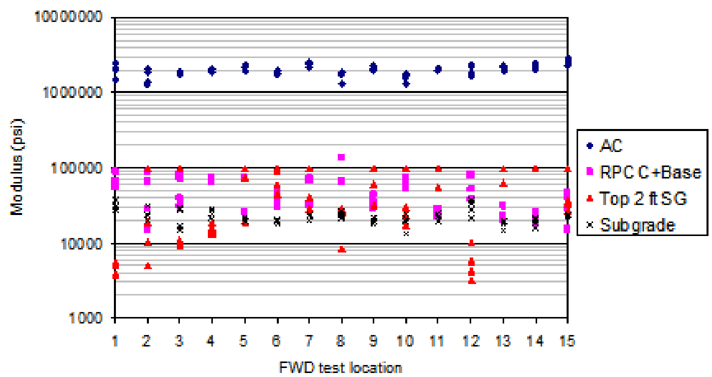
Figure 30. Graphs. Comparison of MICHBACK© and EVERCALC© backcalculation results for four-layer system with separate rubblized layer (case I).



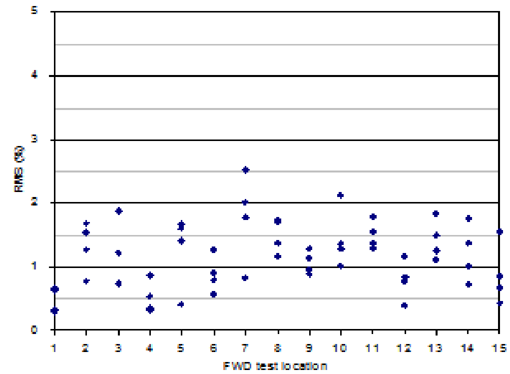
(a) MICHBACK Moduli



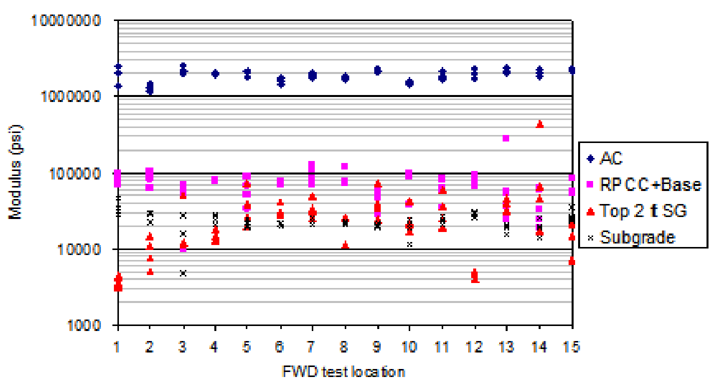
(d) MICHBACK RMS



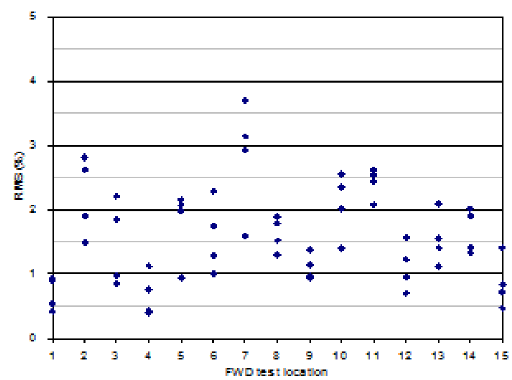
(b) EVERCALC Moduli



(e) EVERCALC RMS



(c) MODCOMP Moduli



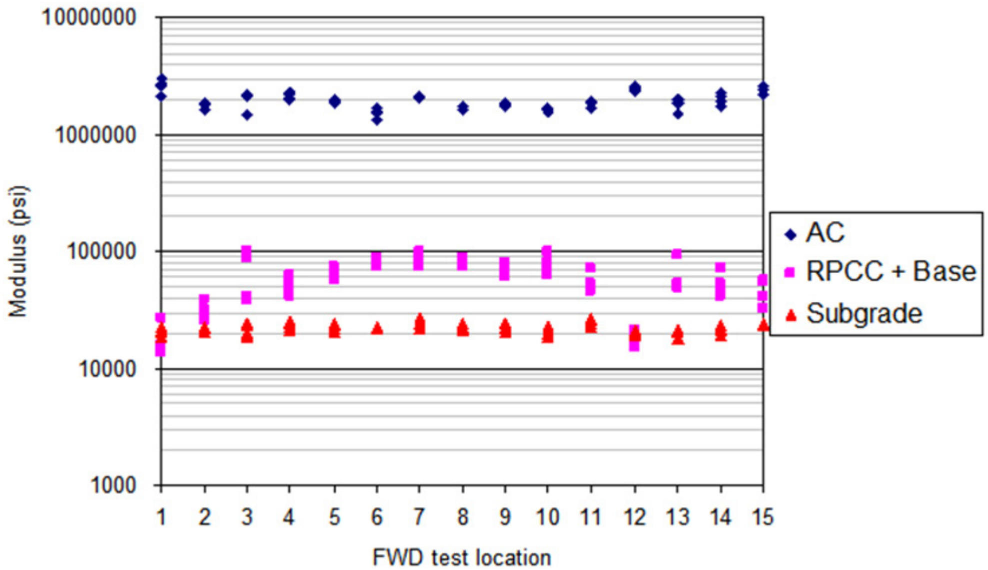
(f) MODCOMP RMS

1 psi = 6.89 kPa.

Figure 31. Graphs. Comparison of MICHBACK©, EVERCALC©, and MODCOMP© backcalculation for four-layer system with combined base and rubblized layer (case II).

For case III, only EVERCALC© was used because it gave the lowest RMS values overall, and the backcalculated layer moduli from the different programs were reasonably close to each other.

Figure 32 and figure 33 show the backcalculation results from EVERCALC© for a three-layer system, with combined RPCC and base layer and an infinite subgrade (case III). The results were less variable than those from a four-layer analysis. The most variable layer, nonetheless, was the combined RPCC and base layer. The great majority of the RMS values were below 2 percent.



1 psi = 6.89 kPa.

Figure 32. Graph. EVERCALC© backcalculated layer moduli for three-layer system with combined base and rubblized layer (case III).

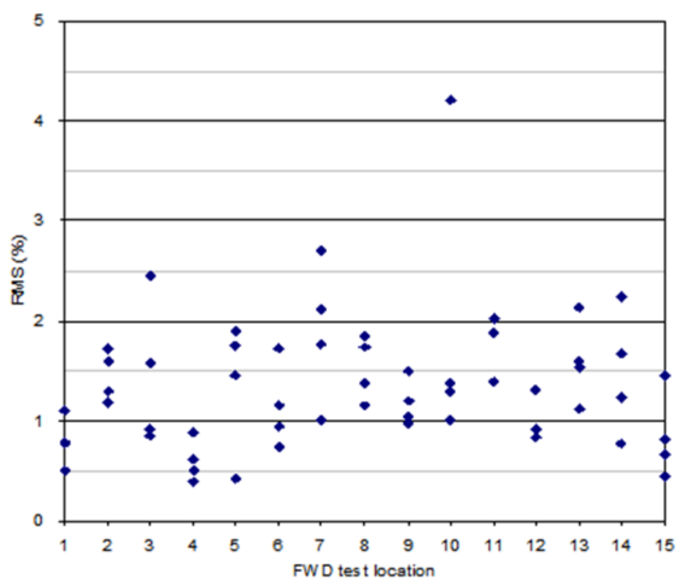


Figure 33. Graph. EVERCALC© RMS values for three-layer system with combined base and rubblized layer (case III).

Table 29 through table 31 show the summary statistics of backcalculation results for cases I, II, and III, respectively. Figure 34 through figure 36 show the average backcalculated modulus values for cases I, II, and III, respectively. Overall, the different programs provided comparable backcalculated modulus values when looking at the overall results. Therefore, the research team concluded that using average or median values would provide reasonable input values for design purposes.

The properties of the intact PCC were also backcalculated using early 1990 data. The AREA method was used to determine the PCC elastic modulus to assess the pre- and post-rubblization properties. Details of this analysis were not included here because rigid pavement backcalculation is discussed in the case studies described in chapters 3 and 4, and backcalculation of intact PCC elastic modulus is only intended for a general comparison of modulus results before and after rubblization. The determined average backcalculated PCC elastic modulus was 6,130,000 psi (42,264,862 kPa).

Table 29. Summary statistics of backcalculation results for four-layer system with separate rubblized layer (case I) (modulus, psi).

Backcalculation Tool	Statistic	HMA	RPCC	Base	Subgrade	Error (percent)
MICHBACK©	Average	1,728,862	112,649	19,400	13,054	7.4
	Standard deviation	278,349	37,838	10,452	1,615	7.2
	COV	.16	.34	.54	.12	.97
	N	16	16	16	16	—
EVERCALC©	Average	1,813,187	110,904	15,043	23,622	0.88
	Standard deviation	302,914	41,004	14,588	2,532	0.34
	COV	.17	.37	.97	.11	.39
	N	23	23	23	23	—

1 psi = 6.89 kPa.
— Indicates not applicable.

Table 30. Summary statistics of backcalculation results for four-layer system with combined base and rubblized layer (case II) (modulus, psi).

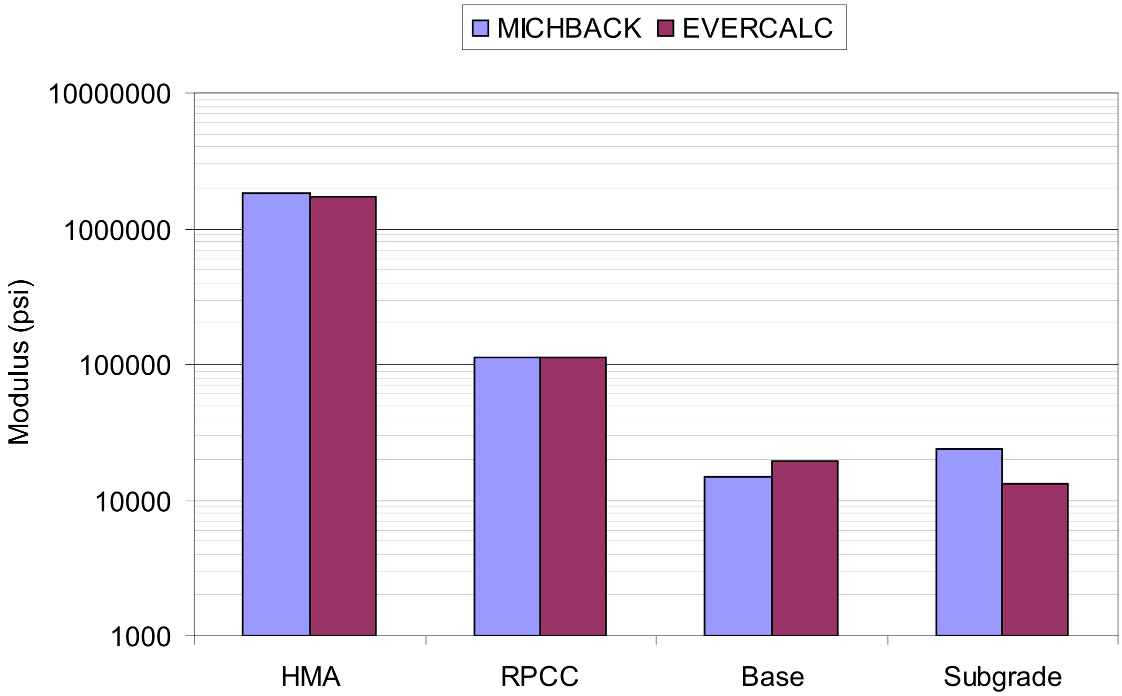
Backcalculation Tool	Statistic	HMA	RPCC + Base	Top 2-ft Subgrade	Subgrade	Error (percent)
MICHBACK©	Average	1,885,826	70,024	16,071	27,190	3.1
	Standard deviation	278,226	18,668	11,668	6,911	5.1
	COV	.15	.27	.73	.25	1.63
	N	43	43	43	43	—
EVERCALC©	Average	1,900,375	65,306	18,250	25,456	0.89
	Standard deviation	319,270	20,555	12,084	4,786	0.44
	COV	.17	.31	.66	.19	.49
	N	32	32	32	32	—
MODTAG	Average	1,884,000	80,470	20,626	24,848	1.5
	Standard Deviation	308,029	34,346	12,125	5,063	0.77
	COV	.16	.43	.59	.20	.51
	N	50	50	50	50	—

1 psi = 6.89 kPa.
— Indicates not applicable.

Table 31. Summary statistics of backcalculation results for three-layer system with combined base and rubblized layer (case III) (modulus, psi).

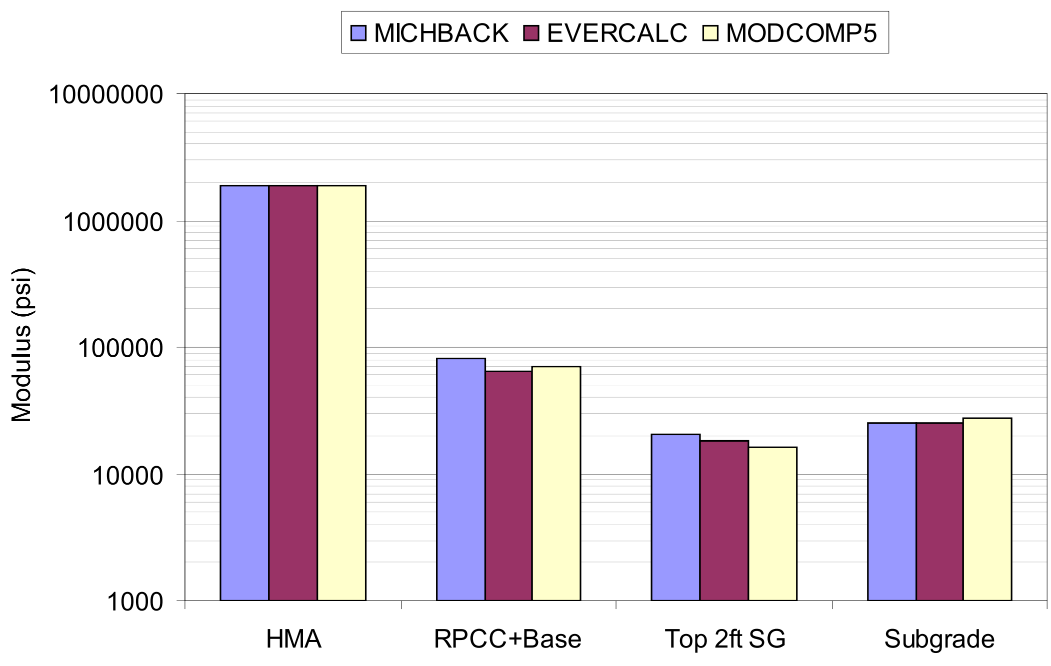
Backcalculation Program	Statistic	HMA	RPCC+Base	Subgrade	Error (percent)
EVERCALC©	Average	1,969,727	57,173	22,257	1.3
	Standard deviation	345,789	23,691	2,037	0.64
	COV	.18	.41	.09	.48
	N	60	60	60	—

1 psi = 6.89 kPa.
— Indicates not applicable.



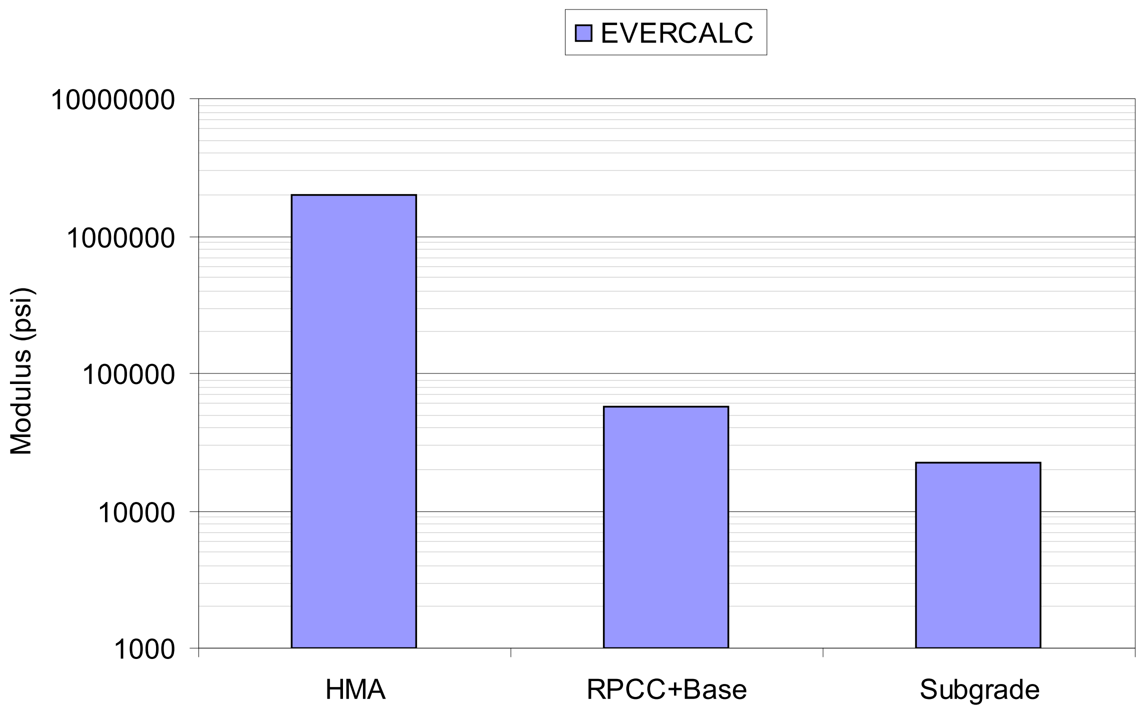
1 psi = 6.89 kPa.

Figure 34. Graph. Summary of average backcalculated moduli for four-layer system with separate rubblized layer (case I).



1 psi = 6.89 kPa.

Figure 35. Graph. Summary of average backcalculated moduli for four-layer system with combined base and rubblized layer (case II).



1 psi = 6.89 kPa.

Figure 36. Graph. Summary of average backcalculated moduli for three-layer system with combined base and rubblized layer (case III).

Backcalculation Modeling Issues and Recommendations

As previously described, the backcalculation results for a rubblized pavement were generally more variable than those of a conventional pavement. The RMS values obtained from MICHBACK© were unacceptably high, while those from EVERCALC© were generally acceptable (below 2 percent). However, the backcalculated layer moduli from the different programs agreed reasonably well when averaged across the FWD test locations within the project. Some locations did give unreasonable backcalculation results, and such results should be excluded from the analysis.

The base and rubblized layer moduli exhibited the greatest variability. Combining the base and rubblized layers into a single layer reduced the possibility of erroneous backcalculation results (e.g., a higher base modulus than rubblized layer modulus) and decreased variability in the results. In case II, the 2 ft (0.6 m) of compacted top subgrade had a lower moduli than the natural subgrade; this can be explained by the fact the natural subgrade modulus represents the effective modulus for a halfspace. This value is generally higher because the modulus increases with depth. This increase is due to (1) higher confinement for sandy soils and (2) consolidation for clayey soils. Also, the modulus of the top 2 ft (0.6 m) compacted subgrade has greater variability, which could be attributable to construction variability.

Comparison of Backcalculation and Laboratory Testing Results

To assist in evaluating which layer characteristics would be appropriate to use in the MEPDG software, the results of the backcalculation (field tests) were compared with results obtained from laboratory testing conducted as part of the LTPP Program. While it was outside of the scope of this study to develop new correlations or conversion factors between the two, it was still beneficial to evaluate these relationships and see how they may influence MEPDG input selection.

Unbound Materials

The difference in stress state is often a general argument when comparing field to laboratory results of unbound materials. To compare field (FWD) obtained results with those obtained from laboratory testing (or vice versa), the resilient modulus was estimated for the stress conditions during FWD testing. The stress conditions account for the overburden pressure of the pavement and the stress due to loading. However, it is recommended (and discussed later) to combine the rubblized layer with the base layer, so this issue becomes less important in this case study.

PCC/Rubblized Layer

The different laboratory results for the intact PCC provided substantially different elastic modulus correlations. Based on compressive strength testing, a PCC modulus of 4 million psi (27,579,000 kPa) was estimated; however, a PCC modulus of 8.5 million psi (58,605,436 kPa) was estimated based on tensile strength testing. The backcalculated average PCC modulus was 6,130,000 psi (42,264,862 kPa), which was approximately 4.9 million psi (33,784,300 kPa) when applying the recommended dynamic-to-static correction factor of 0.8.

For the case I backcalculation (RPCC as an individual layer), the RPCC modulus values ranged from 104,000 to 119,000 psi (717,054 to 820,476 kPa). These values were approximately 2 percent of the backcalculated intact static modulus, and were lower than the level 3 recommendation (150,000 psi (1,034,213 kPa)) provided in the MEPDG. When the RPCC was combined with the unbound base, the moduli were even lower. The MEPDG also provides level 1 recommendations, which are based on anticipated variability in the fracture process. The fractured slab moduli included in the level 1 discussion were much greater than those backcalculated for the rubblized layer in case study 2.

HMA Layer

Laboratory results for HMA layer modulus for several temperatures are shown in figure 26. Temperature at the time of FWD testing was about 14 °C (58 °F) based on borehole temperature data. Based on the data summarized in figure 26, a corresponding thickness-weighted equivalent laboratory HMA modulus of about 1.6 million psi (11,031,611 kPa) was obtained, compared with average backcalculated moduli ranging from about 1,680,000 to 1,920,000 psi (11,583,192 to 13,237,933 kPa), depending on selected layer model and backcalculation program. This translated to a laboratory-to-field (backcalculated) modulus ratio of about 0.83 to 0.95.

REHABILITATION DESIGN RESULTS

An HMA overlay rehabilitation design was analyzed for this case study to assess design differences between backcalculated inputs and laboratory-based inputs. The overall design level for this rehabilitation (HMA over JCPC (fractured)) was level 3. The HMA layer design levels can range from 1 through 3, whereas the RPCC design level cannot be changed and the unbound layers can be level 2 or 3 inputs.

Design Criteria and General Inputs

The MEPDG calls for determining the HMA overlay thickness by trial and error. A trial thickness was assumed, and the program was executed to predict the different performance measures (i.e., cracking, rutting, and IRI). For a given desired rehabilitation design life (20 years), the level of distress at the design life should not exceed a prescribed limiting value, which is summarized in table 32 for case study 2. A reliability level of 90 percent was selected because it corresponded to the average (expected) performance, which can be compared with the actual observed performance.

Table 32. Summary of analysis parameter inputs.

Variable	Value
Initial IRI, inches/mi	63
Terminal IRI, inches/mi	172
HMA surface down cracking, long cracking, ft/mi	2,000
HMA bottom up cracking, alligator cracking, percent	25
HMA thermal fatigue, ft/mi	1,000
Chemically stabilized layer fatigue fracture, percent	25
Permanent deformation—total pavement, inches	0.75
Permanent deformation—HMA only, inches	0.25

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.
1ft/mi = 0.19 m/km.

For new HMA materials, level 1 analysis requires conducting E^* (complex modulus) laboratory testing (ASTM D3496) at loading frequencies and temperatures of interest for the given mixture.[7] Level 2 analysis does not require E^* laboratory testing; instead, the user can input asphalt mix properties (gradation parameters) and laboratory binder test data (from G^* testing or other conventional binder tests). The MEPDG software calculates the corresponding asphalt viscosity values; it then uses the modified Witczak equation to predict E^* and develops the master curve for the HMA mixture. The same procedure is used for level 3 analysis to estimate the HMA dynamic modulus except that no laboratory test data are required for the binder, and typical values for the selected binder grade are used. The inputs summarized previously in table 25 and table 26 for the existing HMA were used for the new HMA material.

Incorporation of Backcalculation Results with the MEPDG Software

Several analyses were executed with the MEPDG design software (summarized in table 33) to evaluate the influence of the unbound and bound layer inputs, as discussed in the following sections. Version 1.003 of the software was used, as well as the nationally calibrated performance models.

Table 33. Summary of inputs for MEPDG runs.

Analysis Run Number	Subgrade Modulus (psi)	Base Modulus (psi)	RPCC Modulus (psi)
1[a]	8,300	9,300	68,800
2[b]	23,600	15,000	110,900
3[c]	8,300	9,300	110,900
4[d]	14,500	15,000	150,000
5[e]	6,400 (24 inches) 8,900 (infinite)	N/A	40,500
6[f]	7,200 (24 inches) 8,700 (infinite)	N/A	49,900
7[g]	18,250 (24 inches) 25,500 (infinite)	N/A	65,300
8[h]	7,800	N/A	35,400
9[i]	22,300	N/A	57,200

[a]Analysis 1: Corrected backcalculation values based on EVERCALC© results (case I).
[b]Analysis 2: Uncorrected backcalculation values based on EVERCALC© results (case I).
[c]Analysis 3: Corrected backcalculation values for subgrade and base, uncorrected value for RPCC based on EVERCALC© results (case I).
[d]Analysis 4: Representative of laboratory and MEPDG values (case I).
[e]Analysis 5: Corrected backcalculation values with combined RPCC and base and a 24-inch (610 mm) upper subgrade based on EVERCALC© results (case II).
[f]Analysis 6: Corrected backcalculation values with combined RPCC and base and a 24-inch (610 mm) upper subgrade based on MODTAG results (case II).
[g]Analysis 7: Uncorrected backcalculation values with combined RPCC and base and a 24-inch (610 mm) upper subgrade based on EVERCALC© results (case II).
[h]Analysis 8: Corrected backcalculation values with RPCC and base combined based on EVERCALC© results (case III).
[i]Analysis 9: Uncorrected backcalculation values with RPCC and base combined based on EVERCALC© results (case III).
1 psi = 6.89 kPa.
1 inch = 25.4 mm.
N/A = Not applicable.

HMA Layer

The MEPDG software does not include the backcalculation-based modulus for an existing HMA layer in a HMA over jointed portland cement concrete pavement (JPCP) (fractured) design. The existing HMA modulus is determined internally based on the HMA mixture properties entered. The required inputs include the aggregate gradation of the asphalt mix and the AC grade (PG in this instance). The new HMA material properties used for this analysis are summarized in table 34 and table 35.

Table 34. Summary of new HMA surface material properties.

Variable	Value
Asphalt grading	PG 64-28
Effective asphalt content, percent	11.5
Air voids, percent	4.0
Total unit weight, lb/ft^3	148

1 lb/ft^3 = 0.0160 g/cm^3.

Table 35. Summary of new HMA surface aggregate sieve analysis.

Sieve Size	Average Percent Passing
$^3/_4$ inch	98.8
$^3/_8$ inch	86.6
No. 4	55.8
No. 200	6.3

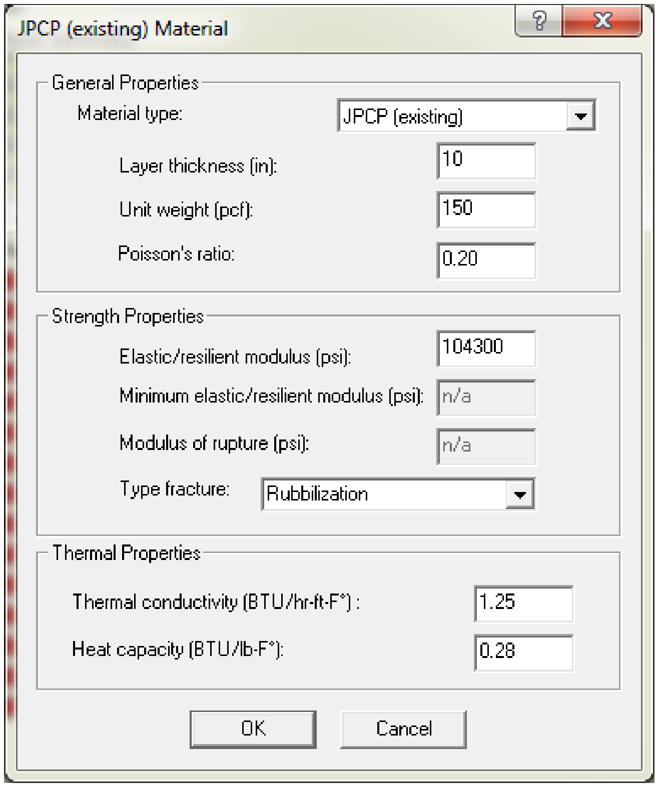
1 inch = 25.4 mm.

PCC/Rubblized Layer

The rubblization option is under "Type fracture" in the JPCP material properties as shown in figure 37. The backcalculated RPCC modulus is also input under the strength properties.

Unbound Materials

For unbound materials (and bedrock), only level 1 analysis calls for FWD testing in rehabilitation and reconstruction designs. However, level 1 for unbound materials is not available in the HMA over JPCP (fractured) design analysis. Level 2 inputs allow the input of a layer modulus or correlation with other strength test data, as shown in figure 38. Level 3 allows the input of a modulus, which is generally based on typical values of the soil classification. The MEPDG lists typical modulus values based on soil classification but warns that they are very approximate and strongly recommends some form of testing, especially noting the use of FWD testing and backcalculation.



1 inch = 25.4 mm.
1 pcf = 16 kg/m^3.
1 psi = 6.89 kPa.
1 BTU/h-ft-°F = 1 W/m-°C.
1 BTU/lb-°F = 4,186.8 J/kg-°C.

Figure 37. Screen Capture. RPCC input screen.

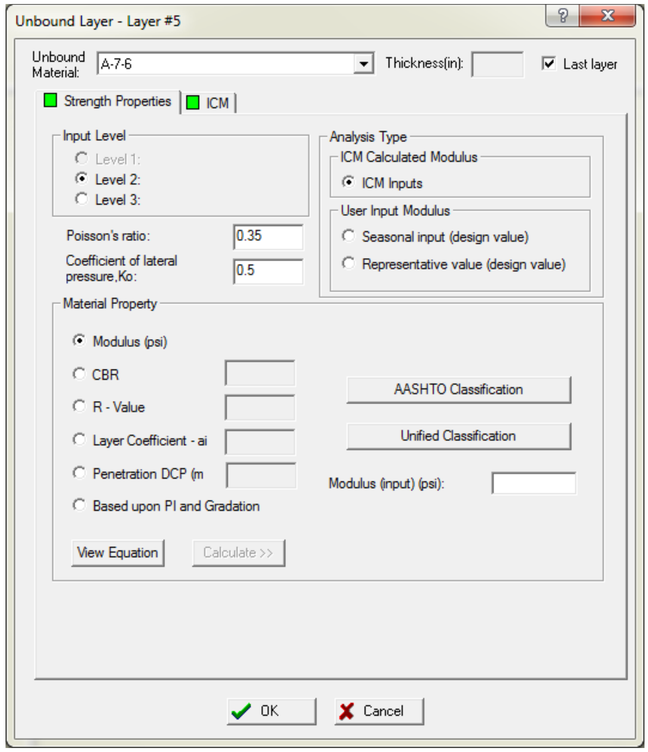


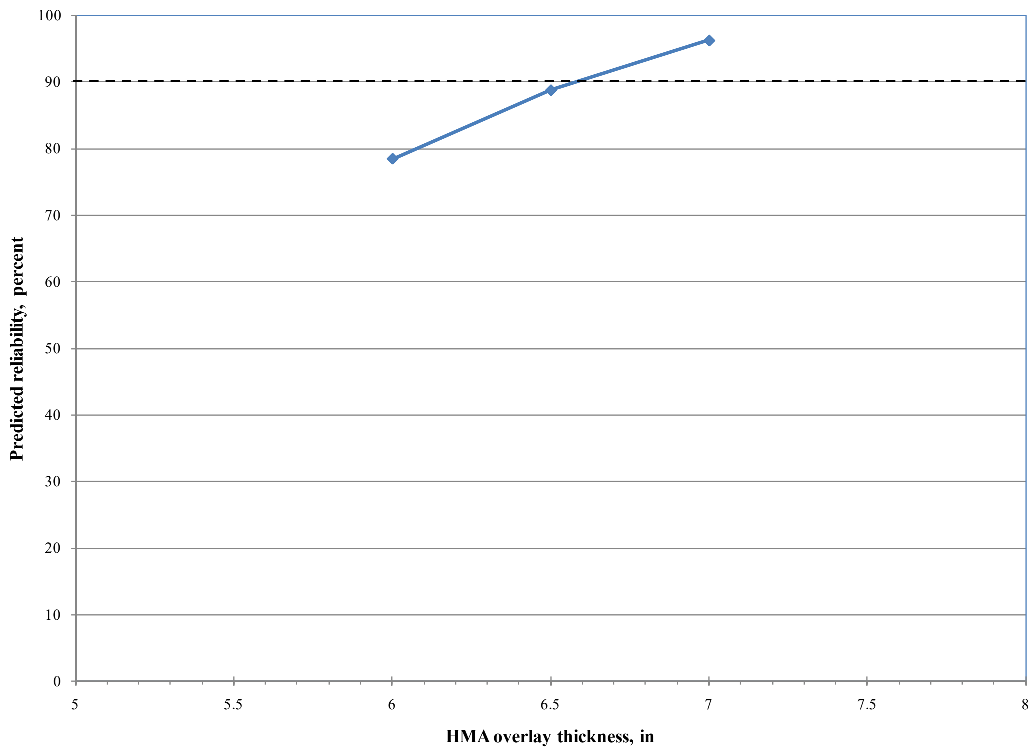
Figure 38. Screen Capture. Unbound layer input screen for HMA over JPCP (fractured) design.

The MEPDG notes that the reason for caution is related to using the wrong assumptions: either a fairly strong subgrade material may be erroneously assumed to be semi-infinite while it may actually be less than 1 m (3 ft) thick (e.g., as part of an embankment), or conversely, a weak subgrade soil may be assumed to be semi-infinite while it may, in reality, be overlying a stronger soil or bedrock.

The MEPDG also notes that for granular materials, moduli values that matched FWD backcalculated results were 50 to 70 percent higher than the typical laboratory-tested values, while they were two to three times the typical laboratory determined values for subgrade soils. Guidance in the MEPDG when this report written was to use the previously established coefficients (summarized in table 3.6.8 of the MEPDG (0.35 for subgrade and 0.62 for aggregate base below a flexible pavement)) to adjust backcalculated layer moduli for use in design.(1) The adjusted value should be entered as the modulus for the unbound layer. The layer properties used in the design analysis are summarized in table 33. ICM modeling was also used because data were not available throughout the year.

Evaluation of Design Results

As noted previously, the analyses summarized in table 33 were run with the MEPDG software (version 1.0) using nationally calibrated performance models to evaluate the influence of varying the backcalculation-based inputs. The required HMA overlay thicknesses were determined for each set of inputs, and the change in distress predictions and reliabilities were compared. The required HMA overlay thickness for all analyses was 178 mm (7 inches) to achieve a 90-percent reliability level. The required HMA overlay thickness was controlled primarily by HMA surface deformation (illustrated in figure 39 for analysis 1), with other distress predictions showing minimal differences between the analyses. A thinner overlay could be selected if maintenance (patching) to repair the rutting was performed in an earlier year.



1 inch = 25.4 mm.

Figure 39. Graph. Summary of required overlay thickness based on surface rutting—analysis 1.

As summarized in table 36, the surface rutting, which controls the overall design, was essentially unaffected by the range of inputs used in the design analyses. In addition, the total rutting had fairly minimal differences between the inputs (or layering) used; that is, the differences did not result in the selection of a different overlay thickness.

Table 36. Summary of predicted rutting.

Analysis Run Number	Surface Rutting (inches)	Total Rutting (inches)
1	0.15	0.43
2	0.16	0.30
3	0.16	0.42
4	0.17	0.34
5	0.15	0.39
6	0.16	0.38
7	0.16	0.27
8	0.15	0.39
9	0.16	0.27

1 inch = 25.4 mm.

SUMMARY

Three backcalculation programs were used to analyze FWD deflection data from various test locations (stations) within the rubblized pavement project. The deflection data showed considerable variability within the project. The RMS values obtained from the MICHBACK© backcalculation program were also very high. It is recommended that one conduct FWD tests at multiple locations and use the average of backcalculated layer moduli. This should provide consistent service life predictions irrespective of the backcalculation program used, and therefore result in a similar recommended overlay design.

For an HMA overlay on rubblized concrete pavements, the critical performance measure in the MEPDG software is surface rutting. However, surface rutting can be addressed through maintenance at some intermediate year (less than 20 years), and a thinner HMA overlay would be acceptable. The surface rutting predictive model is mainly sensitive to HMA overlay thickness and HMA and rubblized layer moduli. Therefore, care should be taken in selecting the modulus for the rubblized layer. In this particular case, the RPCC modulus was slightly lower than the suggested level 3 input in the MEPDG. It may be useful to combine the rubblized layer with the existing unbound base layer when using some backcalculation programs (for example MICHBACK©, in this case study). The remaining unbound layer moduli should be entered as the adjusted backcalculated values (until ongoing studies are completed, the correction factors developed by Von Quintus and Killingsworth should be applied to the backcalculation results), but the existing HMA modulus determined by backcalculation could not be used at the time of this research.(7)

CHAPTER 3. CASE STUDY 3: RIGID PAVEMENT EVALUATION AND OVERLAY DESIGN FOR UNSTABILIZED BASE LAYER

PROJECT OVERVIEW

Project 32-0200 is a part of Interstate 80 (I-80) in Lander County, NV. This pavement section was selected as the case study for rigid pavement rehabilitation alternatives. Specifically, test section 32-0203 was selected as a rigid pavement cross section with a JPCP and granular base layer over subgrade. This section is representative of the following selection factors:

- Rigid pavement.
- Dry-freeze climate zone.
- Principal interstate, rural functional class.
- "Poor" pavement condition.
- Sandy silt subgrade classification.
- No (or deep) rigid layer.

This project was first introduced into the LTPP Program as a Specific Pavement Study (SPS) in January 1993 and was recorded as construction number 1. Construction number 2 was posted in October 1995 when crack sealing was carried out. Crack sealing was performed again in September 1997, which was referred to as construction number 3. According to the LTPP Program data, the JPCP design consisted of a 295-mm (11.6-inch) PCC surface layer, a 5.7-inch (145-mm) dense graded aggregate base, a 20-inch (513-mm) granular subbase, and a subgrade of which the top 12 inches (305 mm) was treated with lime.

PAVEMENT CONDITION/PERFORMANCE

Distress survey results, in terms of joint faulting and slab cracking, were available in the LTPP Program database from 1996 to 2003. According to the records, faulting did not exceed 0.1 inch (2.5 mm) until 2003. However, when considering slab cracking, the studied pavement section was in very poor condition.

Table 37 presents the number of cracks identified for test section 32-0203 from 1996 to 2003. To convert the number of cracks to the percentage of slabs cracked, the assumption that only one transverse crack occurred within each slab can be made. The section was 500 ft (152.5 m) long, which may have consisted of 33 to 34, 15-ft (4.6-m) slabs per lane. Based on table 37, all the slabs were cracked by 2003, and rehabilitation was needed.

Table 37. Distress surveys on transverse cracking.

Survey Date	No. of Low-Severity Transverse Cracks	No. of Medium-Severity Transverse Cracks	No. of High-Severity Transverse Cracks	Total No. of Transverse Cracks
3/26/1996	50	2	1	53
12/15/1998	0	119	11	130
11/17/1999	2	0	127	129
6/14/2000	3	0	178	181
4/24/2001	1	0	163	164
6/25/2002	13	0	164	177

TEST SECTION DATA

The use of the MEPDG for design requires determination of a significant number of inputs, even for a level 3 analysis. Most of the required inputs can be obtained from the LTPP Program database for test section 32-0203. However, none of the project 32-0200 sections had a complete set of data available for the rehabilitation design using the MEPDG. Therefore, data from sections adjacent to test section 32-0203 were also used to help define the necessary inputs.

Deflection-Testing Data

Deflection data collected in 1995, 1996, 2000, 2002, and 2003 for test section 32-0203 was available in the LTPP Program database.

Equipment

Deflection testing was conducted with a Dynatest® model 8000 FWD.

Sensor Configuration

The sensors configuration for the FWD testing is presented in table 38.

Table 38. Sensor configuration for the FWD testing.

Configuration	Sensor Number 1	2	3	4	5	6	7	8	9
Offset (inches)	0	8	12	18	24	36	48	60	-12

1 inch = 2.54 mm.

Number of Drops and Load Levels

Four load level targets (6,000, 9,000, 12,000, and 16,000 lb (2,724, 4,086, 5,448, and 7,264 kg)) with four drops at each load level were performed and recorded. Seating drops were also performed, but data were not recorded.

Test Locations/Lanes and Increments

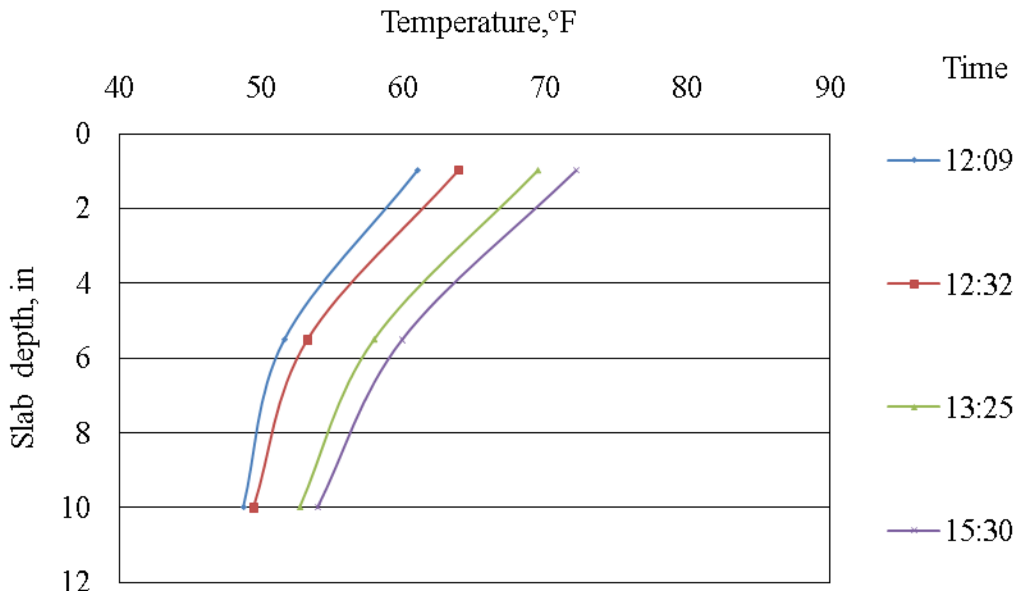
The locations of the LTPP Program FWD testing are presented in table 39. The test location used to backcalculate the PCC elastic modulus and the modulus of subgrade reaction was J1. In addition, to determine the presence of loss of support, the test results at location J2 were used, and the results from locations J4 and J5 were used to establish the characteristics of joint load transfer. The pavement deflections were measured at 10 different locations along the section between 10:58 a.m. and 2:08 p.m. The distance between tests conducted at mid-slab varied between 30 and 70 ft (9.2 and 21.4 m), whereas the testing performed along the pavement edge and at the corners was at intervals varying between 9 and 23 ft (2.7 and 7 m).

Table 39. Location of the deflection test on the slab.

LTPP Code	Location
J1	JPCP middle lane at middle panel
J2	JPCP pavement edge at corner
J3	JPCP pavement edge at mid panel
J4	JPCP load transfer outer wheelpath at joint approach
J5	JPCP load transfer outer wheelpath at joint leave
J6	JPCP outer wheelpath at mid-panel (SPS-4)
J7	JPCP pavement edge at corner on widen lane SPS-2 sections
J8	JPCP pavement edge at mid panel on widen lane SPS-2 sections

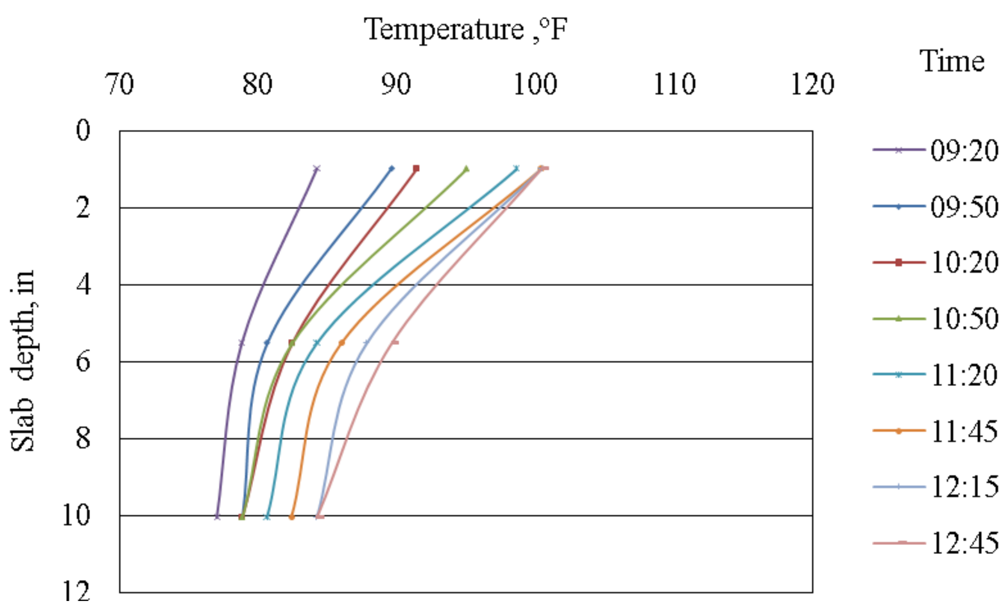
Temperature Measurements

Slab temperature measurements were recorded at depths of 1, 5.5, and 10 inches (25.4, 139.7, and 254 mm) in drilled holes at half-hour intervals during the deflection testing in 1996, 2000, and 2002. For the test performed in 2003, the temperatures were taken at depths of 1, 2, 3.7, and 7.9 inches (25.4, 50.8, 94, and 200.7 mm). Slab temperatures measured in conjunction with the FWD tests on test section 32-0203 from 1996 to 2003 were extracted from the LTPP Program database and presented in figure 40 to figure 43. As observed in these figures, slab temperatures are season and time dependent. For instance, the average slab temperature for the 1996 test that was carried out in late March was 58 °F (14.4 °C), whereas the average temperature for the 2002 test performed in late June was 96 °F (35.6 °C). The effect of the slab temperature distribution on the backcalculation results is discussed in the following sections.



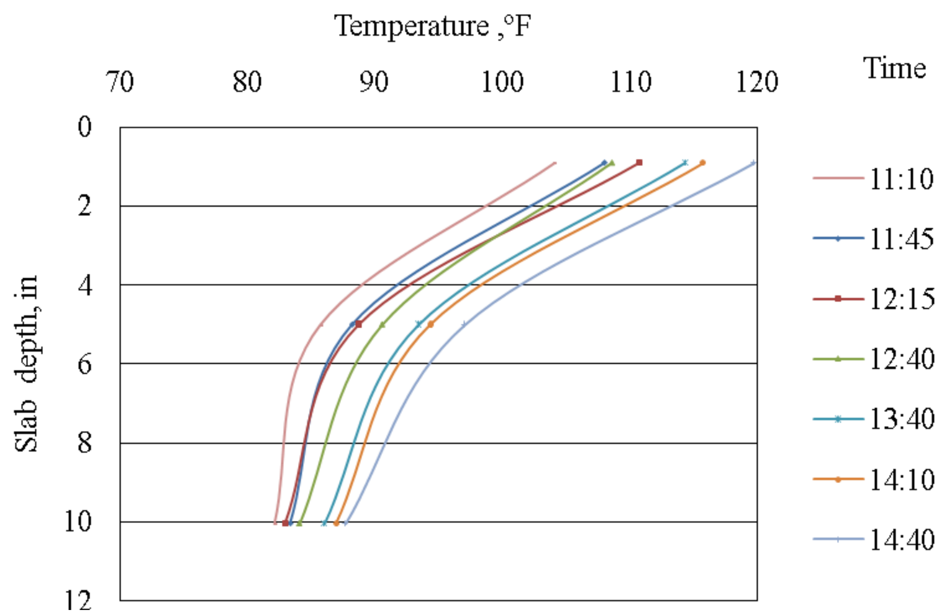
1 inch = 2.54 cm.
°F = 1.8 × °C + 32.

Figure 40. Graph. Slab temperatures for the FWD test performed on 03/26/1996—sunny to partly cloudy.



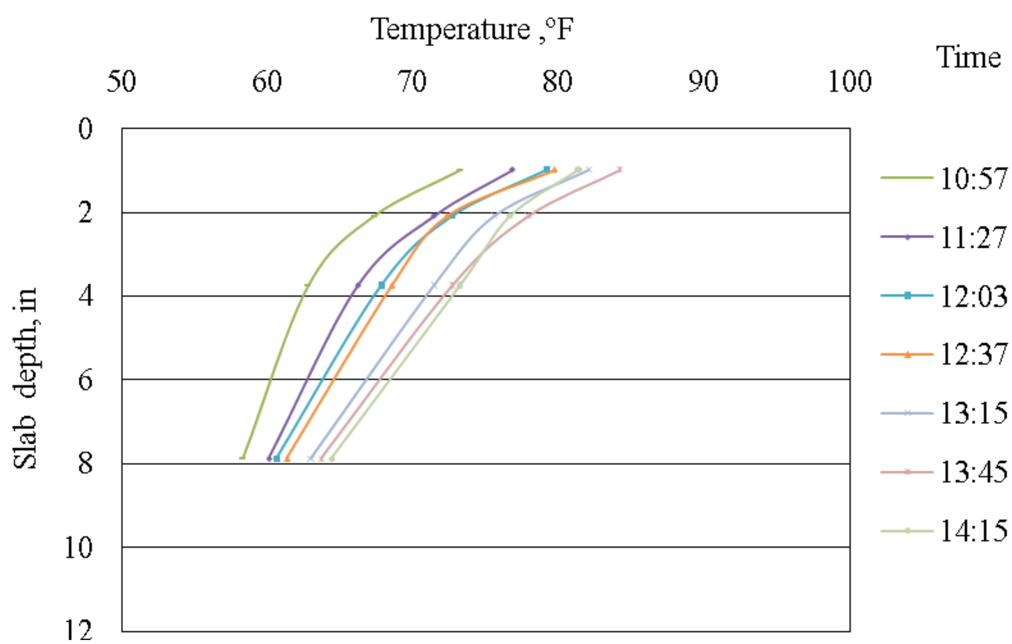
1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 41. Graph. Slab temperatures for the FWD test performed on 06/15/2000—sunny.



1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 42. Graph. Slab temperatures for the FWD test performed on 06/25/2002—sunny to cloudy.



1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 43. Graph. Slab temperatures for the FWD test performed on 10/28/2003—sunny.

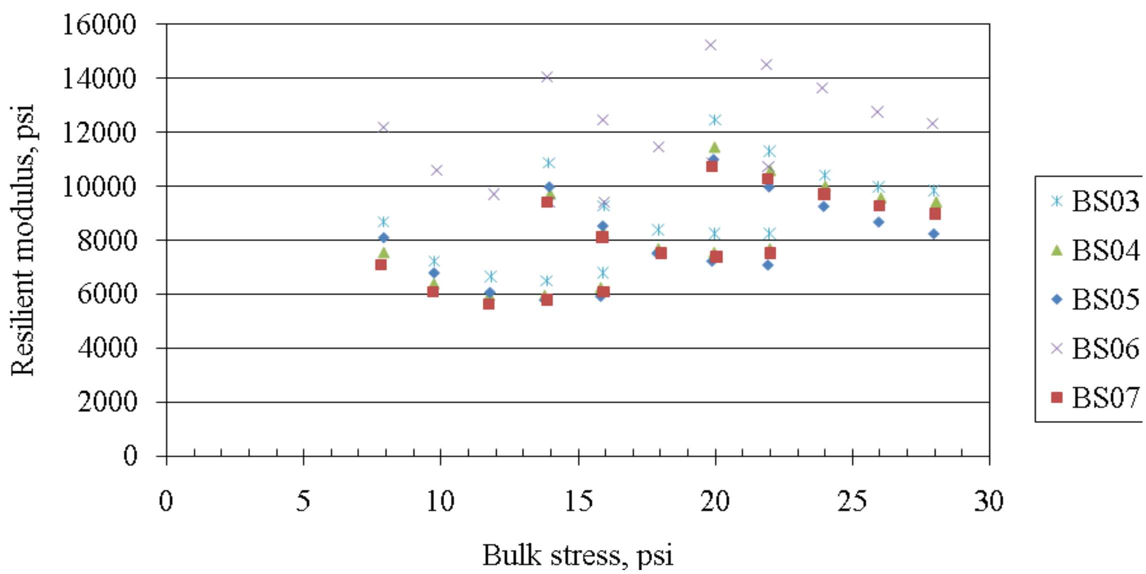
Material Properties Data

This section summarizes the data obtained from LTPP database for test section 32-0203 regarding its subgrade, base, subbase, PCC material properties, traffic, climate, and depth of water table or stiff layer.

Subgrade

Samples were retrieved from the subgrade to determine the soil classification according to the AASHTO system.[(2)] Although the subgrade of test section 32-0203 was not sampled, it was assumed to consist of AASHTO A-4 soil in this study because this test section was located between test sections 32-207 and 32-211, which both had AASHTO A-4 subgrades. The top 305 mm (12 inches) of the subgrade was treated with lime.

Laboratory resilient modulus testing was performed on several subgrade samples. In total, six specimens were tested for the project 32-0200 sections, five of which were from sections that had AASHTO A-4 subgrade. The results are depicted in figure 44. The sample ID is used as legend in figure 44, which also indicates from which section the sample was collected (see table 40).



1 psi = 6.89 kPa.

Figure 44. Graph. LTPP Program data of laboratory resilient modulus testing for the subgrade.

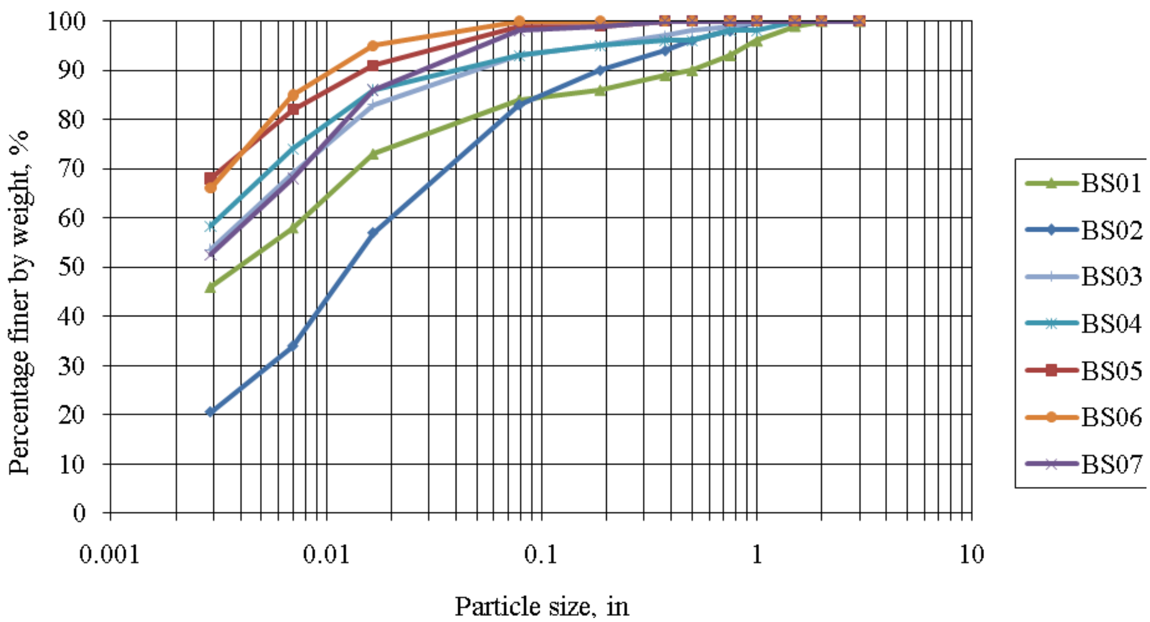
No laboratory tests were performed on the Poisson's ratio, coefficient of lateral pressure, or Atterberg limits for any of this pavement section. The Atterberg limits were established only on one sample (BS01 from test section 32-0205), which yielded liquid and plastic limits of 42 and 23 percent, respectively. However, test section 32-0205 was defined to have AASHTO A-7-6 subgrade, and the Atterberg limits could not be used for test section 32-0203. Therefore, the default values suggested for AASHTO A-4 subgrade in the MEPDG were used. The average moisture content was 16.3 percent.

The particle gradation is summarized in table 40; gradation is also illustrated in figure 45. This was based on the assumption that test section 32-0211, the closest section to test section 32-0203, had the same subgrade as test section 32-0203.

Table 40. Summary of subgrade gradation.

Sieve Size	Average Percent Passing
3 inch	100.0
2 inch	100.0
1.5 inch	99.9
1 inch	99.1
$^{3}/_{4}$ inch	98.3
$^{1}/_{2}$ inch	97.1
$^{3}/_{8}$ inch	96.6
No. 4	94.9
No. 10	92.9
No. 40	81.6
No. 80	67.1
No. 200	52.1

1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 45. Graph. Gradation of the subgrade samples.

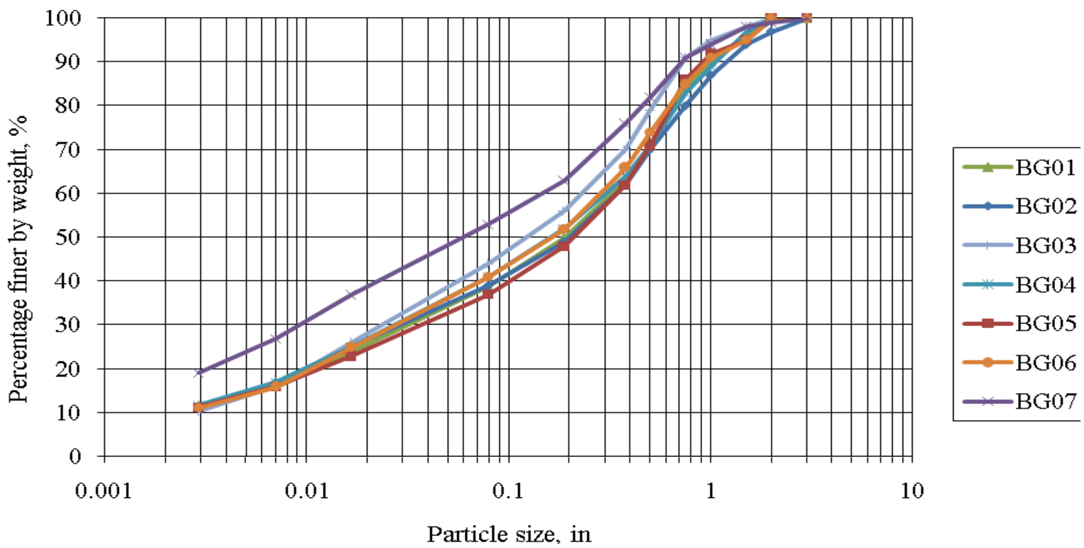
Granular Subbase

Aggregate gradation for the subbase material is summarized in table 41, with gradation also illustrated in figure 46. Average moisture content was determined to be 6.0 percent. No information regarding the Atterberg limits was available for the subbase, so the default values for crushed gravel were used for the analysis.

Table 41. Summary of subbase aggregate gradation.

Sieve Size	Average Percent Passing
3 inch	100.0
2 inch	99.4
1.5 inch	96.1
1 inch	91.1
$^{3}/_{4}$ inch	85.7
$^{1}/_{2}$ inch	74.0
$^{3}/_{8}$ inch	66.1
No. 4	52.9
No. 10	42.0
No. 40	26.4
No. 80	17.9
No. 200	12.4

1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 46. Graph. Gradation of the subbase samples.

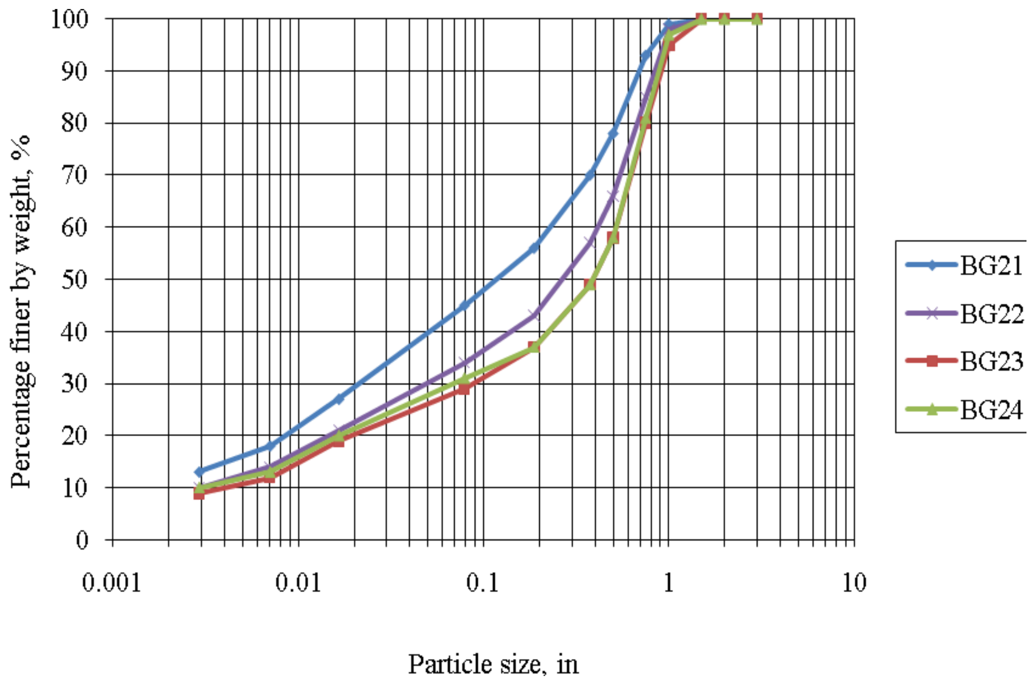
Granular Base

Aggregate gradation for the base layer is summarized in table 42; figure 47 illustrates base aggregate gradations. Three coarse aggregate samples were collected and classified as crushed gravel. The average moisture content was 5.0 percent, but no information regarding the Atterberg limits was available. Therefore, the default inputs for the crushed gravel material were used in the analysis. The resilient modulus of these samples was also measured, as illustrated in figure 48.

Table 42. Summary of base aggregate gradation.

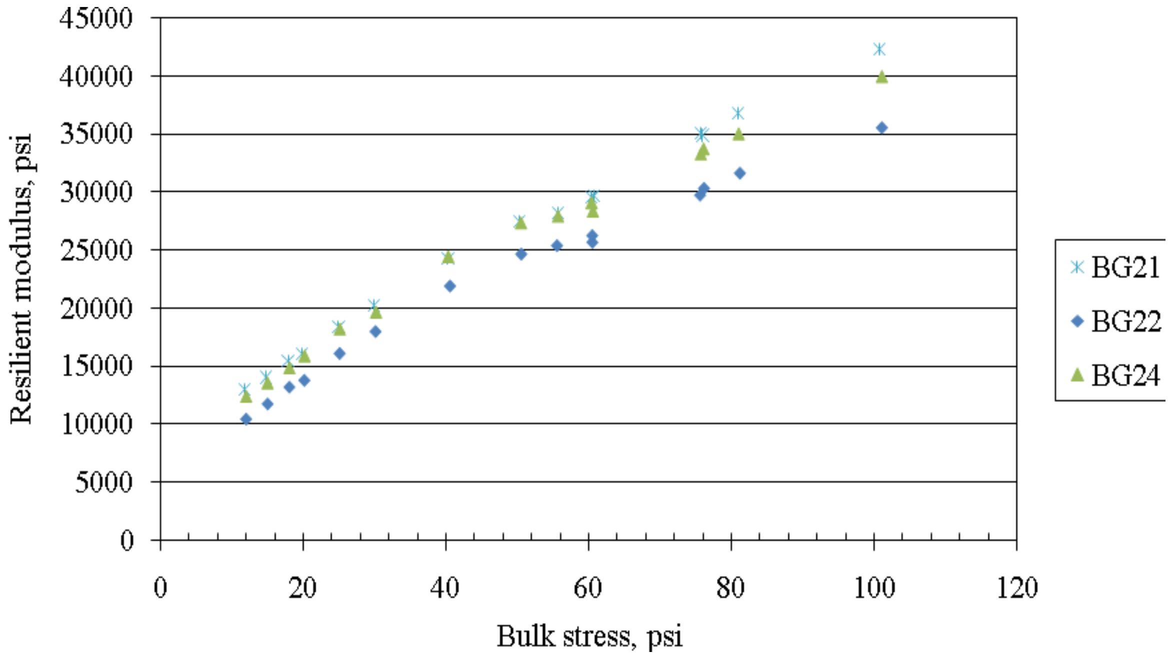
Sieve Size	Average Percent Passing
3 inch	100.0
2 inch	100.0
1.5 inch	100.0
1 inch	97.3
$^{3}/_{4}$ inch	84.8
$^{1}/_{2}$ inch	65.0
$^{3}/_{8}$ inch	56.3
No. 4	43.3
No. 10	34.8
No. 40	21.8
No. 80	14.3
No. 200	10.5

1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 47. Graph. Gradation of the base samples.



1 psi = 6.89 kPa.

Figure 48. Graph. LTPP data of laboratory resilient modulus testing for the aggregate base.

PCC Layer

The LTPP Program database contained the 28-day compressive strength, flexural strength, static elastic modulus, and density data for the PCC of the project 32-0200 sections. The available test results are summarized in table 43. Tests for the coefficient of thermal expansion (CTE) were also performed, but only on one sample (CP09) from test section 32-0258 (which yielded a CTE value of 5.3×10^{-6}/°F (9.5×10^{-6}/°C)).

Table 43. Summary of PCC laboratory testing.

Laboratory Test	Average Test Result[a]
Compressive strength (psi)	3,423
Modulus of rupture (psi)	572
Elastic modulus (psi)	2,783,000
Poisson's ratio	0.12
Density (lb/ft^3)	138.6

[a]Based on available data for odd-numbered sections.
1 psi = 6.89 kPa.
1 lb/ft^3 = 0.0160 g/cm^3.

The Poisson's ratio adopted was 0.12, as shown in table 43. The PCC measured Poisson's ratio for test section 32-0203 was 0.1, which was not representative of a typical value (between 0.15 and 0.20). However, the research team noticed that the measured Poisson's ratio was quite consistent within the odd-numbered test sections (32-0201, 32-0203, etc.) with a mean of 0.12 and a standard deviation of 0.02. The measurements were also consistent within the even-numbered test sections (32-0202, 32-0204, etc.) with a mean of 0.15 and a standard deviation of 0.02. After looking into the PCC mix designs of all the sections, the team found that the mix design (material used as well as the proportioning) of the odd-numbered sections was the same, and for these sections, the water-to-cement ratio was 0.49. On the other hand, for the even-numbered sections, which also had a common mix proportioning, the water-to-cement ratio was 0.32. According to the study by Vu et al., PCC with higher water-to-cement ratios tends to present lower Poisson's ratios because of its higher level porosity.(9) Considering the high water-to-cement ratio for test section 32-0203, the team concluded that the Poisson's ratio was low, but the consistent measurement of the Poisson's ratio made it reliable for use in design. The remaining properties summarized in table 43 are also based on the odd-numbered sections.

Although type IP cement was used in the PCC mix, Type I was used in the MEPDG design because type IP was not an available option in the software. The proportioning of the PCC mix for test section 32-0203 was extracted from the LTPP Program database and summarized in table 44. The curing method was not documented in the LTPP Program database, but a curing compound was assumed (because this was the default curing method in the MEPDG).

Table 44. Mixture design for the existing PCC layer.

Cement (lb/yd^3)	Coarse Aggregates Content (lb/yd^3)	Coarse Aggregate Type	Fine Aggregates Content (lb/yd^3)	Water Content (lb/yd^3)	Water-to-Cement Ratio
423	2,024	Dolomite	1,198	206	0.49

$lb/yd^3 = 0.593\ kg/m^3$.

For section test 32-0203, dowel bars were 18 inches (457.2 mm) long and 1.5 inches (38.1 mm) in diameter. Joints were spaced 15 ft (4.6 m) apart and sealed with silicone. In the LTPP Program database, the width of the widened design lane was 14 ft (4.3 m). No specific information was available regarding the shoulder type. In this design, no extra edge support from the shoulder was assumed.

Depth to Rigid Layer/Water Table

The depth to the water table is also required for designing an overlay using the MEPDG. However, no such data was available for the project 32-0200 sections. Therefore, the default value of 10 ft (3.05 m) in the MEPDG was adopted for the design.

Climate/Environment Data

The climatic data from the weather station at Elko, NV, was used in the MEPDG to represent the climate for the studied pavement section. This weather station was about 80 mi (128.8 km) east of the project 32-0200 sections and 500 ft (152.5 m) higher in elevation. The general climatic category for test section 32-0203 is dry-freeze.

Traffic Data

In the LTPP Program database, the only available two-way annual average daily traffic (AADT) was reported as 5,880 in 1996, and it was also recorded that 926 trucks were counted in the design lane. Therefore, the percentage of trucks in the two-way AADT in the design lane was determined to be 16 percent. To establish a growth rate, traffic counts in terms of the two-way AADT performed by the Nevada Department of Transportation for the studied pavement were employed. Assuming the same percentage of trucks, the AADTT in the design lane was calculated from year 1990 to 2007 and presented in table 45. Through a simple regression analysis, a compound truck traffic growth rate was calculated as 2.3 percent, as depicted in figure 49.

Table 45. AADTT in the design lane of test section 32-0203 from 1990 to 2007.

Year	AADTT
1990	812
1991	824
1992	865
1993	846
1994	878
1995	900
1996	942
1997	866
1998	877
1999	1,054
2000	1,043
2001	1,030
2002	1,056
2003	1,016
2004	1,088
2005	1,080
2006	1,184
2007	1,248

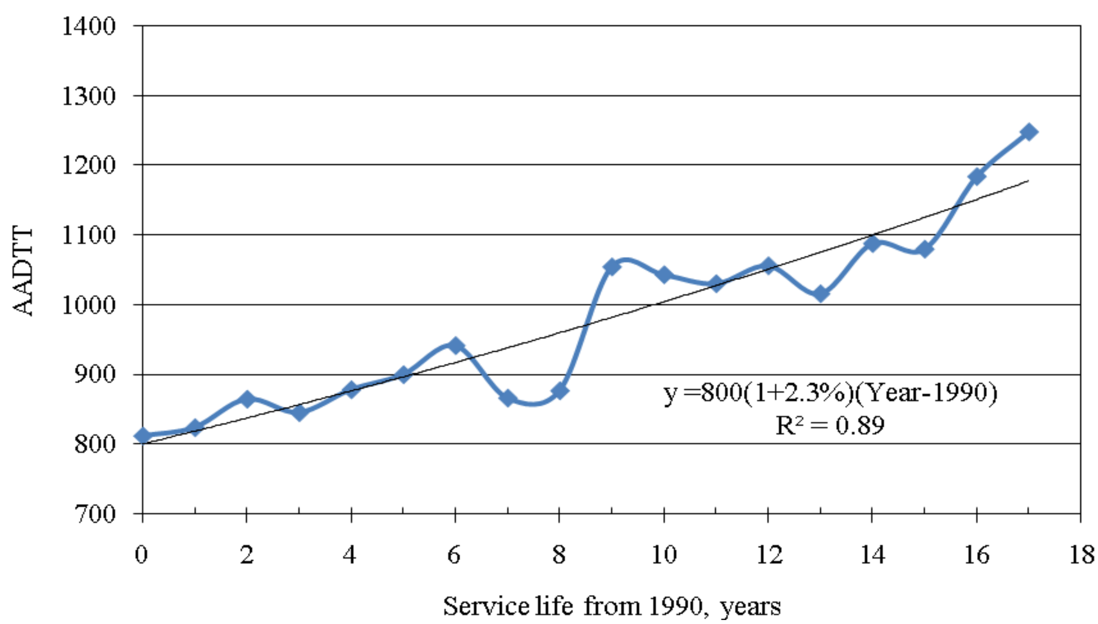


Figure 49. Graph. Regression analysis for the estimated truck traffic growth rate.

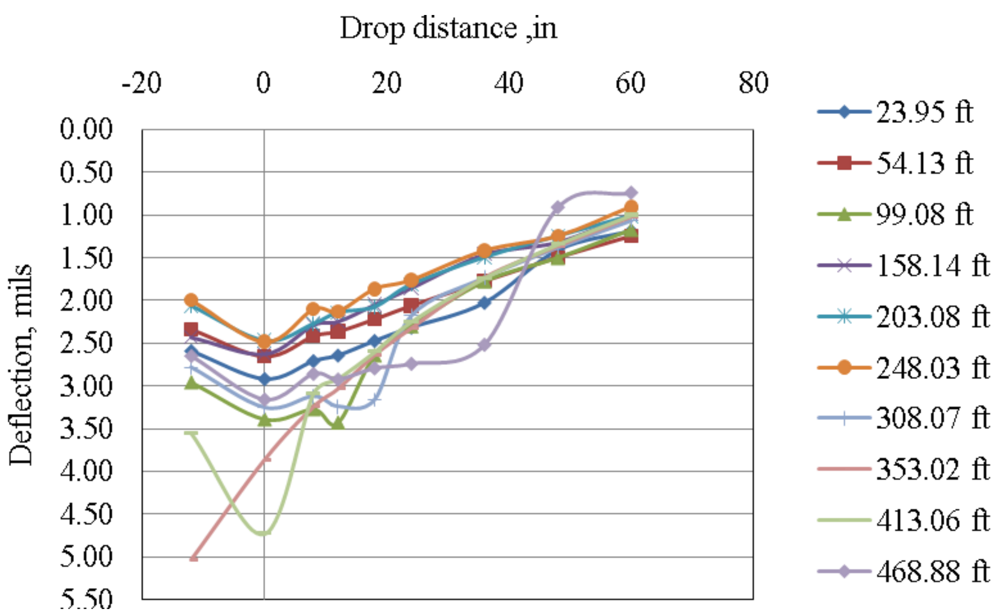
ANALYSIS AND INTERPRETATION OF FWD TESTING DATA

This section presents the data checks; backcalculation analysis of the FWD data, LTE, and void detection; as well as a comparison of the backcalculation results with laboratory testing.

Preprocessing of the Deflection Data

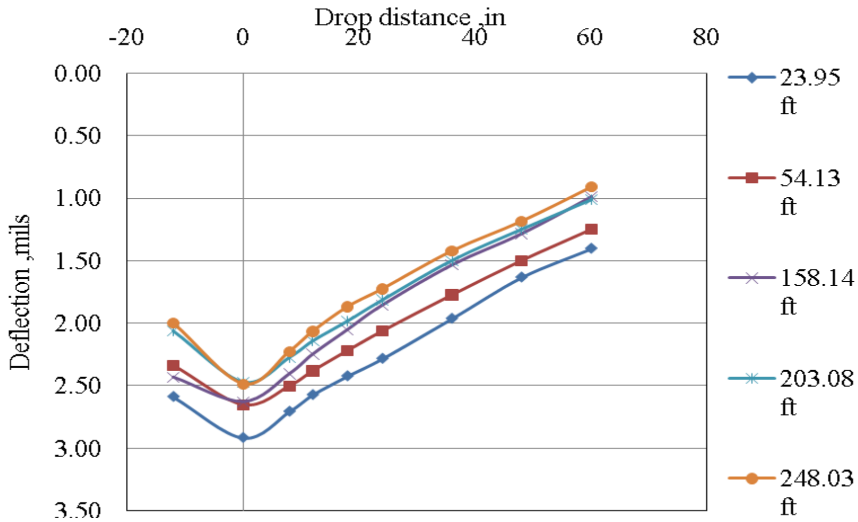
FWD testing on test section 32-0203 was performed on five separate occasions. These test dates were July 21, 1995; March 26, 1996; June 15, 2000; June 25, 2002; and October 28, 2003. To carry out the analysis representing the most current conditions, the FWD data collected on October 28, 2003, was selected for backcalculating the PCC elastic modulus and the modulus of subgrade reaction (*k*-value), assessing void potential beneath the slab corners, and characterizing the joint performance. Data from 1996 to 2002 were also used to analyze the variation in the load transfer efficiency (LTE) and the slab support conditions over time.

The average load and deflection of the recorded four drops for the 9,000-lb (4,086-kg) target load were used in the backcalculation of PCC elastic modulus and *k*-value. The first step in the data analysis was to plot the deflection basin based on the measured deflections for each test location. As illustrated in figure 50, the deflection basins for stations at 99.1, 308.1, 353, 413.1, and 468.9 ft (30.2, 94, 107.7, 126, and 143 m) did not present an acceptable profile because of the irregular relationship between their load positions and deflections. Therefore, the data from these test locations were not used in backcalculation. In addition, for the remaining stations (i.e., those at 24, 54.1, 158.1, 203.1, and 248 ft (7.3, 16.5, 48.2, 61.9, and 75.6 m)), the outlier data points were realigned by adjusting their position to obtain a smooth deflection basin. Figure 51 presents the adjusted deflection basins used in backcalculation.



1 mil = 0.0254 mm.
1 inch = 25.4 mm.

Figure 50. Graph. Original deflection basins for test section 32-0203.



1 mil = 0.0254 mm.
1 inch = 25.4 mm.

Figure 51. Graph. Corrected deflection basins for test section 32-0203.

Table 46 summarizes the loads and deflections used for backcalculation. The loads and deflections used to determine joint load transfer characteristics and the presence of voids are shown in table 47 and table 48, respectively.

Table 46. Loads and deflections used in backcalculation.

Station (ft)	Location	Drop Load (lb)	Deflection, mil							
			Sensor 1	Sensor 2	Sensor 3	Sensor 4	Sensor 5	Sensor 6	Sensor 7	Sensor 8
23.95	J1	9,212	2.91	2.70	2.64	2.47	2.31	2.02	1.40	1.18
54.13	J1	9,212	2.65	2.41	2.36	2.22	2.06	1.77	1.50	1.25
158.14	J1	9,116	2.62	2.28	2.24	2.05	1.85	1.46	1.31	0.98
203.08	J1	9,153	2.47	2.27	2.14	2.07	1.81	1.50	1.25	1.01
248.03	J1	9,142	2.48	2.10	2.13	1.86	1.76	1.42	1.25	0.91

1 ft = 0.305 m.
1 mil = 0.025 mm.
1 lb = 0.454 kg.

To graphically evaluate the deflection variation along the section, the deflections presented in table 46 were normalized to a standard 9,000-lb (4,086-kg) load and plotted against distance, as shown in figure 52. The deflections for stations at 99.1, 308.1, 353, 413.1, and 486.9 ft (30.2, 94, 107.7, 126, and 148.5 m) were not included, as discussed previously. As illustrated in figure 52, there was a variation in the structural response of the pavement along the section, with slightly higher deflections at the beginning of the section.

Table 47. Loads and deflections used to determine the joint load transfer characteristics of test section 32-0203.

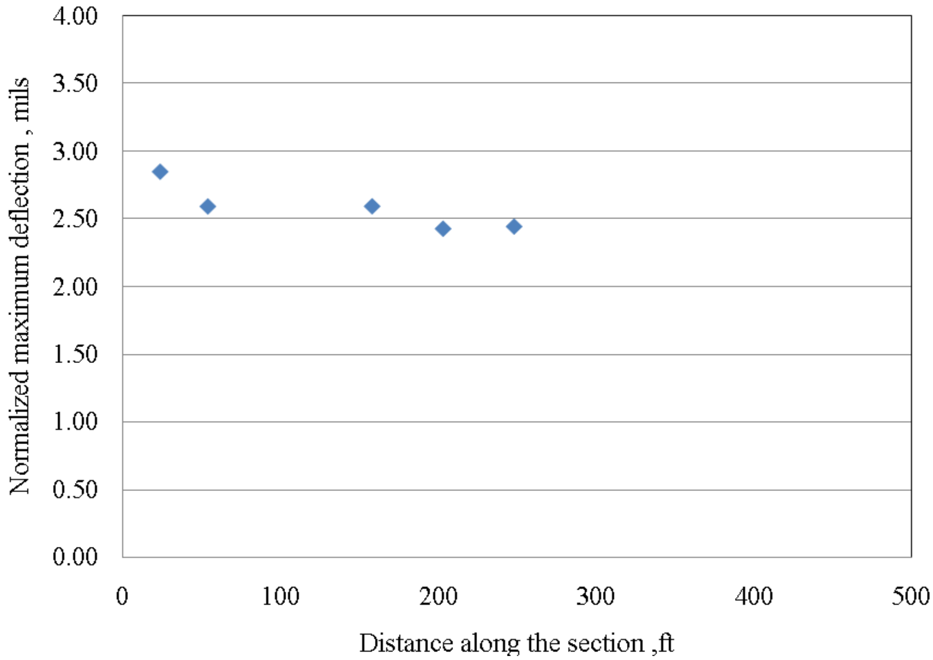
Station (ft)	Joint	Test Location	Drop Load (lb)	Deflection, mil		
				Sensor 9	Sensor 1	Sensor 2
4.9	1	J4	9,116	2.90	3.50	3.28
5.2	1	J5	9,069	3.04	3.31	2.89
14.3	2	J4	9,053	3.01	3.83	3.18
14.6	2	J5	9,079	2.99	3.56	3.07
28	3	J4	9,121	2.66	3.32	3.18
28.3	3	J5	9,185	2.85	3.24	2.80
46.3	4	J4	8,920	2.95	3.90	3.64
46.6	4	J5	8,989	3.28	3.79	3.22
60	5	J4	9,015	3.81	4.91	4.65
60.4	5	J5	8,978	4.42	4.92	4.21
73.8	6	J4	9,031	3.29	4.28	3.73
74.1	6	J5	9,068	3.43	4.09	3.49
92	7	J4	9,005	2.98	3.78	3.45
92.4	7	J5	8,983	3.31	3.86	3.31
105.8	8	J4	9,015	2.61	2.93	2.70
106.1	8	J5	9,037	2.62	2.87	2.65
124.1	9	J4	9,042	3.46	4.20	3.77
124.4	9	J5	9,100	3.66	6.94	3.58
146.9	10	J4	9,031	3.37	4.44	4.13
147.2	10	J5	9,031	3.95	4.42	3.66

1 ft = 0.305 m.
1 mil = 0.025 mm.
1 lb = 0.454 kg.

Table 48. Loads and deflections used for assessing void potential.

Station (ft)	Location	Drop Load (lb)	Sensor 1 Deflection (mil)
17.06	J2	9,212	4.27
17.06	J2	12,241	5.52
17.06	J2	15,657	7.11
47.9	J2	9,137	4.33
47.9	J2	12,177	5.68
47.9	J2	15,499	7.27
92.85	J2	9,110	3.75
92.85	J2	12,193	4.97
92.85	J2	15,727	6.47
152.89	J2	9,031	4.90
152.89	J2	12,077	6.35
152.89	J2	15,642	8.02
198.16	J2	8,989	5.47
198.16	J2	12,066	7.18
198.16	J2	15,689	9.17
243.11	J2	9,010	5.39
243.11	J2	12,045	6.89
243.11	J2	15,525	8.70
303.15	J2	9,031	4.45
303.15	J2	12,077	5.81
303.15	J2	15,409	7.43
348.1	J2	9,095	3.19
348.1	J2	12,098	4.23
348.1	J2	15,600	5.46
408.14	J2	9,090	5.25
408.14	J2	12,008	6.72
408.14	J2	15,345	8.48
482.94	J2	9,085	6.17
482.94	J2	12,040	7.81
482.94	J2	15,425	9.59

1 ft = 0.305 m.
1 mil = 0.025 mm.
1 lb = 0.454 kg.



1 ft = 0.305 m.
1 mil = 0.0254 mm.

Figure 52. Graph. Deflections along the section.

Backcalculation Analysis

Backcalculation of the test section 32-0203 deflection data was performed using the $AREA_{60}$ method (discussed in chapter 5, volume I) to determine the *k*-value and layer moduli. The analysis consisted of a two-layer system: PCC slab and spring foundation.

Backcalculation Results

Table 49 presents a summary of the calculated $AREA_{60}$ parameter as well as the calculated radius of relative stiffness for each station along the section. The dynamic *k*-value and PCC elastic modulus for each station are presented in table 50. The average static values are also summarized in table 50, assuming a dynamic-to-static correction of 0.5 for the *k*-value and 0.80 for the PCC elastic modulus.

Table 49. $AREA_{60}$ parameter and radius of relative stiffness.

Station (ft)	$AREA_{60}$	Radius of Relative Stiffness (inches)
23.95	43.88	38.27
54.13	43.84	38.18
158.14	39.78	31.03
203.08	41.11	33.11
248.03	38.94	29.83

1 ft = 0.305 m.
1 inch = 25.4 mm.

Table 50. Average dynamic and static PCC elastic modulus and *k*-value.

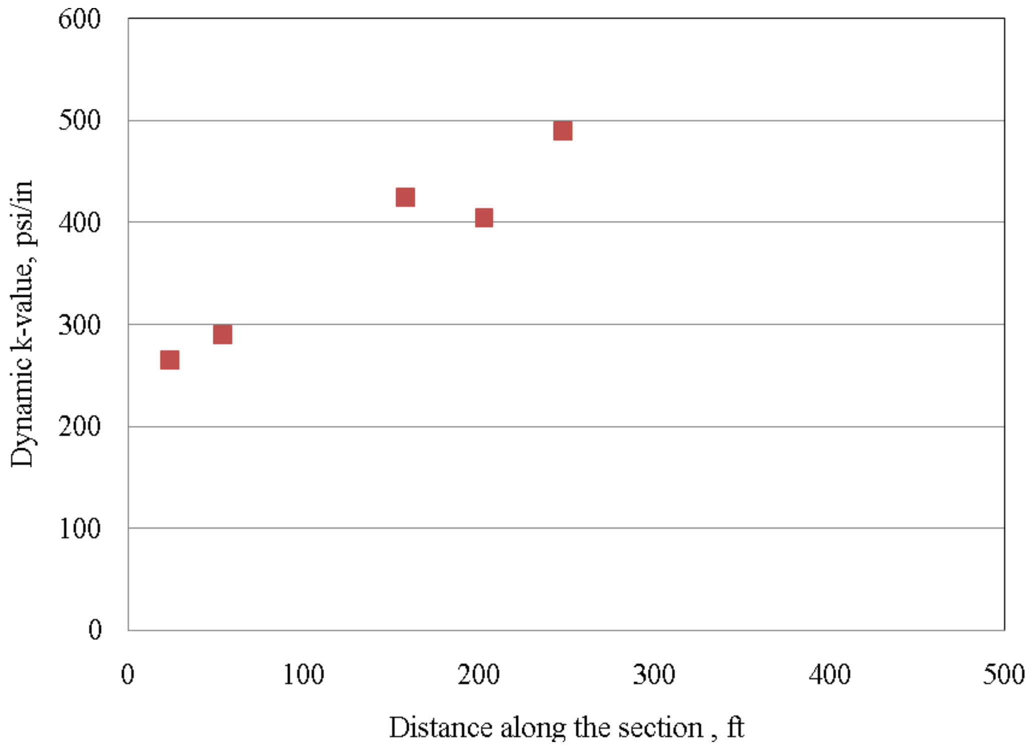
Station (ft)	Dynamic *k*-value (psi/inch)	Dynamic PCC Elastic Modulus (psi)
23.95	265	4,900,000
54.13	290	4,750,000
158.14	425	3,200,000
203.08	405	3,850,000
248.03	490	2,800,000
Average dynamic value	**375**	**3,900,000**
Average static value	**190**	**3,100,000**

1 ft = 0.305 m.
1 psi/inch = 0.263 kPa/mm.
1 psi = 6.89 kPa.

As shown in figure 53, the *k*-values along the section presented an increasing trend that may suggest that the stiffness of the section varies. As stated by Hall et al., a COV in backcalculated *k*-value less than 20 percent, after screening of outliers, is reasonable.[10] Significantly higher COVs would suggest significant changes in the subgrade soil type, the embankment thickness, or the depth to bedrock or water table. In this case, the COV of the backcalculated *k*-values was 25 percent, which suggests possible variations in subgrade support along the section. Because the subgrade of this section was not sampled, and there were no available data regarding the depth of the water table, it was not possible to definitely ascertain the causes of the variation in the structural response of the pavement.

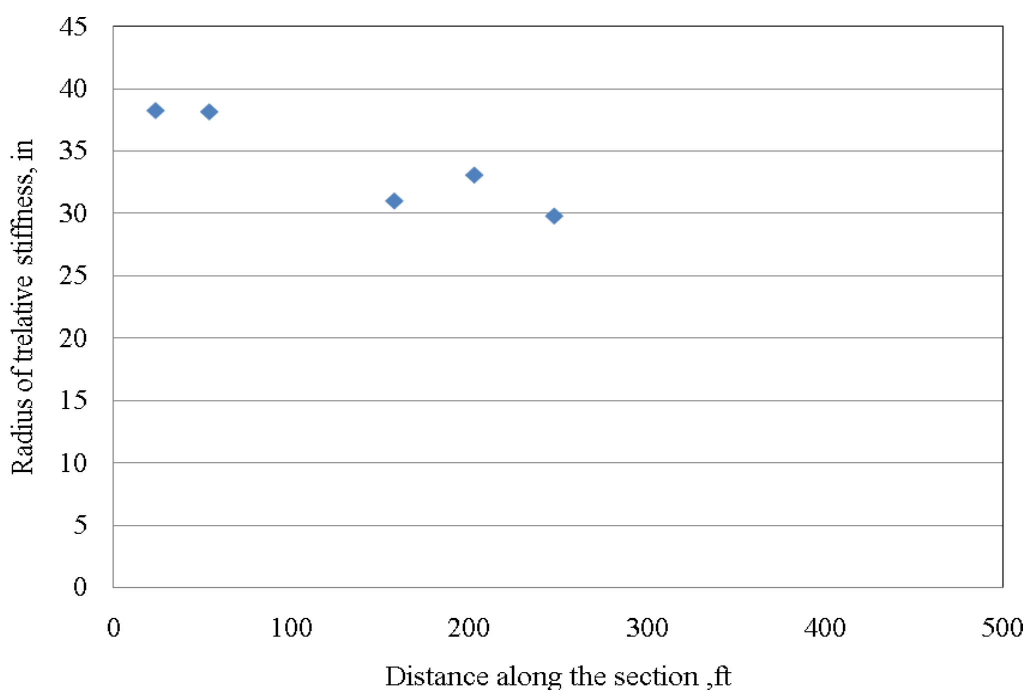
The variation of the radius of relative stiffness along the section, which is the relative stiffness of the slab to that of the pavement foundation, is presented in figure 54. For this relationship, the COV for this parameter was close to 12 percent, which corresponded to the large COV found for the backcalculated PCC elastic moduli and the *k*-values along the section.

The variation of PCC elastic moduli along the section is depicted in figure 55. According to the values presented in this figure, the COV for this property was around 22 percent, which was higher than the 15 percent that was typically assumed acceptable for a concrete mixture design. Moreover, the research team also found that the high backcalculated *k*-values always correlated to low PCC elastic moduli. This could be an indication that the backcalculation technique was capturing the overall stiffness but it was overestimating the *k*-value and, therefore, the elastic modulus of the PCC was being underestimated.



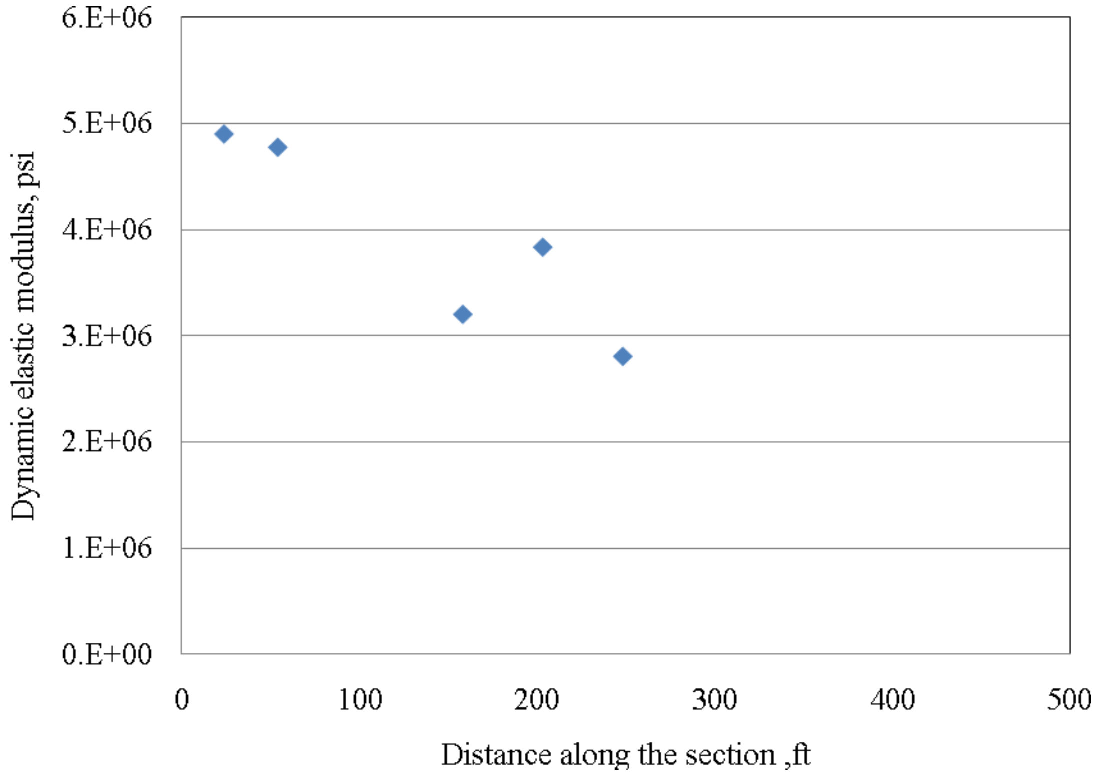
1 ft = 0.305 m.
1 psi/inch = 0.263 kPa/mm.

Figure 53. Graph. *k*-values along the section.



1 inch = 25.4 mm.
1 ft = 0.305 m.

Figure 54. Graph. Radius of relative stiffness along the section.



1 psi = 6.89 kPa.
1 ft = 0.305 m.

Figure 55. Graph. PCC elastic modulus along the section.

Joint Load Transfer

Joint LTE and differential slab deflections were determined using the equations presented in chapter 5, volume I. Table 51 shows the lowest of the calculated LTEs between approach and leave side, and the differential deflections based on the deflections presented in table 47. As shown in table 51, most of the LTE values were above the acceptable level of 75 percent, except at joint 9, which exhibited only a fair LTE (between 50 and 75 percent). The fact that all of the differential deflections were less than 0.25 mm (10 mil), as presented in table 51, supports the fact that the joints were performing well. The deflection data used for determining the LTEs was collected at higher temperatures than recommended (typically 70 ºF (21 ºC)), which may contribute to joint lock-up. In a subsequent section of this report, it is shown that voids were not found to be present beneath the slabs, indicating that good load transfer was being provided for the majority of the time even in the presence of colder temperatures.

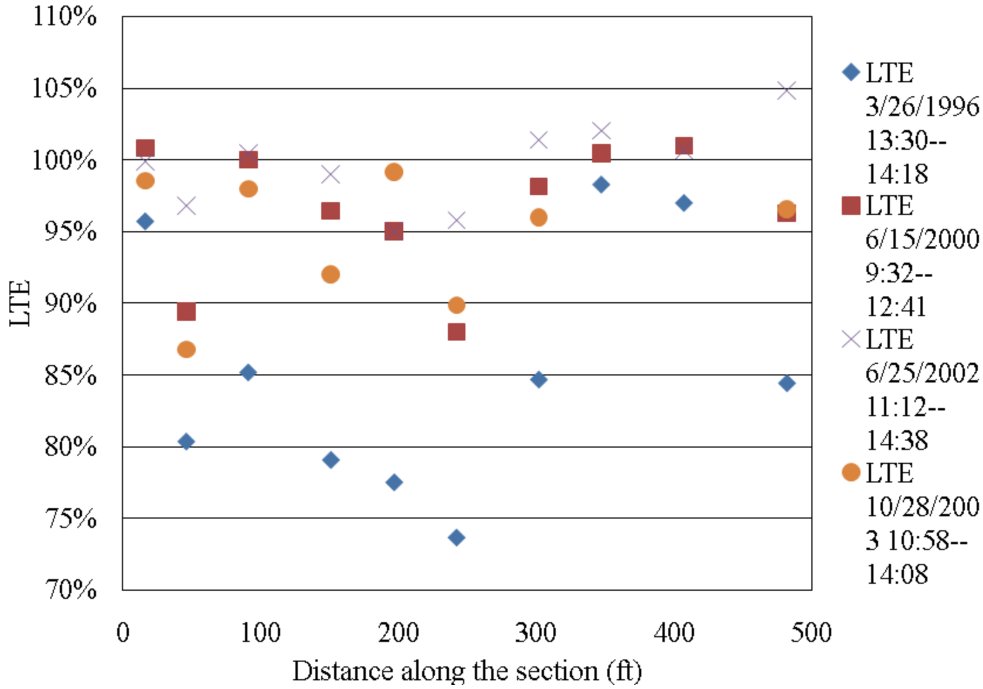
Table 51. LTEs and differential deflections for the approach and leave slabs.

Joint	LTE (percent)	Approach Slab Differential Deflections (mil)	Leave Slab Differential Deflections (mil)
1	99	-0.4035	0.6004
2	87	-0.8465	0.8268
3	98	-0.3248	0.6594
4	92	-0.6791	0.9449
5	99	-0.5413	1.1024
6	90	-0.8268	0.9843
7	96	-0.4921	0.7972
8	100	-0.2854	0.3051
9	59	-0.5906	0.7283
10	97	-0.5906	1.0630

1 mil = 0.0254 mm.

Variation in LTE Over Time

Figure 56 shows the LTEs from the available data between 1996 and 2003. As shown in the figure, the LTE range was different for each year. For instance, the range of LTEs for the testing performed in 1996 was from 75 to 100 percent, whereas for the FWD testing performed in 2002, the range of LTE was between 95 and 104 percent. This variation was related to the slab temperature at the time of testing. At warmer temperatures, slabs expand causing the joints to lock up. Therefore, a higher LTE is obtained. This effect is shown in figure 56. The lowest LTE was observed in 1996 when the FWD testing was performed during the spring (late March), and the highest LTEs were observed in 2002 when the testing was performed during the summer (late June). In addition, figure 56 shows that the trend of the variations in the LTE along the project was maintained for all test periods. This indicates that the performance level of the joints along the section remained relatively constant over time. If the LTE was low, repair of the joints would need to be performed.



1 ft = 0.305 m.

Figure 56. Graph. LTE values from 1996 to 2003.

Void Detection

The presence of voids is determined based on the fact that pavement deflections respond nonlinearly when voids exist. The data presented previously in table 48 was used to plot edge deflections against the applied load for each station to assess the potential for voids in the method proposed by Crovetti and Darter.[11] The best-fit line of the data points should go through the origin if the response of the pavement is linear. When voids are present, a deflection occurs with a minimal applied load until the slab comes in complete contact with the base layer. Therefore, the presence of voids causes an upward shifting of the plot of load versus deflection, and the positive intercept on the y-axis determines the relative size of the voids. An intercept of the y-axis greater than 2 mil (0.05 mm) may be an indication of the existence of voids and the magnitude of the y-intersect determines the relative size of the voids. If voids are present, repairs would be needed as part of the rehabilitation.

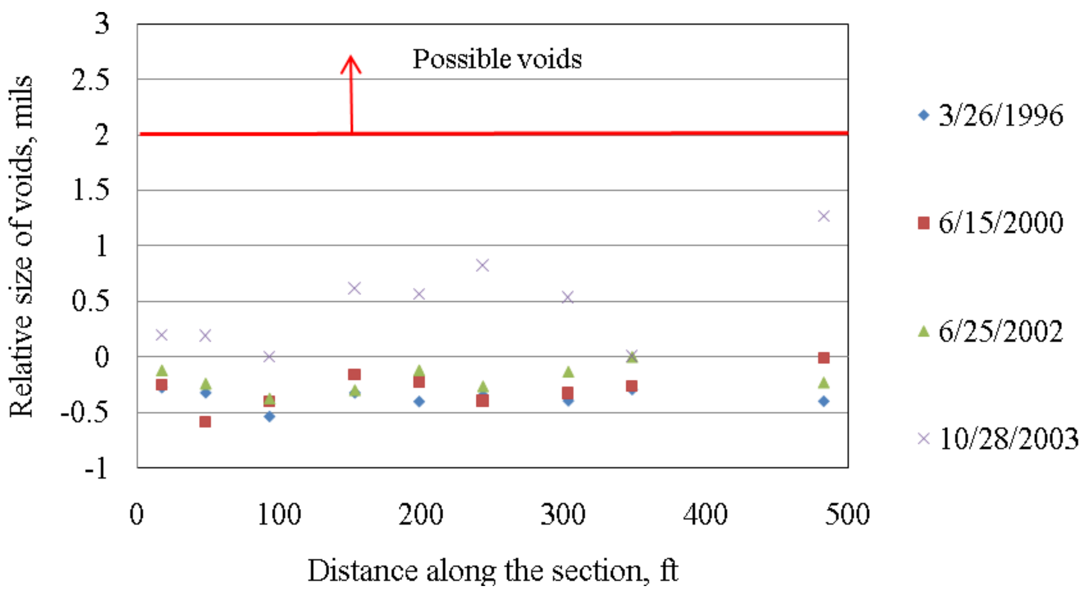
The temperature distribution throughout the depth of the slab at the time the FWD testing is performed can be critical when using the deflection data for void detection. If a positive temperature gradient is present in the slab, then the corners of the slab will curl downward so that erosion of the base might have occurred while not being detected. If a negative gradient is present causing the slab corners to curl upward, erosion of the base might be detected even it has not occurred. To reduce the potential for false positive or false negative results, it is recommended that FWD testing be performed when the slabs are flat. The research team found that the slabs were most likely to be flat when a positive temperature gradient was present in the slabs due to the existence of built-in gradients.[12] As shown in figure 40 through figure 43, a positive temperature gradient was present in the slabs during the FWD testing. The extent of these gradients, which were assumed linear, is presented in table 52. As shown this table, these gradients presented similar magnitudes for the 1996, 2000, and 2003 tests. The temperature gradient of the slab for the 2002 test was about 1 °F (0.556 °C) higher than the rest of the tests, implying the presence of a more pronounced downward curvature of the slab at the time the FWD test was conducted.

Table 52. Slab temperatures and gradients for the FWD test performed on test section 32-0203.

FWD Test Date	Average Temperature Gradient Throughout the Test Day (ºF/inch)	Average Slab Temperature (ºF)
3/26/1996	1.27	58
6/15/2000	1.50	87
6/25/2002	2.21	96
10/28/2003	1.54	71

1 °F/inch = 0.041 °C/cm
°F = 1.8 × °C + 32.

The intercept values from the load versus deflection plots for the FWD tests performed in test section 32-0203 are summarized in figure 57. As shown in this figure, the y-intercepts for all the sections were negative values before 2003, and only the test in 2003 indicated the possible existence of loss of support. However, because the magnitudes of the y-intercepts for the 2003 test are smaller than 2 mil (0.05 mm), the existence of voids along the section was not probable at that moment. On the other hand, the observed trend from 1996 to 2003 may indicate a loss of support process that apparently started between 2002 and 2003 and was likely to develop with time.



1 mil = 0.0254 mm.
1 ft = 0.305 m.

Figure 57. Graph. Y-intercepts from void detection test results.

Comparison of Backcalculation and Laboratory Testing Results

Determination of individual layer properties from the composite *k*-value is a difficult task. The backcalculation only provided the elastic modulus for the PCC layer and the composite *k*-value representing the stiffness of all the underlying layers (i.e., base, subbase and subgrade). On the other hand, the laboratory measurements are usually obtained for each individual layer. Efforts were made to break down the composite *k*-value obtained in the backcalculation to provide the basis for the comparison. One possible solution was to use the method proposed in the Portland Cement Association (PCA) guide for pavement design to account for the increase in the static *k*-value owing to the use of a base/subbase.(13) Another possible solution could be obtained using layered elastic analysis to match the measured deflections—similar to the conversion for new rigid pavement design within the MEPDG software. However, this approach was not investigated for this case study.

Bound Materials

As presented in table 50, the average backcalculated static PCC elastic modulus was 3.1 million psi (21,359,000 kPa) whereas, according to the LTPP Program data, the average laboratory-measured static modulus is 2.8 million psi (19,292,000 kPa). The difference was insignificant when considering that the measured value was only 0.46 standard deviations away from the mean of the backcalculated values. However, this apparently good correlation was due to the relatively high standard deviation of the backcalculated values 750,000 psi (5,000 kPa). As stated before, there was variation between the backcalculated PCC elastic modulus along the project, and it might suggest that the backcalculation technique was capturing the overall stiffness while overestimating the *k*-value and, therefore, underestimating the elastic modulus of the PCC. The average static elastic modulus for the first portion of the section (station 0 ft (0 m) to station 54.1 ft (16.5 m)) was 3.9 million psi (26,871,000 kPa), whereas this value for the remaining of the section was 2.6 million psi (17,914,000 kPa)). Therefore, the laboratory measured values agreed well with the backcalculated values for the second portion of the section, but relatively poorly with the values obtained for the first portion.

Unbound Materials

The backcalculation procedures for rigid pavements provide a *k*-value, whereas the laboratory testing of material samples results in the determination of a resilient modulus value. One available correlation for *k*-value and resilient modulus is shown in figure 58.(2)

$$M_R = 26(k\text{-}value)^{1.284}$$

Figure 58. Equation. Correlation for *k*-value and resilient modulus.

Where:

M_R = Resilient modulus, psi.
k-value = Modulus of subgrade support, psi.

As discussed previously, the *k*-value determined using the $AREA_{60}$ backcalculation method is a dynamic *k*-value and includes the contribution of the unbound base and subbase layers as well as the subgrade (and bedrock, if present). A factor of 0.5 is commonly applied to convert a dynamic *k*-value to a static *k*-value. So, for this section the dynamic *k*-value corresponded to a static composite *k*-value of approximately 52 kPa/mm (190 psi/inch), and a resilient modulus of 21,900 psi (151,000 kPa). These values are considered somewhat low for a composite material that includes the effects of both the subgrade and the aggregate base and subbase materials.

The PCA pavement design guide was used to account for the increase in the static *k*-value due to the use of a base/subbase.[13] Based on table 53, a composite static *k*-value of 190 psi/inch (51 kPa/mm) (0.5 times the backcalculated dynamic *k*-value of 375 psi (2,584 kPa)) was contributed by a subgrade that has a static *k*-value of 155 psi/inch (42 kPa/mm) and a 6-inch (152-mm)-thick subbase. This suggests a resilient modulus of 16,900 psi (116,300 kPa) for the material below the base layer, which is relatively low considering the presence of subbase aggregate and a lime-stabilized subgrade layer. A modulus value for the base layer would then need to be assumed, as discussed later.

Table 53. Effect of untreated subbase on *k*-values.[13]

Subgrade *k*-value (psi/inch)	Subbase *k*-value (psi/inch)		
	4-Inch Subbase	6-Inch Subbase	9-Inch Subbase
50	65	75	85
100	130	140	160
200	220	230	270
300	320	330	370

1 psi/inch = 0.263 kPa/mm.
1 inch = 25.4 mm.

The resilient modulus of the subgrade ranged from 6,000 to 15,000 psi (41,340 to 103,350 kPa) for a bulk stress between 8 to 28 psi (55 to 193 kPa), as previously shown in figure 44. Considering the fact that resilient modulus testing was not performed on any samples from test section 32-0203 and recognizing the difficulty in determining the resilient modulus because of the unknown confinement condition, the typical resilient modulus suggested by the MEPDG for AASHTO A-4 subgrade, 15,000 psi (103,350 kPa) was adopted. This is much lower than the composite backcalculated *k*-value, but only slightly lower than the estimated separate subgrade modulus.

With respect to the 5.7-inch (145-mm) granular base layer, although resilient modulus laboratory testing was performed for three sections (figure 48), the stress-dependent feature of this property makes it hard to determine, especially when the confinement in the field is unknown. Therefore, the resilient modulus suggested by the MEPDG, 25,000 psi (172,250 kPa), is used for the base, which corresponds to a bulk stress between 50 and 60 psi (345 and 413 kPa) according to figure 48. The granular base layer was determined to be A-1-a in the AASHTO system based on its gradation.[2] The resilient modulus was then determined using a correlation between the soil's AASHTO classification and the typical range of resilient modulus.[2,13] The typical resilient modulus of AASHTO A-1-a soil was between 22,000 and 28,000 psi (151,580 and 192,920 kPa), which agreed well with the default value suggested in MEPDG for crushed gravel. This value was slightly higher than the backcalculated composite *k*-value, but was likely reasonable based on the aggregate layer being thin relative to the overall subgrade thickness.

The composite *k*-value can be obtained based on the modulus that was determined previously for each layer and for comparison with the composite *k*-value from the backcalculation. In the MEPDG (table 3.6.8), adjustment factors are provided between the backcalculated and laboratory-determined elastic modulus; for this study, values of 1.32 and 0.35 were used for the base/subbase and subgrade, respectively. Therefore, the laboratory-determined modulus as described previously (i.e., 15,000 psi (103,350 kPa) for the subgrade); 25,000 psi (172,250 kPa) for the base, can be converted to the backcalculated modulus, namely 5,250 psi (36,170 kPa) for the subgrade and 33,000 psi (227,370 kPa) for the base. The backcalculated modulus was then used to compute the composite *k*-value based on figure 3.3. in the 1993 AASHTO *Guide for Design of Pavement Structures*.[2] The result was approximately 300 psi/inch (80 kPa/mm) for a base with 5.7-inch (145-mm) thickness, which was much higher than the composite *k*-value from the backcalculation (i.e., 190 psi/inch (50 kPa/mm)). This again verifies that the backcalculated modulus of the unbound materials tends to be smaller than the laboratory-determined modulus for the studied pavement section.

RECOMMENDED REHABILITATION DESIGN INPUTS

The objective of this study was to learn the reliability of using FWD backcalculated values in design based on the comparison between designs with inputs from laboratory tests and designs with inputs obtained from FWD backcalculation. Rehabilitation designs were analyzed for an HMA overlay, an unbonded JPCP overlay, and a bonded JPCP overlay for this section. Designs were determined using version 1.003 of the MEPDG software and the nationally calibrated performance models.[1]

Design Criteria and General Inputs

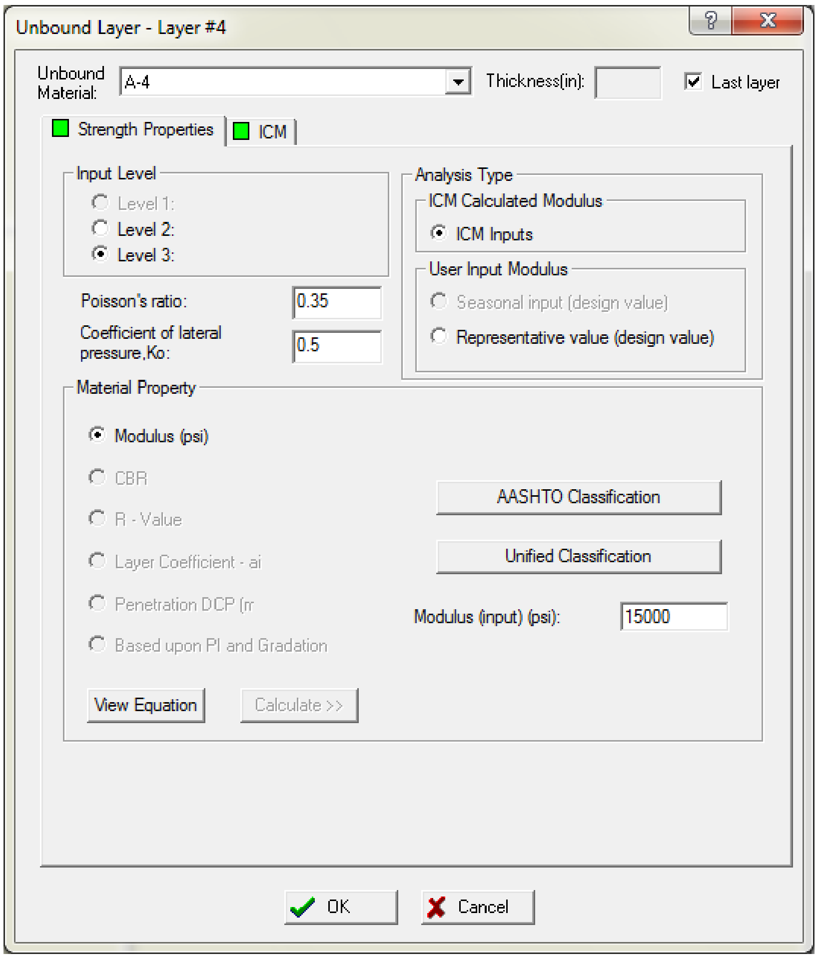
The general inputs used for the design program were based on the available LTPP Program data (as previously summarized), estimated inputs from standard specifications, and on default values within the program when data were not available from other sources. The primary inputs that were evaluated for this case study were those corresponding to properties obtained from the backcalculation process (PCC elastic modulus and *k*-value).

The rehabilitation design also considered the percentage of cracked slabs. As discussed in the previous section, all of the pavement slabs had cracked by 2003 (see table 37). In terms of the MEPDG design, the percentage of cracked slabs before restoration was then assumed to be 100 percent. Because almost all the transverse cracks are working cracks, they should all have been repaired before the placement of the overlay. Therefore, the percentage of cracked slabs before the overlay was assumed to be 0 percent.

As mentioned previously, the top 12 inches (305 mm) of the subgrade was treated with lime. In the MEPDG, there is no lime stabilized subgrade, but there is a lime stabilized base. The use of a lime-treated base layer between the subbase and the subgrade, in an effort to reflect the top 12 inches (305 mm) of lime-treated subgrade, caused an error displayed as “Failed the ICM stability check,” which may be related to the much larger stiffness of a lime stabilized base layer compared with that of a lime stabilized subgrade. A second effort was then made by designating the subgrade as AASHTO A-2-4 soil instead of AASHTO A-4 soil to reflect the improvement of the subgrade resulting from the treatment. Furthermore, the AASHTO A-2-4 subgrade

(16,000 psi (110,240 kPa)) might be combined with the aggregate subbase (assumed as the base, 25,000 psi (172,250 kPa)), the weighted average can be used as the modulus for the subgrade (approximately, 20,000 psi (137,800 kPa)). However, no difference was found in terms of the design thickness of the overlay resulting from this change in the modulus of the subgrade. Therefore, the lime treatment top 12 inches (305 mm) of the subgrade was neglected in this study and the subgrade remains AASHTO A-4 soil, which is a conservative approach. Other possible modeling considerations (such as adjusting the subbase thickness and modulus) were not pursued because the subgrade appeared to have no influence on the design thickness.

The climatic modeling (see figure 59) was used for the unbound layers. While a "Representative value" was also allowed, testing data and backcalculation results were not available for all of the months or seasons to estimate an appropriate representative value.



1 inch = 25.4 mm.
1 psi = 6.89 kPa.

Figure 59. Screen Capture. Unbound layer input screen illustrating ICM selection.

Inputs for the HMA Overlay Rehabilitation

The overall design level for an HMA overlay over JPCP is level 3 and cannot be changed. The input level for the individual layers can be adjusted, with levels 1 through 3 for the HMA overlay and existing PCC, and levels 2 and 3 for unbound materials. For the HMA overlay analysis, the flexural strength and elastic modulus were entered for the existing PCC as required in level 1, and the unbound layer data were entered as level 3.

From the design runs performed during this case study, the HMA overlay design appeared to be very sensitive to the HMA properties of the overlay. The results presented later in this chapter are based on the following HMA mixture properties. The asphalt binder grade was selected based on the Superpave performance grading system using LTPPBIND Version 2.1 that was developed for the Federal Highway Administration in 1999. The Superpave PG obtained for the specific location of test section 32-0203 was 58-22 with a reliability of about 87 percent. The aggregate gradation, the effective binder content, and the percentage of air voids were extracted from a Strategic Highway Research Program (SHRP) level-1 mix design that had been used in the MnROAD HMA cells.[14] The effective binder content was 5.6 percent, the air voids were 4 percent, and the aggregate gradation was presented in table 54.

Table 54. Aggregate gradation for the additional HMA overlay alternative design.

Sieve Size	Value
$^{3}/_{4}$ inch, percent retained	0
$^{3}/_{8}$ inch, percent retained	18
No. 4, percent retained	33
No. 200, percent passing	4

1 inch = 25.4 cm.

The performance criteria for the HMA overlay suggested by the MEPDG are presented in table 55. A design reliability of 90 percent was used and a 20-year performance period.

Table 55. Performance criteria for the 20-year HMA design.

Performance Criteria	Limit
Initial IRI, inches/mi	63
Terminal IRI, inches/mi	172
Transverse cracking, percent slabs cracked	15
AC surface down cracking (long. cracking), ft/mi	2,000
AC bottom up cracking (alligator cracking), percent	25
AC thermal fracture (transverse cracking), ft/mi	1,000
Chemically stabilized layer (fatigue fracture)	25
Permanent deformation (AC only), inches	0.25
Permanent deformation (total pavement), inches	0.75
Reflective cracking, percent	100

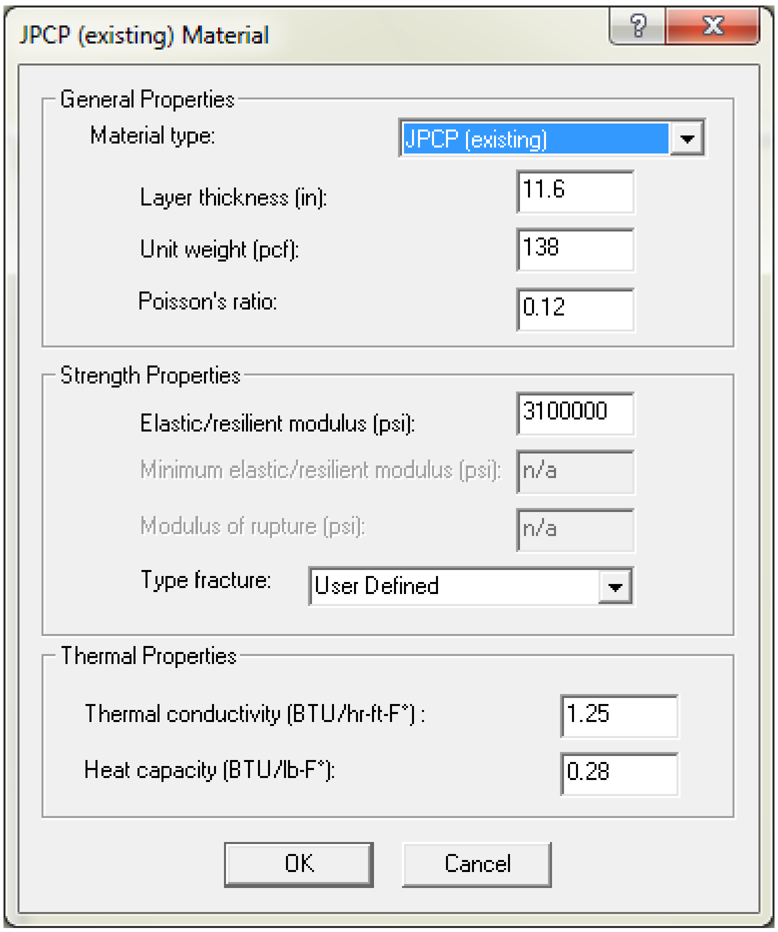
1 inch/mi = 0.0158 m/km.
1ft/mi = 0.19 m/km.
1 inch = 25.4 mm.
AC = asphalt concrete.

In the HMA overlay design, the modulus of rupture for the existing PCC layer is also required in the MEPDG. It can be obtained using the correlation between the PCC elastic modulus and the modulus of rupture provided within the MEPDG. Based on this correlation, a modulus of rupture of 3,790 kPa (550 psi) is obtained for an elastic modulus of 21,359,000 kPa (3.1 million psi).

Inputs for Unbonded JPCP Overlay Rehabilitation

The same PCC properties as the existing PCC layer are used for the JPCP unbonded overlay design, which were as follows: unit weight of 138 lb/ft^3 (2,211 kg/m^3), Poisson's ratio of 0.12, CTE of 5.3×10^{-6}/°F (9.5 ×10^{-6}/ºC), and a modulus of rupture of 570 psi (3,927 kPa). The PCC mix design used for the existing PCC layer as shown in table 44 was also used to remain consistent with these properties.

The existing PCC was not being rubblized or crack-and-seated, so the fracture type was left as "User Defined" (see figure 60). In addition, the existing PCC layer was fixed as a level 3 input in an unbonded PCC overlay design.



1 inch = 25.4 mm.
1 pcf = 16 kg/m^3.
1 psi = 6.89 kPa.
1 BTU/h-ft-°F = 1.73 W/m-°C.
1 BTU/lb-°F = 4,186 J/kg-°C.

Figure 60. Screen Capture. Existing PCC layer input screen for unbonded PCC overlay.

The PCC slabs were designed as 15 ft (4.6 m) long and 14 ft (4.3 m) wide to be compatible with the existing PCC layer and the joints were sealed with silicon and doweled with 1.5-inch (38-mm) bars. With respect to the required bond breaker layer (2 inches (51 mm) thick as suggested by the MEPDG), the same HMA was employed as was used in the HMA overlay design.

The performance criteria for the unbonded JPCP overlay suggested by the MEPDG are presented in table 56. A reliability level of 90 percent is used.

Table 56. Performance criteria for the 20-year unbonded JPCP design.

Performance Criteria	Limit
Initial IRI (inches/mi)	63
Terminal IRI (inches/mi)	172
Transverse cracking (percent slabs cracked)	15
Mean joint faulting (inches)	0.12

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.

Inputs for Bonded JPCP Overlay Rehabilitation

The inputs, as well as the performance criteria for the JPCP bonded overlay, were the same inputs as those used for the unbounded JPCP overlay with the difference that the bonded overlay did not include an asphalt interlayer. A bonded JPCP overlay also required the existing PCC modulus of rupture, similar to the HMA overlay design. In addition, the input level for the existing PCC layer can be based on levels 1 through 3 for a bonded PCC overlay.

REHABILITATION DESIGN RESULTS

To validate the reliability of using FWD backcalculation results in MEPDG designs, two design alternatives were originally planned for each kind of overlay, the first of which would employ the laboratory testing results as inputs, and the second would employ the backcalculated values as inputs. However, the research team later found (from a study on the conversion of layer moduli into *k*-value, which is part of this report) that the MEPDG software did not always use the entered dynamic *k*-value to represent the stiffness of all the layers beneath the base layer as is stated in the MEPDG documentation. Sometimes, the stiffness of the base layer is also taken into account in the composite *k*-value. Therefore, a third alternative had to be designed to learn how the MEPDG uses the input *k*-value. The details of the three design alternatives are presented in the following subsections.

Alternative 1

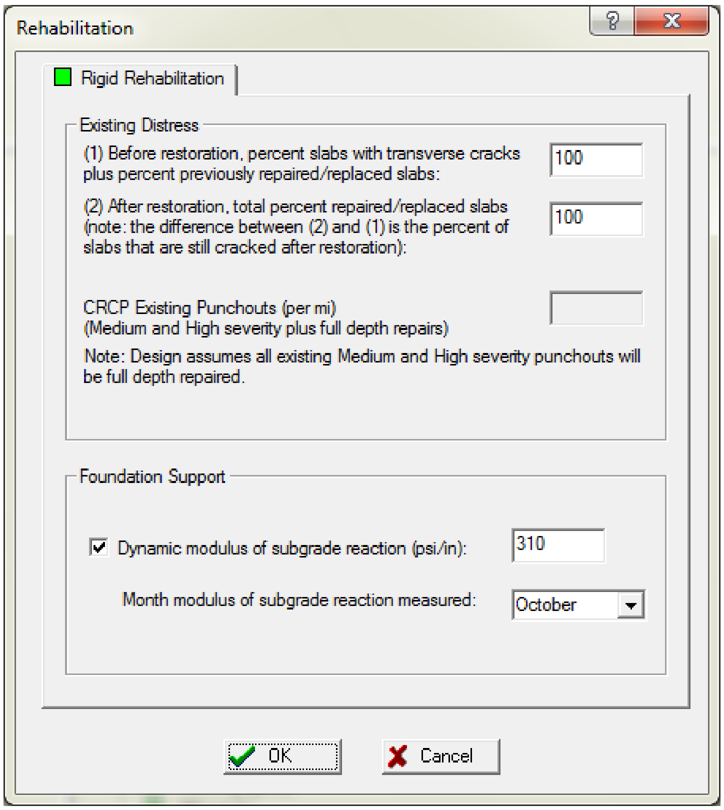
The first design alternative served as a benchmark, where the measured or recommended material properties were used as inputs in the MEPDG. For this alternative, the backcalculated dynamic *k*-value was not used to represent the stiffness of the supporting layers.

Alternative 2

In the second design alternative, the backcalculated values, in terms of the dynamic elastic modulus for the existing PCC layer and the dynamic *k*-value for the supporting layers, were used as inputs, assuming that the *k*-value reflected the stiffness of all the layers beneath the base.

Based on the FWD testing, the average backcalculated elastic modulus for the existing PCC layer was 3.1 million psi (21,359,000 kPa) (see table 50).

The average backcalculated dynamic *k*-value is 375 psi/inch (101 kPa/mm). However, this *k*-value was selected to represent the composite stiffness of all the layers beneath the PCC layer. As discussed previously, using the adjustment method in the PCA pavement design guide, an individual dynamic *k*-value for the subgrade can be estimated.[13] Therefore, for the second alternative, the static *k*-value of all layers beneath the 145-mm (5.7-inch) base was estimated to be 155 psi (1,068 kPa) for test section 32-0203. Thus, the dynamic *k*-value for all the layers beneath the base is 310 psi/inch (84 kPa/mm) and is entered in the "Rehabilitation" screen, as shown in figure 61. The rest of the inputs besides the dynamic elastic modulus for the existing PCC layer and the dynamic *k*-value for all layers beneath the base were kept the same as those in the first alternative.



1 inch = 25.4 mm.
1 psi = 6.89 kPa.

Figure 61. Screen Capture. Dynamic *k*-value input for overlay design options.

Alternative 3

For the last alternative, the same inputs as those used in the second alternative were used, except that the dynamic *k*-value was assumed to be 375 psi/inch (101 kPa/mm). It was assumed that this reflected the composite stiffness of all layers beneath the slab. If the stiffness of the base layer were included in the *k*-value by the MEPDG, different design results may be expected for alternative 2 and 3.

Design Results for the HMA Overlay

In the design, the thickness of the overlay was varied until the thinnest HMA layer was obtained that still satisfied the performance criteria as shown in table 55. HMA overlays that were 12 inches (305 mm) thick were found capable of carrying the estimated future traffic, regardless of the different sets of inputs used, as summarized in table 57.

Table 57. Design thickness of HMA overlay for three alternatives.

Alternative	Required Thickness (inches)
1	12.0
2	12.0
3	12.0

1 inch = 25.4 mm.

The obtained design thickness was significantly greater than what would be typically constructed by a highway agency. The controlling distress predicted by the MEPDG for this HMA overlay design was rutting, which was both load- and material-related distresses. This implies that changes in the HMA mix design would substantially affect the design thickness of the HMA overlay. If rutting was not considered (such as assuming the rutting would be patched at some intermediate year), the required HMA overlay thickness became less than 4 inches (100 mm).

The research team noticed that during the design analyses, the *k*-values reported in the design output files were the same, whether or not the dynamic *k*-value was entered as an input. Therefore, additional design runs were executed to assess how the *k*-value was being calculated. Based on the design runs, varying the subbase stiffness had very little influence on the determined *k*-value, which seemed to suggest the stiffness of the subbase layer was not taken into account in the calculated *k*-value. In addition, the reported *k*-values in the output file were identical for cases with varying base layer stiffness, indicating the stiffness of the base was likely not included in the *k*-value calculation. Furthermore, it appears that the MEPDG ignored the entered dynamic *k*-value and calculated the *k*-values based on the entered layer moduli because the summarized values were the same regardless of what dynamic *k*-value was entered or when a dynamic *k*-value was not entered.

Design Results for the Unbonded PCC Overlay

The design thickness of the unbonded JPCP overlay was the thinnest one that still satisfied the performance criteria shown in table 56. However, the use of unbonded JPCP overlays thinner than 178 mm (7 inches) is not recommended by the MEPDG. This is because the unbonded overlay acts independently from the supporting layers, so a minimum allowable thickness is needed to ensure adequate structural capacity. The research team found that a 7-inch (178-mm)-thick or thicker JPCP unbounded overlay can meet all criteria for the three alternatives, so the design thickness is determined to be 7 inches (178 mm) for each of the three alternatives, as shown in table 58.

Table 58. Design thickness of unbonded PCC overlay for three design alternatives.

Alternative	Required Thickness (inches)
1	7
2	7
3	7

1 inch = 25.4 mm.

As with the HMA overlay design, additional analyses were performed to assess how the *k*-values are calculated within the design program. The output file *k*-values for a stiff and less stiff asphalt interlayer were only approximately 10 percent different. The difference in *k*-values between a low and high existing PCC elastic modulus are also approximately 10 percent different. These results seem to indicate that the stiffness of the interlayer and the existing PCC are not considered in the calculation of the *k*-value.

The research team concluded that the base layer was taken into account by the difference in *k*-values when using a stiff and weak base layer. In addition, the *k*-values agreed well with the entered dynamic *k*-value, suggesting that the MEPDG uses the entered value for unbonded PCC overlay designs. However, when no *k*-value was designated, it was not explicit which layers were taken into account in the calculation of the *k*-value.

Design Results for the Bonded PCC Overlay

The design thickness of the bonded JPCP overlay was determined by reducing the thickness of the overlay until the performance criteria shown in table 56 were no longer fulfilled. As presented in table 59, the design thickness for all the alternatives was 4 inches (100 mm). Thinner slabs would fail due to high IRI, while little cracking and a small amount of faulting occurred. The high IRI was not contributed mainly by distresses, but possibly some other site factors such as freezing index and fines in the supporting layers.

Table 59. Design thickness of bonded PCC overlay for three design alternatives.

Alternative	Required Thickness (inches)
1	4)
2	4
3	4

1 inch = 25.4 mm.

The *k*-values in the design output for a high and low base layer modulus had noticeable variation. Therefore, it seems that the modulus of the base layer was considered in the calculation of the *k*-value for bonded JPCP overlay designs, which agreed with the assumptions made for bonded PCC overlays. It is also apparent that the calculated *k*-values matched the entered dynamic *k*-value.

Evaluation of the Design Results

It is reasonable that the design thickness of a bonded JPCP overlay is smaller than an unbonded JPCP overlay. Bonded JPCP overlays use the remaining structural capacity of the existing PCC, so their thickness can be relatively thin. On the other hand, unbonded JPCP overlays work independently so some restraints in the minimum thickness must be provided to guarantee their structural capacity. The research team concluded that the unreasonably thick HMA overlay can be attributed primarily to the HMA mix design used. As mentioned before, a thinner HMA overlay can be achieved by adjusting the HMA mix design properties. A thinner HMA overlay can also be considered with planned maintenance because all other performance criteria were met when not considering rutting.

For each kind of overlay, there is no difference in terms of the design thickness among the three design alternatives, which implies that it is quite reliable to employ the backcalculated dynamic elastic modulus for the PCC layer and the composite, dynamic *k*-value for all layers beneath the slab in the MEPDG design. Furthermore, no difference in terms of the design thickness of the overlays was observed between alternative 2 and alternative 3 (adjusted *k*-value and backcalculated *k*-value) for any of the three types of overlay, indicating that the dynamic *k*-value backcalculated from the FWD data can be used to represent the stiffness of the layers beneath the base, even though in reality it represents the composite stiffness of all layers beneath the slab. This is most likely because the base for test section 32-0203 was unstabilized and thus had a relatively small contribution to the overall stiffness. If a stabilized base were used, different design thicknesses could be expected among the three design alternatives.

SUMMARY

Test section 32-0203 represents a rigid pavement on granular base cross section case study. The backcalculation of PCC elastic modulus and *k*-value using the most recent data was only possible for the first half of test section 32-0203 (station 0 to 248 ft (0 to 75.6 m)) because of the inconsistency in the deflection basin profiles for the rest of the stations. However, the information included in the LTPP Program database for this section was sufficient to determine the load transfer characteristics and the support conditions for the entire section. The variation of the backcalculated *k*-value and the PCC elastic moduli along the section can be an indication that the backcalculation technique was capturing the overall stiffness, but it appears to be overestimating the *k*-value and, therefore, underestimating the elastic modulus of the concrete for several of the test locations.

The average laboratory-measured static PCC modulus 2.8 million psi (19,292,000 kPa) presented a good correlation with the average backcalculated static PCC elastic modulus of 3.1 million psi (21,359,000 kPa); however, the variation of the backcalculated values along the section was 22 percent, which was substantially higher than the 15 percent typically assumed acceptable.

The $AREA_{60}$ method determines a composite *k*-value, which is inclusive of the unbound base and subgrade layers. For the aggregate layers and lime-stabilized subgrade layer included in the pavement cross section, the backcalculated *k*-value was relatively low. Using the PCA's design method to back out a base layer contribution still resulted in a *k*-value that appears low considering the pavement cross section.

The LTE values along the section were above the typical acceptable level of 75 percent. This high level was constant over time, and it was not affected by cold temperatures, indicating the doweled joints were performing well. According to the void detection analysis, it appears that voids were not present beneath the slab in this section of roadway. However, there was a possibility that some erosion had begun to occur beginning in 2002 based on the increase over time observed in the void potential plot.

Based on the latest available LTPP Program data, the studied section was in poor condition, with a large number of transverse cracks. The necessary MEPDG design inputs are not all available in the LTPP Program database for the studied test section 32-0203, or any other project 32-0200 sections. Therefore, based on the LTPP Program data, appropriate inputs for rehabilitation designs in the MEPDG were discussed and determined.

Three kinds of overlays—HMA overlay, unbonded JPCP overlay, and bonded JPCP overlay—were designed for the rehabilitation. The thinnest design thickness was obtained by using a bonded JPCP overlay. The MEPDG design runs provided an unreasonably thick HMA overlay design, which was attributed mainly to the HMA mix design properties assumed. However, by adjusting the HMA mix design or by assuming intermediate maintenance, a more reasonable HMA overlay design thickness (on the order of 100 mm (4 inches) can be obtained.

No difference in terms of the design thickness was found among the three design alternatives (laboratory/material default values, adjusted backcalculated *k*-value and backcalculated PCC elastic modulus, and backcalculated PCC elastic modulus and *k*-value), indicating the reliability of using both the backcalculated dynamic elastic modulus for the PCC layer and the dynamic *k*-value for the supporting layers in the MEPDG design.

The research team found that the backcalculated *k*-value that represented the composite stiffness of all layers beneath the slab can be directly entered into the MEPDG without having a significant influence on the design thickness for the pavement structure analyzed. However, this does not definitively mean that the MEPDG takes the stiffness of the base layer into account in the *k*-value. It could be either because of the insensitivity of the design thickness on the input *k*-value or because the unstabilized granular contributed very little to the composite stiffness of all layers beneath the slab. Other observations made regarding the calculation of *k*-value within the MEPDG program include the following:

- For the HMA overlay design, varying the subbase stiffness had very little influence on the determined *k*-value, which seemed to suggest the stiffness of the subbase layer was not taken into account in the calculated *k*-value. In addition, the reported *k*-values in the output file were identical for cases with varying base layer stiffness, indicating the stiffness of the base was likely not included in the *k*-value calculation. Furthermore, it appears that the MEPDG ignores the entered dynamic *k*-value and calculates the *k*-values based on the entered layer moduli because the summarized values were the same regardless of whether the dynamic *k*-value was entered or not.

- For the unbonded PCC overlay, additional design runs seemed to indicate that the stiffness of the interlayer and the existing PCC were not considered in the calculation of the *k*-value. It did appear that the base layer was taken into account by the difference in *k*-values when using a stiff and weak base layer. In addition, the *k*-values agreed well with the entered dynamic *k*-value, suggesting that the MEPDG uses the entered value for unbonded PCC overlay designs.

- It appears that the modulus of the base layer was considered in the calculation of the *k*-value for bonded JPCP overlay designs, which agreed with the assumptions made for bonded PCC overlays. It was also apparent that the calculated *k*-values matched the entered dynamic *k*-value.

CHAPTER 4. CASE STUDY 4: RIGID PAVEMENT EVALUATION AND OVERLAY DESIGN FOR STABILIZED BASE LAYER

PROJECT OVERVIEW

Project 05-0200 was located on I-30 in Saline County, AR. Test section 05-0218—a JPCP placed on a stabilized base layer—was selected for the rigid pavement rehabilitation case study. This section is representative of the following selection factors:

- Rigid pavement.
- Wet-nonfreeze climate zone.
- Rural principal arterial—interstate functional class.
- “Poor” pavement condition.
- Gravel subgrade classification.

The test section was originally constructed in September 1993. In the LTPP Program database, this was referred to as construction number 1. Since the initial construction, the section has been rehabilitated twice. The first rehabilitation was in February 1997 (referred to as construction number 2), in which lane-shoulder and longitudinal-joint sealing was performed. The second rehabilitation occurred in December 2002, in which transverse joint and crack sealing was performed on the section and was identified as construction number 3. Based on LTPP Program core data, the typical cross section of test section 05-0218 consisted nominally of 8 inches (203 mm) of PCC (surface layer), 7 inches (178 mm) of a treated base (lean concrete), 4 inches 102 mm) of a granular layer, a woven geotextile (interlayer), and a subgrade layer.

PAVEMENT CONDITION/PERFORMANCE

Distress survey results, in terms of joint faulting and slab cracking, were available in the LTPP Program database from 1996 to 2007. According to the available records, only four joints had faulting greater than 0.1 inches (2.54 mm), which did not appear until 2001.

In terms of slab cracking, the studied pavement section was in poor condition. Table 60 presents the number of cracks identified for test section 05-0218 from 1996 to 2007. To convert the number of cracks to the percentage of slabs cracked, the assumption that only one transverse crack occurred within each slab was made. The section is 500 ft (152.5 m) long, and slabs are 15 ft (4.6 m) long, so there are approximately 33 to 34 per lane, with a total of 66 to 68 slabs. With 39 cracks identified in 2007, it was approximated that more than 50 percent of the slabs were cracked.

Table 60. Transverse cracking distress data for test section 05-0218.

Survey Date	Number of Transverse Cracks			
	Low Severity	Medium Severity	High Severity	Total
11/14/1996	0	0	0	0
6/29/2000	8	7	0	15
7/25/2001	17	11	2	30
8/28/2002	11	19	3	33
10/07/2003	9	22	1	32
9/15/2004	9	23	1	33
5/23/2007	4	34	1	39

TEST SECTION DATA

The MEPDG program requires a significant number of inputs, particularly for a level 1 analysis. The required design data for test section 05-0218 were obtained from the LTPP Program DataPave database. The data were generally not complete for any specific test section within project 05-0200; therefore, sometimes the data available from other sections of the project were used to define the necessary design inputs.

Deflection-Testing Data

Deflection data for test section 05-0218 were available from the LTPP Program database for several years of testing, including 2001, 2003, and 2004. The data used for the backcalculation were from the most recent date because that was a better representation of the current conditions in the field. Deflection testing was conducted following the LTPP Program protocols.

Equipment

Deflection testing was conducted with a Dynatest® FWD (SN 8002-132).

Sensor Configuration

The sensor configuration used in the conduct of the FWD testing is presented in table 61.

Table 61. Sensor configuration for the FWD testing.

Configuration	Sensor Number								
	1	2	3	4	5	6	7	8	9
Offset (inches)	0	8	12	18	24	36	48	60	-12

Number of Drops and Load Levels

Three load level targets—9,000, 12,000, and 16,000 lb (4,077, 5,436, and 7,248 kg) with four drops at each load level were performed, and the resultant data were recorded. Seating drops were also performed at each test point, but deflection data for those drops were not recorded.

Test Locations/Lanes and Increments

The location of the FWD testing is presented in table 62. The 2004 testing occurred from 12:30 to 3:37 p.m. at 10 different locations along the section. The data from J1 were used to backcalculate *k* and *E* values, data from J4 and J5 were used to determine LTE and differential deflections, and data from J2 were used to determine the presence of voids under the slab. Tests were conducted at mid-panel, adjacent to the outside lane/shoulder joint at the corner and at mid-panel, and in the outer wheelpath on the approach and leave sides of the transverse joints. The distance between tests conducted at mid-slab was between 30 and 70 ft (9.15 and 21.4 m), whereas the tests carried out on the pavement edges at corners had intervals between 9 and 23 ft (2.7 and 7 m).

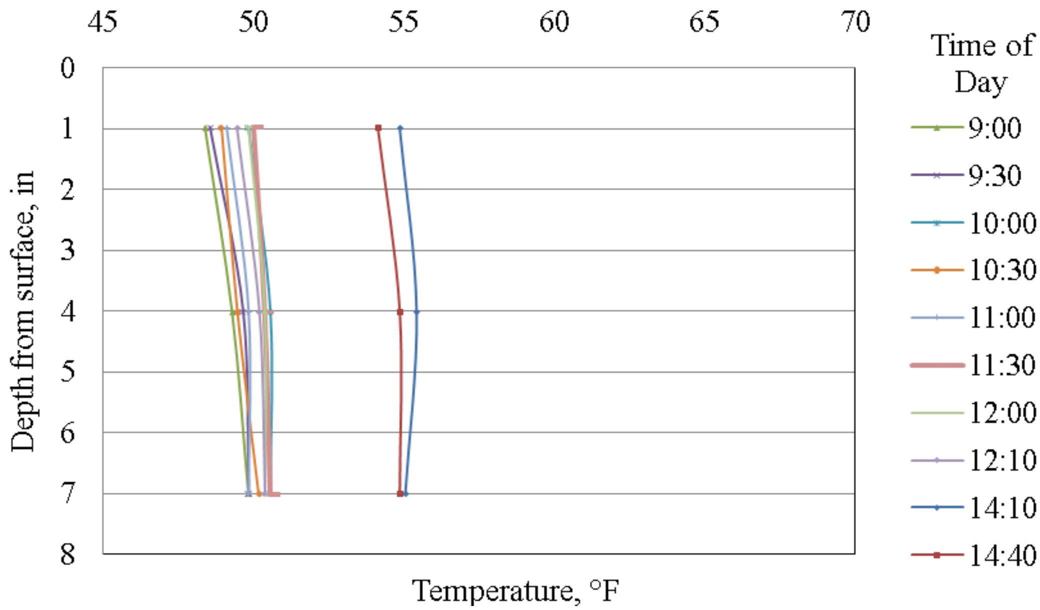
Table 62. Location of the deflection test on the slab.

LTPP Code	Location
J1	JCP middle lane at middle panel
J2	JCP pavement edge at corner
J3	JCP pavement edge at mid panel
J4	JCP load transfer outer wheelpath at joint approach
J5	JCP load transfer outer wheelpath at joint leave

Temperature Measurements

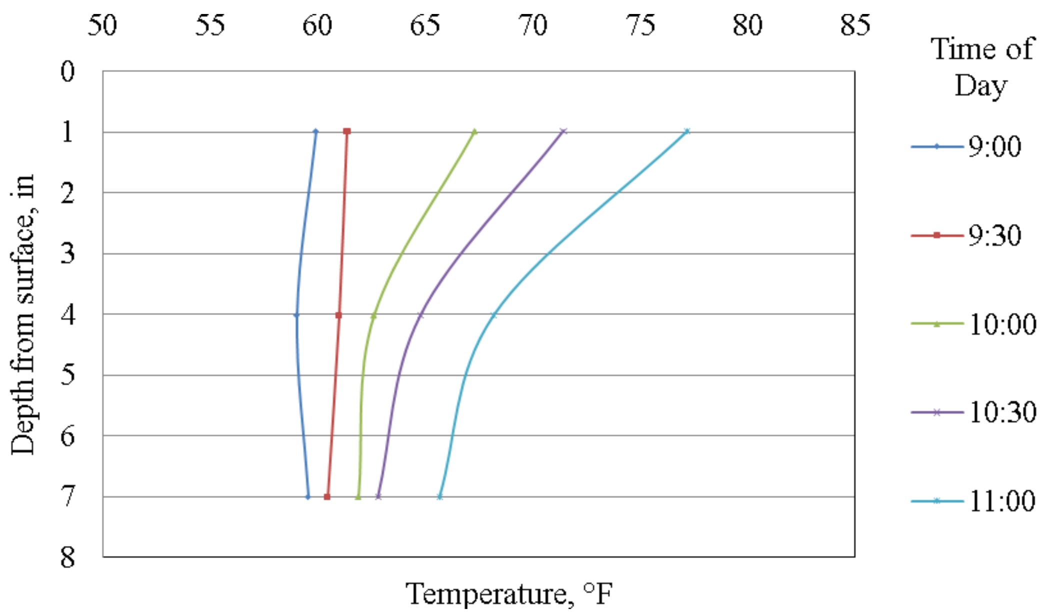
Temperature measurements were taken using holes drilled in the pavement at time intervals of either 15 or 30 min during the deflection testing. In 1996, 1999, and 2001, the holes were at depths of 1, 4, and 7 inches (25, 102, and 178 mm). In 2003 and 2004, the holes were at depths of 0, 1, 2, 4, and 8 inches (0, 25, 51, 102, and 203 mm). Figure 62 through figure 67 show temperatures in the slab at the time of each FWD test, along with the prevailing weather conditions. The slab temperatures were extracted from the LTTP database. It should be noted that the temperature readings were taken on two different days in 1999, namely April 15 and April 19. The temperature profiles for those 2 days were put onto the same graph because they were only a few days apart and the weather was similar on those 2 days. As it can be seen in the figures, the temperatures throughout the PCC slab changed daily and seasonally.

The temperature measurements collected in 1996 were taken during November. The slab temperature difference for this month is nearly zero. This means the shape of the slab was defined by the construction gradient present at the time the slab set. The next series of slab temperature measurements occurred in April 1999. Two records were available for this month, one in the afternoon and one in the morning. In the morning, the sun was out, and in the afternoon, it was cloudy. Figure 63 and figure 64 are good examples of how solar radiation affected the temperature distribution in the slab—the temperature difference was positive in both figures and was highest at about 11:00 a.m. A positive temperature difference will cause the slab to curl downwards. The next measurements were taken in July 2001. The temperature difference was much higher in the afternoon for the month of July when compared with the same measurements taken in April, and it reached a high of 24.3 ºF (13.5 ºC) at 1:30 p.m. causing the slab to curl downwards. The next two measurements were taken in October 2003 and September 2004. The average temperature difference was 14.9 and 19.7 ºF (8.3 and 10.9 ºC) in October and September, respectively. Overall, the highest temperature difference was observed in July, and the temperature difference was larger in the afternoon.



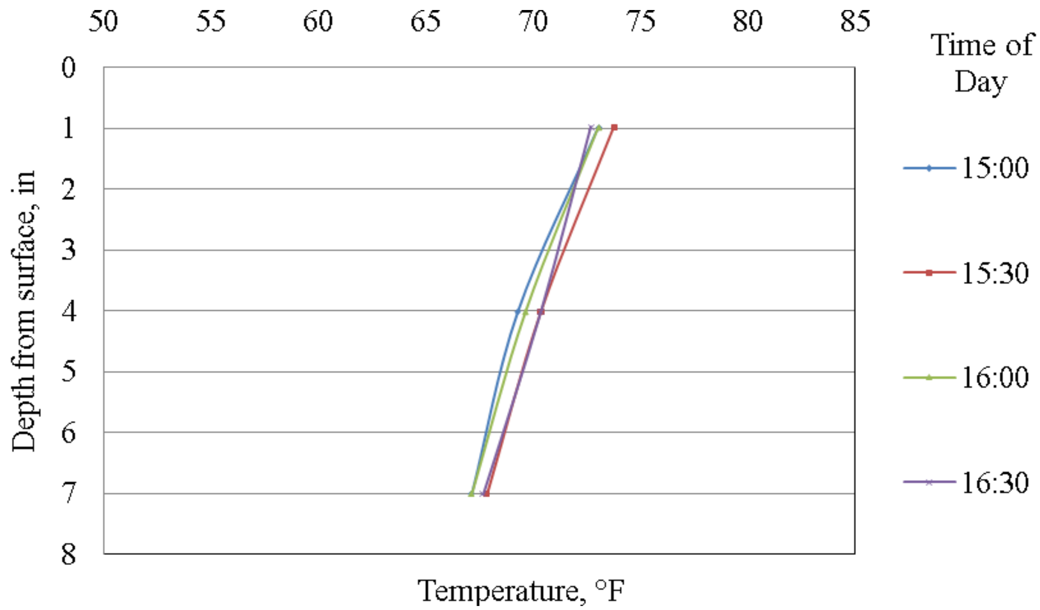
1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 62. Graph. Slab temperatures for the FWD test performed on 11/14/1996—cloudy.



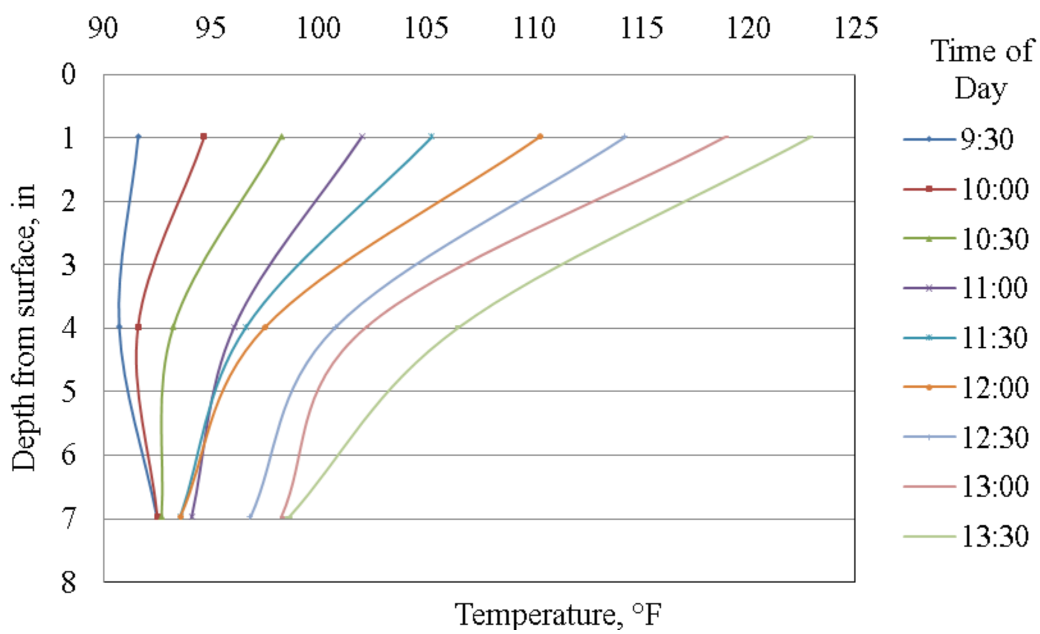
1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 63. Graph. Slab temperatures for the FWD test performed on 4/19/1999—sunny.



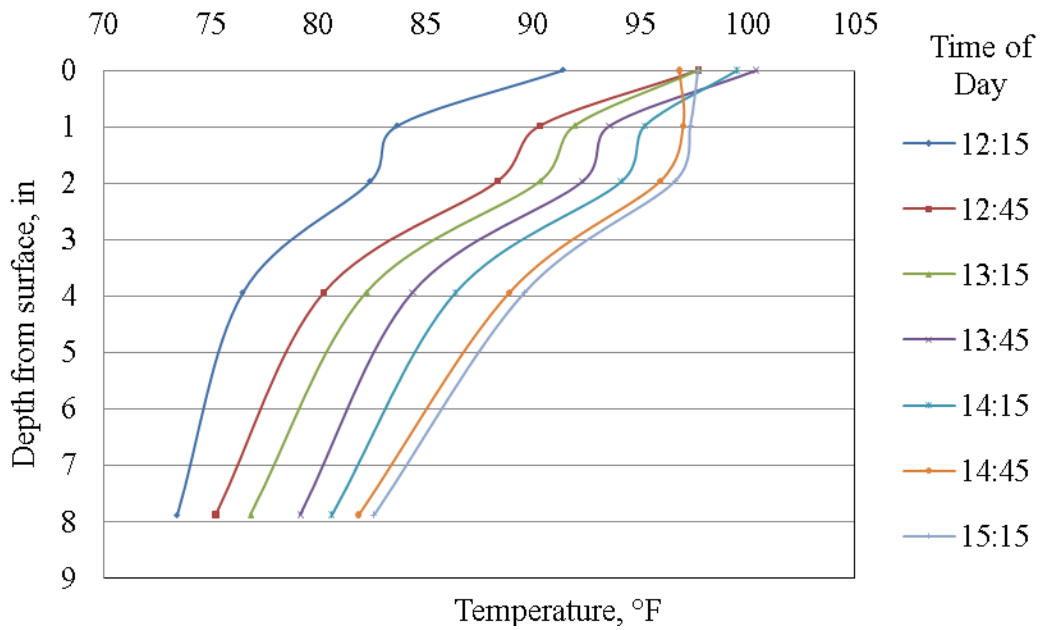
1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 64. Graph. Slab temperatures for the FWD test performed on 4/15/1999—cloudy.



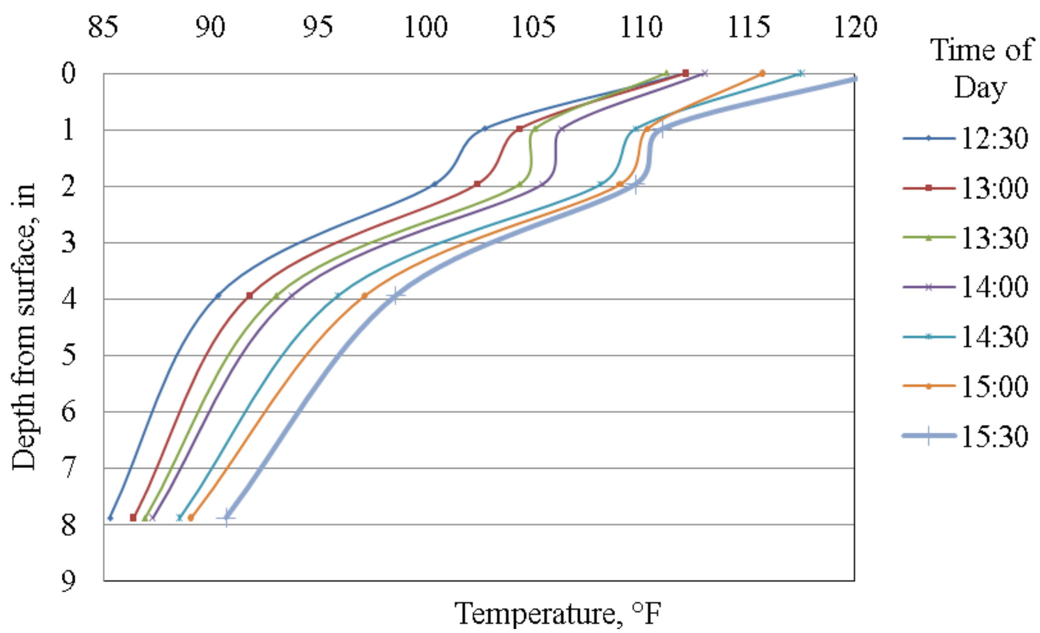
1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 65. Graph. Slab temperatures for the FWD test performed on 7/26/2001—sunny.



1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 66. Graph. Slab temperatures for the FWD test performed on 10/8/2003—sunny.



1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 67. Graph. Slab temperatures for the FWD test performed on 9/16/2004—sunny/partly cloudy.

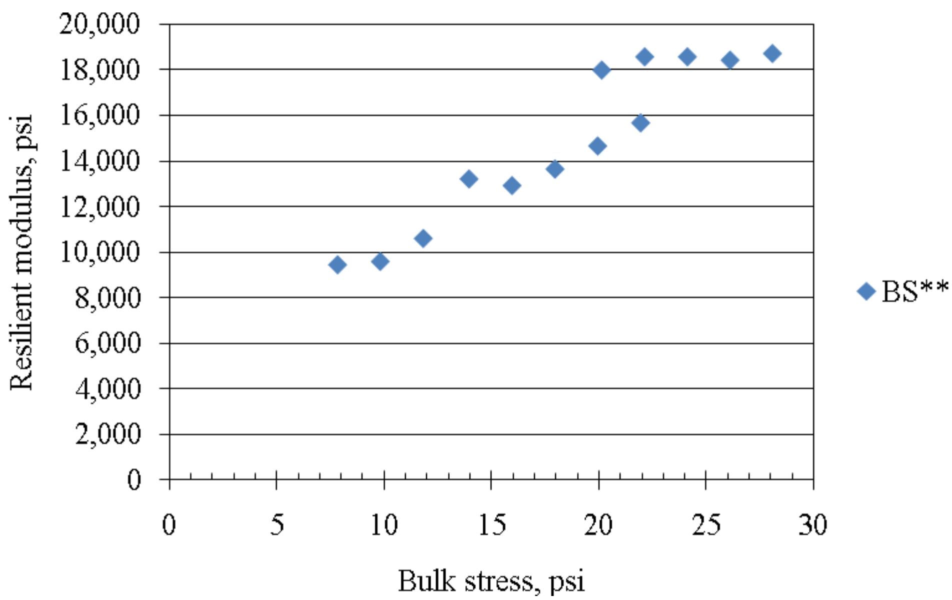
Material Properties Data

This section summarizes the data obtained from the LTPP database for test section 05-0218 regarding its subgrade, treated base, subbase, PCC material properties, traffic, climate, and depth of water table or stiff layer.

Subgrade

Nineteen subgrade samples were retrieved from the project 05-0200 as part of the LTPP Program. No specific soil classification data were available for test section 05-0218. Subgrade soil samples from sections 05-0217and 05-0219 were tested and classified as AASHTO A-2-4, and this soil type was assumed for test section 05-0218.

Laboratory resilient modulus testing was performed on 10 samples from section 05-0224 in December 2002. The test results are illustrated in figure 68, where BS** is the code name for the sample. No other sections had resilient modulus test information. As summarized in figure 68, the resilient modulus of the subgrade ranged from approximately 9,000 to 19,000 psi (62,053 to 131,000 kPa) for a bulk stress between 8 and 28 psi (55 and 193 kPa).



1 psi = 6.89 kPa.

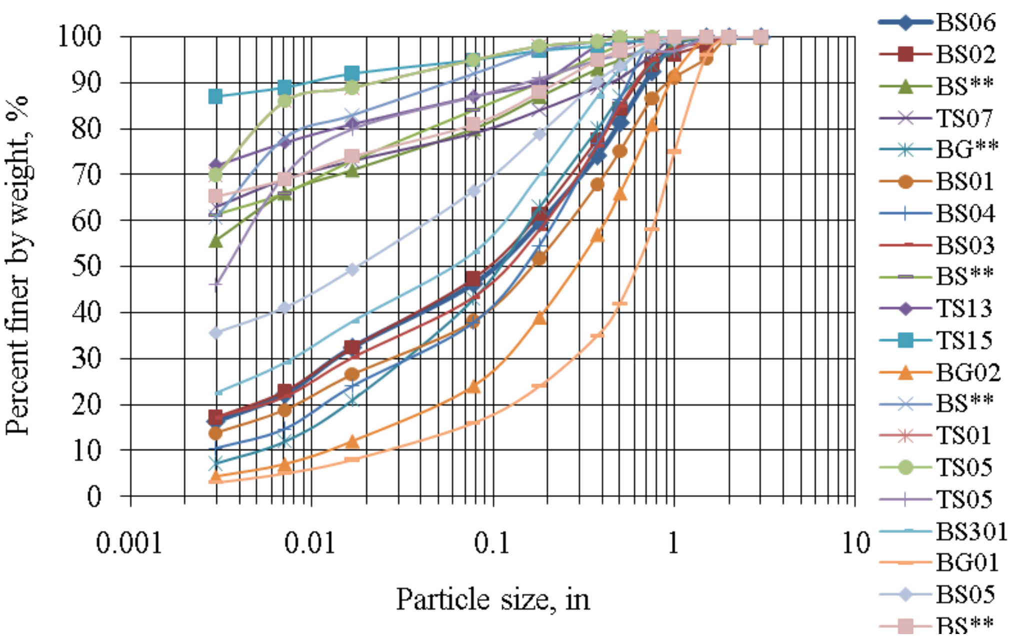
Figure 68. Graph. LTPP Program data of laboratory resilient modulus testing for the subgrade.

Additional subgrade properties, including the sieve analysis are summarized in table 63. Gradation of the subgrade samples is shown in figure 69. The average moisture content was 2.2 percent. Other properties (such as Poisson's ratio and coefficient of lateral pressure) were assumed to be the default values for the soil type. It should be noted that the Atterberg limits for the samples taken from test sections 05-0217 and 05-0219 were very high and did not match the soil classification (AASHTO A-2-4); therefore, the default values in the MEPDG were used for these inputs.

Table 63. Gradation of the subgrade.

Sieve Size	Average Percent Passing
3 inch	100.0
2 inch	100.0
1.5 inch	99.6
1 inch	98.5
$^3/_4$ inch	96.8
$^1/_2$ inch	91.9
$^3/_8$ inch	87.9
No. 4	77.5
No. 10	68.1
No. 40	57.3
No. 80	50.5
No. 200	42.1

1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 69. Graph. Gradation of the subgrade samples.

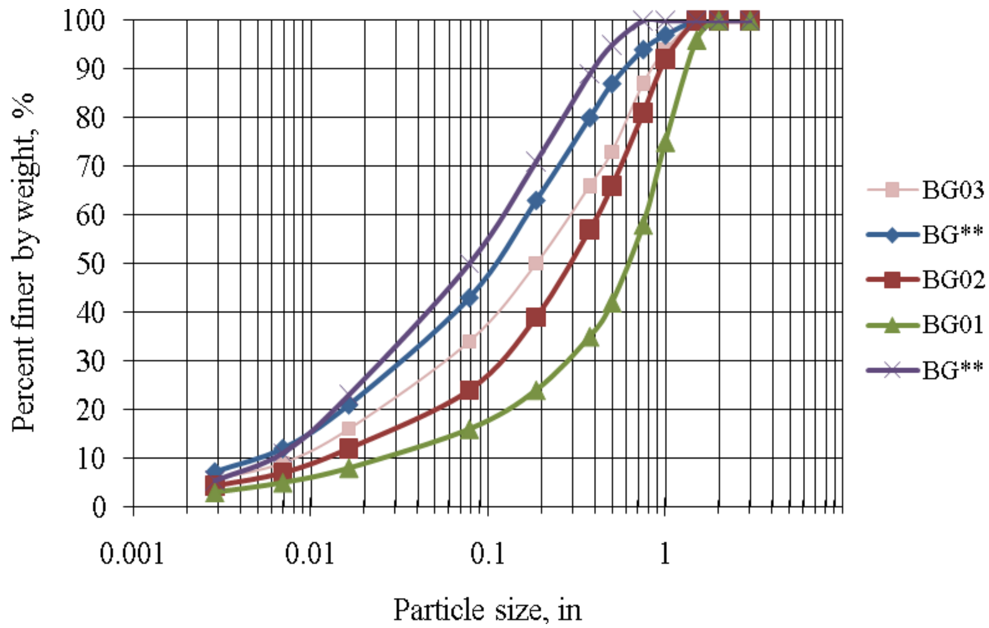
Granular Subbase

The granular subbase was identified as a crushed stone. The sieve analysis for the subbase layer is summarized in table 64; figure 70 illustrates the gradations of the subbase samples. The average moisture content was 3.5 percent. No information regarding Atterberg limits was available; therefore, the MEPDG default values based on material type were used. Laboratory resilient modulus testing results were available only for section 05-0213. The test was performed in December 1999 for 15 samples, with the test results illustrated in figure 71.

Table 64. Gradation of the granular base samples.

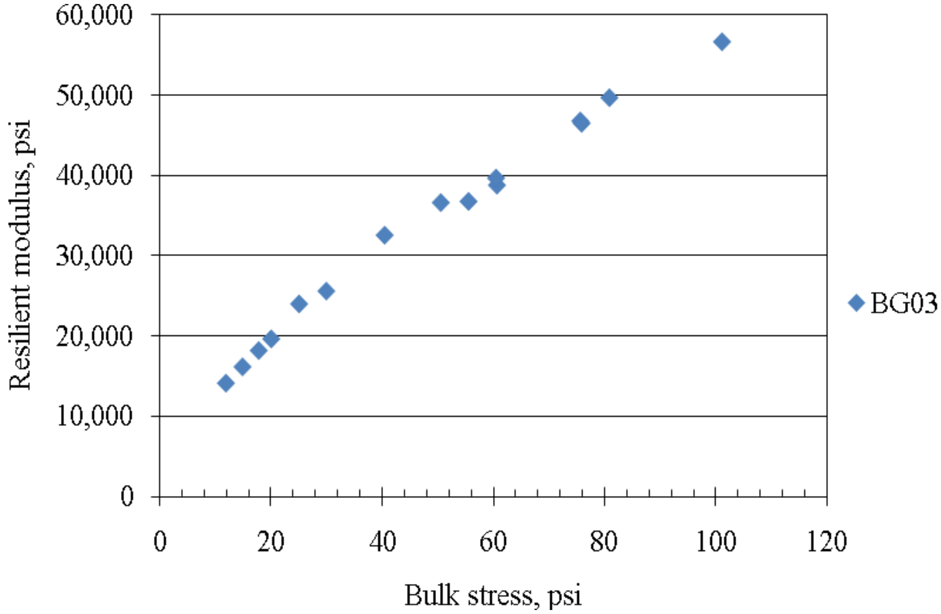
Sieve Size	Average Percent Passing
3 inch	100.0
2 inch	100.0
1.5 inch	99.2
1 inch	91.6
$^{3}/_{4}$ inch	84.0
$^{1}/_{2}$ inch	72.6
$^{3}/_{8}$ inch	65.4
No. 4	49.4
No. 10	33.4
No. 40	16.0
No. 80	8.8
No. 200	5.1

1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 70. Graph. Gradation of the granular base samples.



1 inch = 25.4 mm.
1 psi = 6.89 kPa.

Figure 71. Graph. Summary of the base aggregate resilient modulus from LTPP Program laboratory testing.

Based on the test results summarized in figure 71, the resilient modulus for this layer varied between 10,000 to 60,000 psi (68,900 to 413,400 kPa) for the bulk stress of 20 to 100 psi (138 to 689 kPa). Considering the fact that resilient modulus testing was not performed on any samples from test section 05-218 and, moreover, recognizing the difficulty in determining the resilient modulus due to the unknown confinement condition, the typical resilient modulus suggested by the MEPDG for the granular layer of 30,000 psi (206,820 kPa) was used.

Other properties of this layer (such as Poisson's ratio and coefficient of lateral pressure) were defined as the MEPDG default values for the material type.

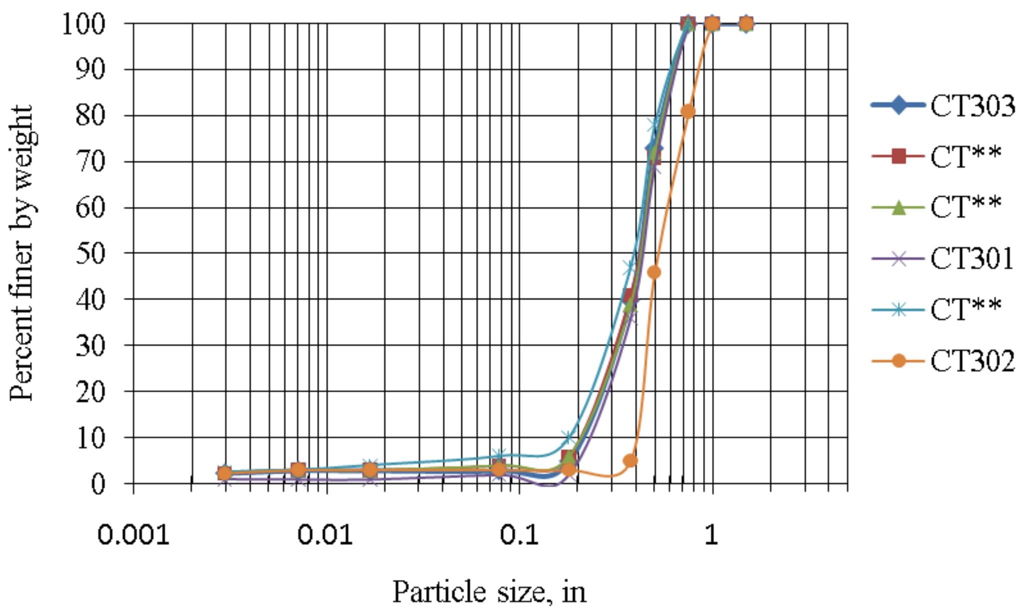
Treated Base Layer

The treated base layer was a 7-inch (178-mm) lean concrete mixture. Sieve analysis test results were available for the coarse aggregate used in the lean concrete base. Six coarse aggregate samples were obtained as part of the LTPP Program data collection and were classified as quartzite. The results are presented in table 65 and figure 72. In addition to the coarse aggregate properties, the LTPP Program database contained the compressive strength, static elastic modulus, flexural strength, and the split tensile strength for the lean concrete base. The testing dates of these samples varied for each test. The compressive strength specimens were tested 4 and 13 years after the section was constructed; however, the database did not specify whether the specimens were cast at the time of construction or sometime thereafter. Similarly, the static elastic modulus was tested 13 years after the initial construction date, but the age of the specimen was unknown. The flexural strength specimens were tested 2 years after the construction date. The split tensile strength was the only test that had a known age, which was 28 days. A summary of these tests for the lean concrete base is presented in table 66. The density of the cores was also measured. There was no information regarding the CTE of the lean concrete.

Table 65. Sieve analysis test results for the lean concrete base layer.

Sieve Size	Average Percent Passing
3 inch	100.0
2 inch	100.0
1.5 inch	96.8
1 inch	68.2
$^{3}/_{4}$ inch	34.7
$^{1}/_{2}$ inch	5.3
$^{3}/_{8}$ inch	3.7
No. 4	2.8
No. 10	2.7
No. 40	2.2

1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 72. Graph. Comparison of gradation of the lean concrete base samples.

Table 66. Summary of lean concrete base laboratory testing results.

Laboratory Test	Average Test Result
4-year compressive strength (psi)	7,331
13-year compressive strength (psi)	5,480
Elastic modulus (psi)	3,317,000
Modulus of rupture (psi)	522
Splitting tensile strength (psi)	629
Poisson's ratio	0.29
Density (lb/ft^3)	140.3

1 psi = 6.89 kPa.
1 lb/ft^3 = 0.0160 g/cm^3.

There was a significant amount of variability in the elastic modulus testing results (approximately 60 percent COV). Because the strength properties of the cement stabilized layer were defined through the elastic modulus in the MEPDG software and the very high variability from the test results, the typical value for a cement stabilized layer (2 million psi (13,780,000 kPa)) was selected for this parameter. There was also significant variability in the Poisson's ratio results, so the default value (0.20) was also used for this property.

PCC Layer

The PCC layer had an average thickness of 7.8 inches (198.1 mm) based on available coring data. The LTPP Program database contained laboratory compressive strength, elastic modulus, Poisson's ratio, modulus of rupture, and split tensile test results for PCC samples from project 05-0200. The testing dates of these samples varied for each test. The compressive strength specimens were tested 4 and 13 years after the section was constructed; however, the database did not specify whether the specimens were cast at the time of construction or sometime after. Similarly, the static elastic modulus and Poisson's ratio were tested 9 and 13 years after the initial construction date, but the age of the specimen was unknown. The flexural strength specimens were tested 2 years after the construction date. The split tensile strength was the only test that had a known age, which was 28 days. These data are provided in table 67.

The average Poisson's ratio for all the 05-200 sections ranged from 0.20 (9 years) to 0.24 (13 years), as summarized in table 67. However, the Poisson's ratio measured for test section 05-0218 in 2006 was reported as 0.15, which is the value typically assumed for PCC. Therefore, the value determined specifically for the section was used in the analysis. Similarly, the unit weight determined specifically for test section 05-0218 (138 lb/ft^3 (2.2 g/cm^3)) was used in the analysis, which was only slightly lower than the overall average.

Table 67. Summary of PCC laboratory testing results.

Laboratory Test	Average Test Result
4-year compressive strength (psi)	8,189
13-year compressive strength (psi)	7,798
9-year elastic modulus (psi)	5,530,000
13-year elastic modulus (psi)	4,020,000
2-year splitting tensile strength (psi)	475
13-year splitting tensile strength (psi)	735
9-year Poisson's ratio	0.20
13-year Poisson's ratio	0.24
Modulus of rupture (psi)	623
Density (lb/ft^3)	141.6

1 psi = 6.89 kPa.
1 lb/ft^3 = 0.0160 g/cm^3.

The laboratory-measured elastic modulus specific to test section 05-0218 was 3.6 million psi (24,804,000 kPa) from the 2006 data, which was slightly lower than the overall average. Using the correlation from the MEPDG between the elastic modulus and modulus rupture, for an elastic modulus of 3.6 million psi (24,804,000 kPa), a modulus of rupture of 630 psi (4,341 kPa) was obtained. This modulus of rupture estimated for the section was close to the overall project average.

No data were available on the thermal properties of the PCC. For the purposes of this analysis, it was assumed that the aggregate type was a syenite, with a CTE of 5.2×10^{-6}/ºF (9.4×10^{-6}/ºC).

The average PCC air content was 7.8 percent. Additional mixture properties are summarized in table 68 and table 69.

Table 68. Summary of PCC mix design information.

Variable	Value
Cement type	Type I
Cementitious material content (lb/yd^3)	719
Water-to-Cement ratio	0.38
Curing method	Membrane curing compound

1 lb/yd^3 = 0.593 kg/m^3.

Table 69. Summary of PCC pavement design features.

Variable	Value
Joint spacing (ft)	15
Sealant type	Liquid (silicone)
Dowel diameter (inches)	1.25
Dowel length (inches)	18
Dowel spacing (inches)	12
Edge support	10-ft HMA shoulder
Lane width (ft)	12

1 ft = 0.305 m.
1 inch = 25.4 mm.

Depth to Rigid Layer/Water Table

The depth to the water table is also required by the MEPDG. However, no data were found regarding the location of the water table for test section 05-0218 in the LTPP Program database. Historical groundwater levels from the USGS Web site indicate an average of approximately 25 to 75 ft (7.6 to 22.9 m) below ground surface.(4) This monitoring location is not at the location of the 05 0218 section, but is located in Saline County, AR. A value of 25 ft (7.6 m) was adopted for the design of this section.

Climate/Environment Data

Climate data were obtained from the updated climate files on the MEPDG Web site.(5) Sixteen climate stations were available in the MEPDG for the State of Arkansas. Considering both elevation and distance, the station in Little Rock was chosen for the study. This station was 52.9 mi (85.2 km) from the site and 18 ft (5.5 m) lower in elevation. The general weather category for the case study location is wet-nonfreeze.

Traffic Data

AADT data were available from the LTPP Program database for 2 years: 1996 and 1998. The average AADTT data were available from the LTPP Program database for 2000, 2007, and 2008. These values are listed in table 70. Only the data from these years were used to calculate the compound growth factor. A linear regression of the traffic from these 3 years gave a compound growth factor of 1.9 percent and a two-way AADTT of 11,265 for 2009 with a R^2 value of 0.99. Including traffic data from years 1996 and 1998 would decrease the R^2 value to 0.14, so these values were omitted. Assuming a directional distribution factor of 0.5 and a design lane distribution factor of 0.95, 45 percent of the two-way truck traffic was in the design lane.

Table 70. AADTT from the LTPP Program database.

Year	Two-Way AADTT
1996	7,360
1998	12,760
2000	9,684
2007	10,840
2008	11,080

ANALYSIS AND INTERPRETATION OF FWD TESTING DATA

This section presents the data checks; backcalculation analysis of the FWD data, LTE, and void detection, as well as a comparison of the backcalculation results with laboratory testing.

Preprocessing Deflection Data

FWD testing was performed on five separate occasions—November 13, 1996; April 15, 1999; July 26, 2001; October 8, 2003; and September 16, 2004. The FWD data collected on September 16, 2004, were selected to backcalculate the PCC elastic modulus and the *k*-value.

Table 71 is a summary of the deflection data for each location as well as the respective loads. The 9,000-lb (4,086 kg) load group was used for all backcalculation analysis. The data for the four drops at this load level were averaged and used for backcalculation. The loads and deflections used to determine joint load transfer characteristics and the presence of potential voids are shown in table 72 and table 73, respectively.

Table 71. Loads and deflections used to backcalculate the PCC elastic modulus and *k*-value.

Station (ft)	Location	Drop load (lb)	Deflection, mil								
			Sensor 9	Sensor 1	Sensor 2	Sensor 3	Sensor 4	Sensor 5	Sensor 6	Sensor 7	Sensor 8
35.10	J1	9,466	2.49	2.91	2.68	2.53	2.28	2.07	1.63	1.27	1.01
81.04	J1	9,248	3.35	3.62	3.35	3.14	2.80	2.47	1.85	1.34	0.93
126.97	J1	9,264	3.70	4.38	4.24	4.09	3.82	3.56	2.93	2.34	1.75
171.92	J1	9,201	2.86	3.62	3.61	3.50	3.23	2.94	2.14	1.38	0.92
216.86	J1	9,233	2.76	3.14	2.87	2.68	2.31	2.09	1.50	1.09	0.81
277.89	J1	9,148	2.95	3.24	3.01	2.83	2.61	2.35	1.90	1.48	1.13
323.16	J1	9,185	3.41	3.82	3.60	3.36	3.04	2.69	2.14	1.68	1.31
366.14	J1	9,169	3.14	2.93	2.55	2.36	2.07	1.93	1.57	1.29	1.05
413.06	J1	9,069	2.77	3.24	2.97	2.85	2.61	2.35	1.82	1.44	1.13
488.85	J1	9,116	3.27	3.73	3.57	3.44	3.19	2.94	2.47	2.01	1.46

1 ft = 0.305 m.
1 mil = 0.0254 mm.
1 lb = 0.454 kg.

Table 72. Loads and deflections used to determine the joint load transfer characteristics of test section 05-0218.

Station (ft)	Joint	Test Location	Drop Load (lb)	Deflection (mil) Sensor 9	Sensor 1	Sensor 2
27.9	1	J4	9,143	3.74	5.04	4.08
28.9	1	J5	9,100	3.80	3.70	3.07
73.2	2	J4	9,127	2.56	3.30	3.15
74.2	2	J5	9,116	2.78	2.96	2.39
118.1	3	J4	9,111	3.26	3.88	3.44
119.1	3	J5	9,158	3.08	3.15	2.71
164.0	4	J4	9,233	2.55	3.28	2.80
165.0	4	J5	9,211	2.47	2.61	2.34
209.0	5	J4	9,095	2.44	2.78	2.54
210.0	5	J5	9,084	2.32	2.59	2.25
269.0	6	J4	9,164	3.93	4.57	4.22
270.0	6	J5	9,026	3.61	3.49	3.02
314.0	7	J4	9,105	3.89	4.56	4.34
315.0	7	J5	8,931	3.92	3.87	3.44
357.0	8	J4	8,973	3.80	4.69	4.44
357.9	8	J5	9,037	3.87	3.81	3.26
404.9	9	J4	8,947	3.52	4.02	3.54
405.8	9	J5	8,894	3.06	3.07	2.65
480.0	10	J4	8,962	3.13	3.64	3.47
481.0	10	J5	8,999	3.34	3.71	3.39

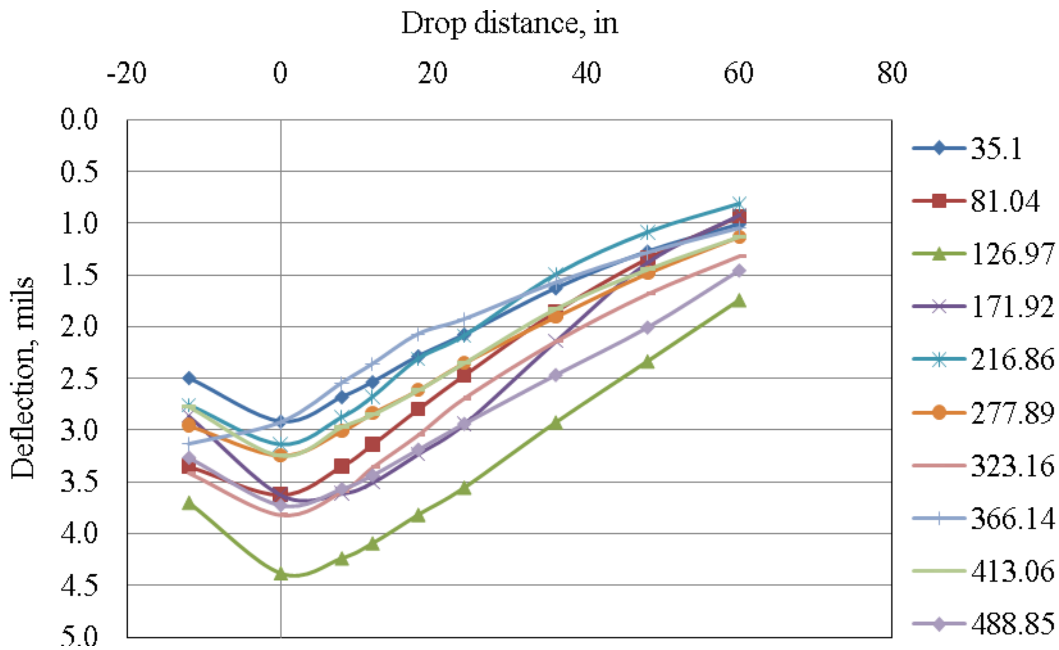
1 ft = 0.305 m.
1 mil = 0.025 mm.
1 lb = 0.454 kg.

Table 73. Loads and deflections used for void detection.

Station (ft)	Location	Drop Load (lb)	Sensor 1 Deflection (mil)
8.8	J2	9,105	7.47
8.8	J2	12,204	9.54
8.8	J2	15,931	12.06
22.6	J2	9,133	5.60
22.6	J2	12,224	7.12
22.6	J2	16,097	8.84
36.3	J2	9,165	4.50
36.3	J2	12,188	5.93
36.3	J2	15,490	7.65
50.3	J2	9,272	3.87
50.3	J2	12,145	4.96
50.3	J2	15,593	6.29
64.0	J2	9,193	4.08
64.0	J2	12,252	5.59
64.0	J2	15,911	7.38
82.3	J2	9,113	4.21
82.3	J2	12,196	5.66
82.3	J2	16,101	7.43
96	J2	9,105	5.01
96.0	J2	12,109	6.58
96.0	J2	15,744	8.46
109.01	J2	8,903	5.77
109.1	J2	12,029	7.30
109.1	J2	15,891	8.76
123.7	J2	9,046	4.01
123.7	J2	12,117	5.39
123.7	J2	16,022	7.15
146.6	J2	9,058	5.29
146.6	J2	12,156	6.95
146.6	J2	15748	8.88

1 ft = 0.305 m.
1 mil = 0.025 mm.
1 lb = 0.454 kg.

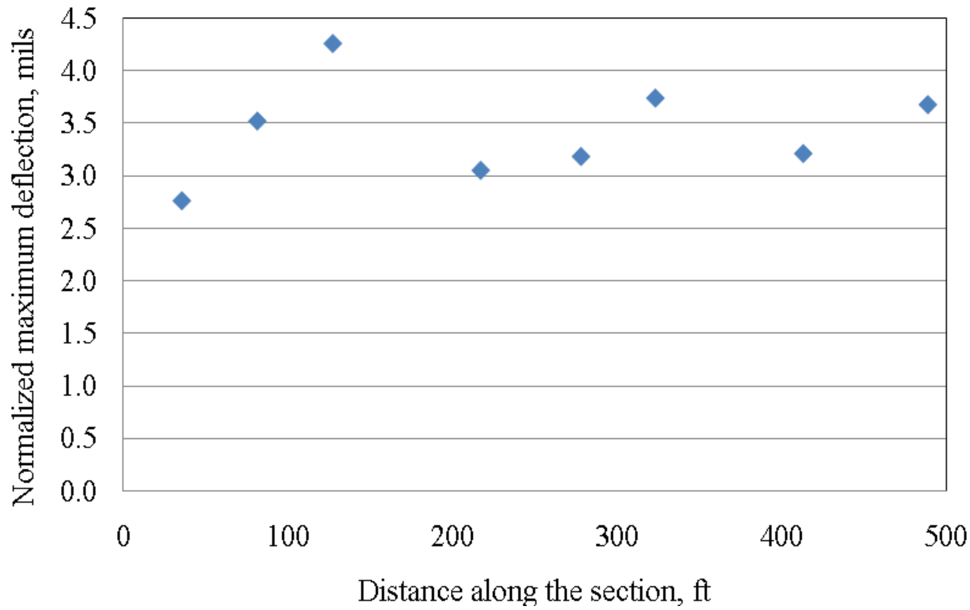
The first step in the data analysis was to plot the deflection basin based on the measured deflections for each test location. The deflection basins for test section 05-0218 are shown in figure 73, where the legend indicates the test locations as presented in table 71. As shown in the figure, all of the locations showed good profiles except for the stations at 171.9 ft (52.4 m) and 366.1 ft (111.7 m). These locations had unacceptable profiles because of the irregular relationship between their load positions and deflections. The deflection data from these stations were not used in the backcalculation of the *k*-value and the PCC elastic modulus.



1 mil = 0.0254 mm.
1 inch = 25.4 mm.

Figure 73. Graph. Deflection basins for test section 05-0218.

Secondly, to graphically evaluate the deflection variation along the section, the maximum deflections, presented in table 71, were normalized to the standard 9,000-lb (4,086 kg) load and plotted against distance, as shown in figure 74. As illustrated in figure 74, there was a variation in the structural response of the pavement along the section, particularly in the first approximately 150 ft (45 m), but there was generally no consistent trend.



1 ft = 0.305 m.
1 mil = 0.0254 mm.

Figure 74. Graph. Normalized deflections along test section 05-0218.

Backcalculation Analysis

Backcalculation of the test section 05-0218 deflection data was performed using the $AREA_{60}$ method to determine the *k*-value and layer moduli. The analysis for this section consisted of a three-layer system: PCC slab, stabilized base, and spring foundation.

Backcalculation Results

Table 74 presents a summary of the determined $AREA_{60}$ and radius of relative stiffness values for each location. The dynamic *k*-value and PCC elastic modulus for each station are presented in table 75 along with their overall static and dynamic average values.

Table 74. Summary of $AREA_{60}$ and radius of relative stiffness.

Station (ft)	$Area_{60}$	Radius of Relative Stiffness (inches)
35.1	39.18	30.17
81.04	36.92	27.23
126.97	43.80	38.10
216.86	35.87	26.02
277.89	40.04	31.42
323.16	39.38	30.45
413.06	39.55	30.69
488.85	43.20	36.88

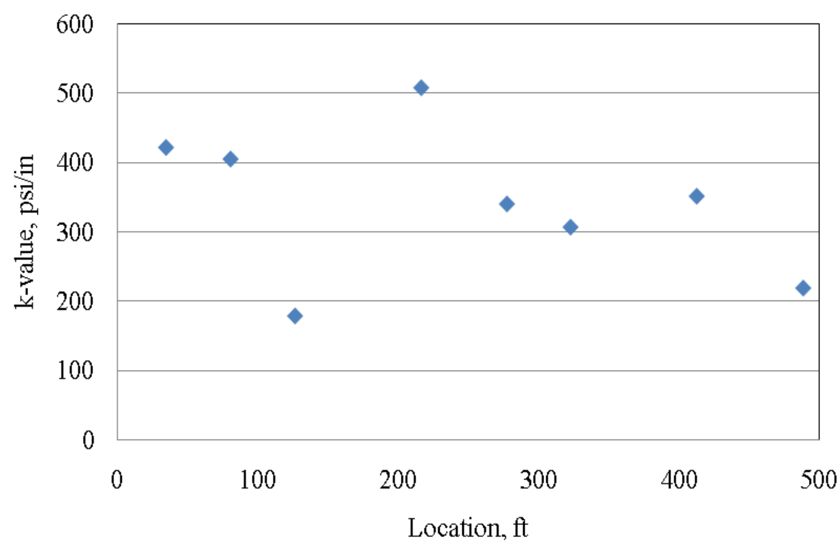
1 ft = 0.305 m.
1 inch = 25.4 mm.

Table 75. Average dynamic and static E and *k*-value.

Station (ft)	Dynamic k-value (psi/inch)	Dynamic PCC Elastic Modulus (psi)*
35.1	422	9,010,000
81.04	405	5,730,000
126.97	178	9,270,000
216.86	508	6,200,000
277.89	340	8,470,000
323.16	307	5,920,000
413.06	352	7,900,000
488.85	219	9,060,000
Average dynamic value	**340**	**7,700,000**
Average static value	**170**	**6,180,000**

*Composite elastic modulus of PCC slab and stabilized base.
1 ft = 0.305 m.
1 psi/inch = 0.263 kPa/mm.
1 psi = 6.89 kPa.

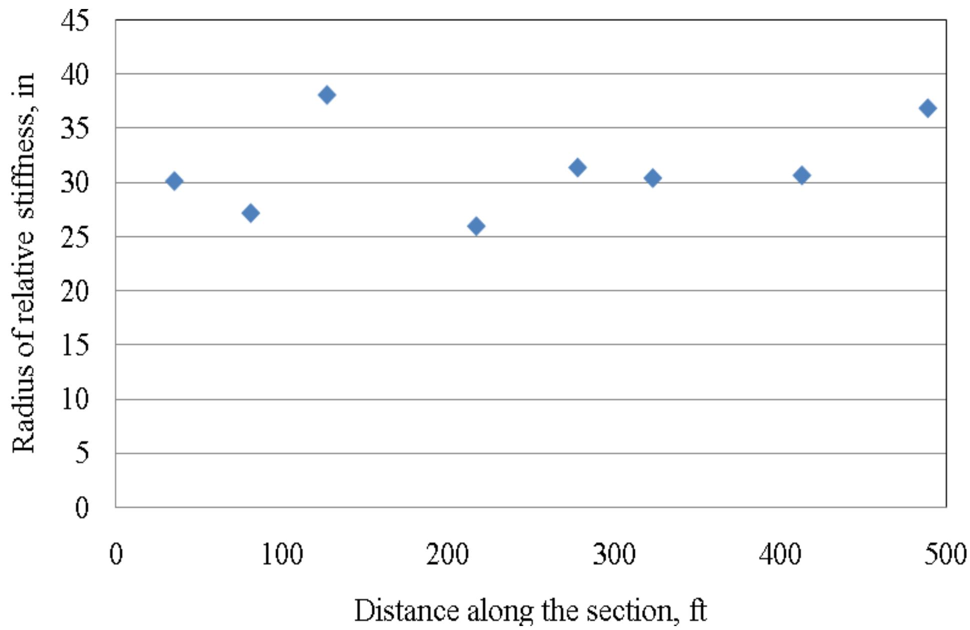
As illustrated in figure 75, there was variation in the *k*-value along the section, which was consistent with the variation observed in the normalized deflections. It appears that the *k*-value was extremely variable across the section. In general, a COV in the backcalculated *k*-value of less than 20 percent, after screening of outliers, is reasonable.(10) Significantly higher *k*-value COVs suggest significant changes in the subgrade soil type, the embankment thickness, or depth to bedrock. In this case, the COV of the backcalculated *k*-values was about 30 percent, which suggests that a variation existed in the subgrade soil type, the embankment thickness, or the water table elevation along the section. Because of the lack of information about these parameters, it was not possible to determine a more precise cause of this variation.



1 ft = 0.305 m.
1 psi/inch = 0.263 kPa/mm.

Figure 75. Graph. *k*-values along test section 05-0218.

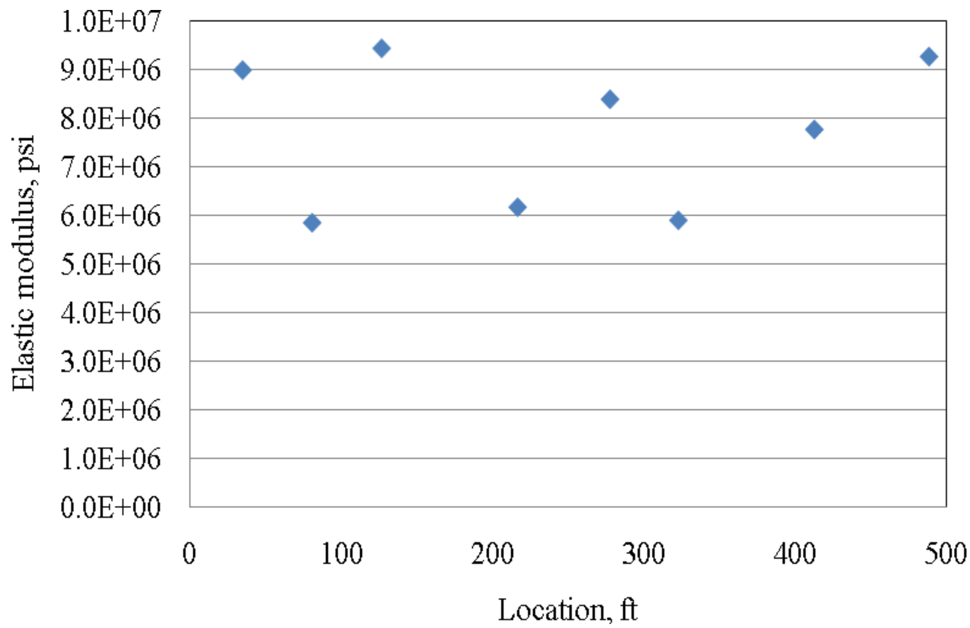
The variation of the radius of relative stiffness along the section, which is the relative stiffness of the slab to that of the pavement foundation, is presented in figure 76. For this relationship, the COV was approximately 13 percent, which corresponded to the large COV found for the backcalculated PCC elastic moduli and the *k*-values along the section.



1 inch = 25.4 mm.
1 ft = 0.305 m.

Figure 76. Graph. Radius of relative stiffness values along test section 05-0218.

The variation of the backcalculated PCC elastic modulus along the section is illustrated in figure 77. According to the backcalculated values, the COV for PCC elastic modulus was approximately 20 percent. This was higher than the 15 percent typically assumed to be acceptable for a PCC mixture design. In addition, the research team also found that, along the section, the high backcalculated *k*-values always correlated to low PCC elastic moduli. This can be an indication that the backcalculation process captured the overall stiffness, but it overestimated the *k*-value which in turn underestimated the elastic modulus of the PCC.



1 psi = 6.89 kPa.
1 ft = 0.305 m.

Figure 77. Graph. PCC elastic modulus values along test section 05-0218.

Using the AREA method, an effective modulus of an equivalent single slab was first calculated. The method presented by Ioannides and Khazanovich, discussed in chapter 5, volume I, was then used to modify the backcalculated modulus for the PCC elastic modulus and to estimate a value for the base layer modulus.(15) An additional assumption must be made relating the modulus of the base layer to the modulus of the PCC layer (β) to obtain the separate moduli. Khazanovich, Tayabji, and Darter have presented a table with typical values for the β for different base materials.(16) Using this table, a value of 0.5 was selected for β for a lean concrete base layer. The layer bonding condition also must be determined; a bonded condition was assumed for this case study.

Using the backcalculated static modulus value of 6,180,000 psi (42,580,200 kPa) for E_e, an h_e of 7.8 inches (203 mm), a β value of 0.5, and assuming a bonded condition, E_1 (PCC modulus) was calculated as 2.4 million psi (16,545,600 kPa) and E_2 (cement-treated base) was 1.2 million psi (8,272,800 kPa). These were both much lower than expected for these layers. For this case, the backcalculated elastic modulus of the slab was modified from 6,180,000 to 2,400,000 psi (42,580,200 to 16,545,600 kPa), which meant that neglecting the stiffness of the stabilized base layer while backcalculating the PCC slab stiffness resulted in an overestimation of the modulus of the PCC slab.

On the other hand, comparing the results obtained for the PCC slab stiffness using the β-method 2.4 million psi (16,545,600 kPa)) with the ones measured in the laboratory in year 2006 3.6 million psi (24,804,000 kPa)) implied that the β-method underestimated the PCC stiffness. Yet it should be noted that significant variability was observed in the PCC elastic modulus laboratory measurements. The significant drop of more than 1 million psi (6,894,000 kPa) in the modulus over 4 years can only be explained through test and measurements inconsistencies. For this reason, no judgments can be made on the modulus predicted for the base layer. However, the predicted value, 1.2 million psi (8,272,800 kPa), was lower than the default value used in the MEPDG, 2 million psi (13,788,000 kPa).

Furthermore, the selection of the β value can have a large effect on the results. Larger values of β mean a stiffer base layer and, therefore, more structural contribution to the effective modulus of the slab. When using this method, special care must be taken in selecting the β value.

Joint Load Transfer

Table 76 summarizes the calculated LTE and differential deflections based on the deflections presented in table 72. In table 76, all of the LTEs are above the acceptable level of 75 percent. In addition, all of the differential deflections were below 10 mil (0.25 mm), which supports the fact that the joints were performing well. The deflection data used for determining the LTEs were collected when pavement surface temperatures were 115 to 120 ºF (46.1 to 48.8 ºC). Such high surface temperatures increase the potential for joint lockup, so it was difficult to determine whether the joints were performing well at lower temperatures based on this testing.

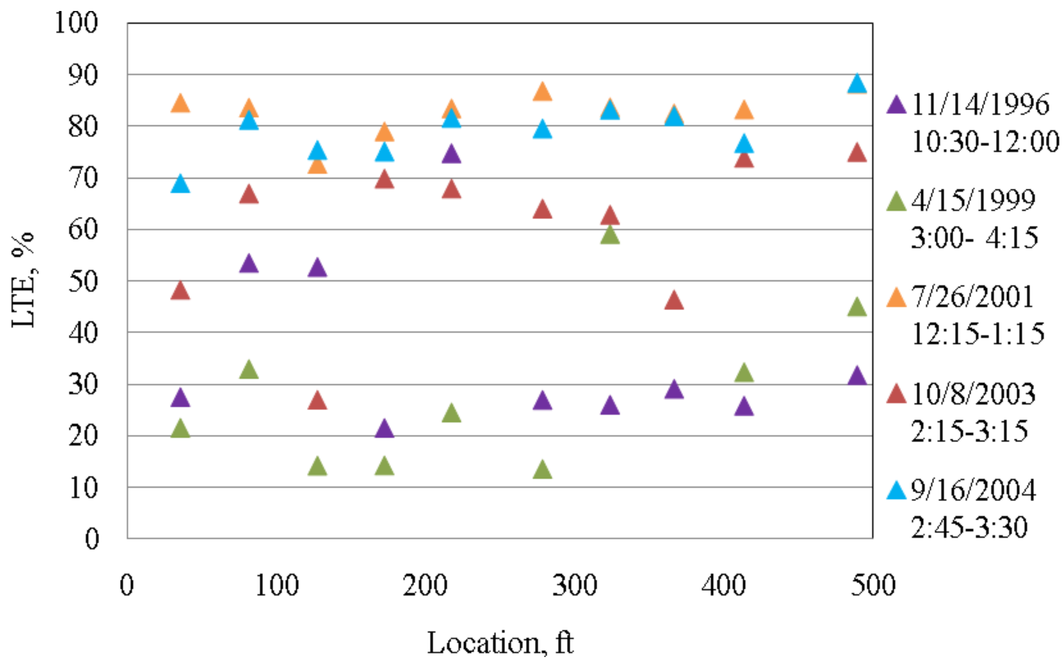
Table 76. LTEs and differential deflections for the approach and leave slabs.

Joint	Approach Slab LTE (percent)	Approach Slab Differential Deflection (mil)	Leave Slab Differential Deflection (mil)
1	78	-0.002	0.000
2	92	-0.001	0.000
3	85	-0.001	0.000
4	85	-0.001	0.000
5	92	-0.013	-0.006
6	90	-0.001	0.000
7	94	-0.001	0.000
8	93	-0.001	0.000
9	87	-0.001	0.000
10	100	0.000	0.000

1 mil = 0.0254 mm.

Variation in LTE Over Time

Figure 78 shows the LTEs determined from the available data between 1996 and 2004. As shown in the figure, the LTEs varied considerably by year. The LTEs were lowest in 1996 and 1999 and highest in the remaining years. The variation was attributed to slab temperature at the time of testing, with the high LTEs likely the result of the extremely high temperatures locking up the joints. The pavement was constructed in 1993, and FWD testing was first performed in 1996 on a cloudy day with slab temperatures ranging from 45 to 50 ºF (7.2 to 10 ºC) at the time of testing. Even though the section was constructed with dowel bars, the LTE was poor for almost every transverse joint only 3 years after construction.



1 ft = 0.305 m.

Figure 78. Graph. LTEs for different years at different testing locations.

In 1999, FWD testing was performed on a cloudy day with slab temperatures ranging from 70 to 75 ºF (21 to 24 ºC). LTEs in all of the joints were calculated to be unacceptable once again. FWD testing was performed in 2001 on a sunny day with slab temperatures ranging from 110 to 125 ºF (43 to 52 ºC). The LTEs for this day were all above the acceptable range, even though there had been no rehabilitation of the joints. Testing from 2003 indicated LTEs slightly lower than 2001. This testing was completed on a sunny day when the temperatures ranged from 95 to 100 ºF (35 to 38 ºC), which was slightly cooler than during the 2001 testing. Finally, FWD testing was completed in 2004 on a partly cloudy day when the temperature range was 115 to 120 ºF (46 to 49 ºC), and all of the joints again had an acceptable LTE. Therefore, it appears that the joints were not performing well except when the slab temperature was higher than 75 to 80 ºF (24 to 27 ºC) and the joints were locked.

Void Detection

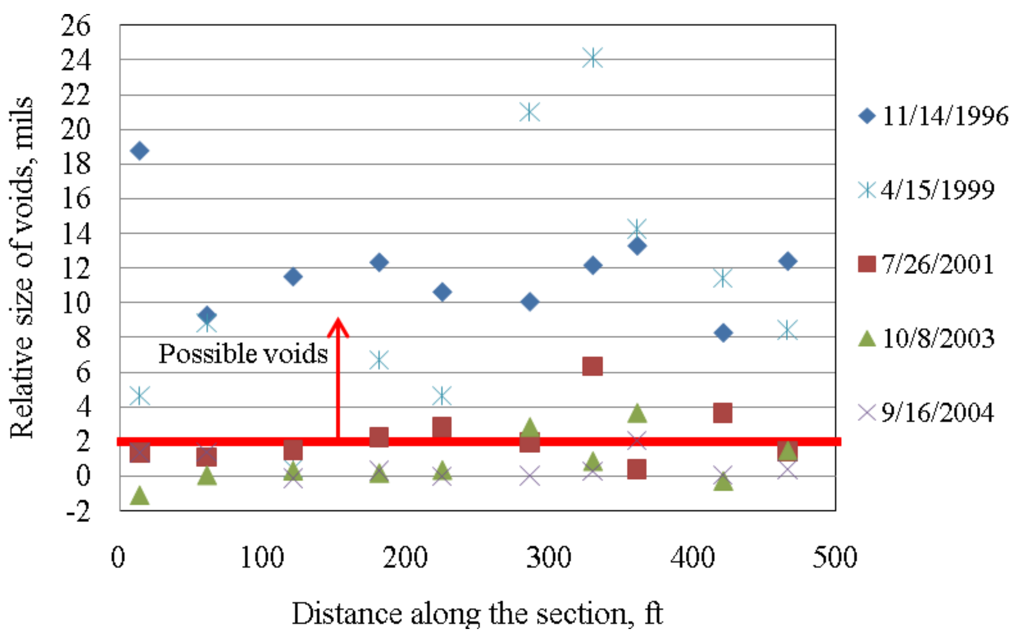
The temperature distribution throughout the depth of the slab at the time FWD testing is performed can be critical when using the deflection data for void detection. If a positive temperature gradient is present in the slab, then the corners of the slab will curl downward so that erosion of the base might have occurred while not being detected. If a negative gradient is present, causing the slab corners to curl upward, erosion of the base might be detected even it has not occurred. To reduce the potential for false positive or false negative results, it is recommended that FWD testing be performed when the slabs are flat. The research team found that the slabs were most likely to be flat when a positive temperature gradient was present in the slabs because of the existence of built-in gradients.[12] As shown in figure 62 through figure 67 and in table 77, the average temperature gradient throughout the FWD test day was positive for all the slabs. The extent of these gradients, which were assumed to be linear, is presented in table 77.

Table 77. Slab temperatures and gradients for the FWD test performed on test section 05-0218.

FWD Test Date	FWD Test Time	Average Temperature Gradient for the Day of Testing (°F/inch)	Average Slab Temperature (°F)
11/14/1996	10:30 a.m.	0.14	50.8
04/15/1996	3:00 p.m.	0.95	70.1
04/19/1999	10:00 a.m.	0.89	64.2
07/26/2001	12:00 p.m.	1.96	99.5
10/08/2003	2:00 p.m.	2.39	88.8
09/16/2004	2:00 p.m.	3.42	101.9

1 °F/inch = 0.041 °C/cm.
°F = 1.8 × °C + 32.

Voids were determined using the void detection by maximum deflection method.[11] In this method, load versus deflection response for each station is determined. Voids were detected by determining the intercepts of each line of maximum deflections with the y-axis. If an intercept was greater than 2 mil (0.05 mm), there was potentially a void under the slab at that location. The intercept values from the load versus deflection plots for the FWD tests performed for test section 05-0218 are summarized in figure 79. As shown in this figure, the majority of the testing locations exhibited possible voids for most of the testing dates.



1 mil = 0.0254 mm.
1 ft = 0.305 m.

Figure 79. Graph. Y-intercepts from void detection test results.

Based on the graph presented in figure 79, few voids were identified in 2001, 2003, and 2004, while voids could be detected at almost all joints in 1996 and 1999. This can be explained by the temperature gradients present in the slabs at the time of testing, as listed in table 77. According to these temperature measurements, the temperature gradient present at the time of testing was higher with each consecutive year. It appears that a negative built-in gradient might have been constructed into the slab. Void detection testing while the corners of the slab are curled upward can result in a false positive, as was observed in 1996 and 1999. As the gradient becomes increasingly positive at the time of testing, the slab corners curl downward until the slab is flat. It is possible for the slab corners to curl downward, with increasing positive gradients, to the point where the slab corners are penetrating into the lower layer. When this occurs, the y-intercept typically becomes negative, as observed in 2003 and 2004 when large positive gradients were present at the time of testing.

Backcalculation Modeling Issues and Recommendations

The pavement structure for this section was a four-layer system or five-layer system if a rigid layer were present. The closed-form solutions for rigid pavements at the time of this report are not well suited for more than two bound layers and multiple unbound layers. As a result, the aggregate subbase layer is included in the determined *k*-value, as is the presence of any rigid layer. In this case, the aggregate subbase layer was thin, so the influence on the determined subgrade *k*-value should be minimal. Because it was a thin layer below a rigid layer, determining the layer modulus would be problematic with any backcalculation method, so combining it with the subgrade was an acceptable step for backcalculation. The available data indicated no near-surface rigid layer, so assuming any influence of a rigid layer in the composite *k*-value was acceptable for this example.

In addition, with the AREA method, an effective modulus for an equivalent single plate was first determined, and the individual bound layer moduli were then determined using a modular ratio. The determination of the separate moduli is influenced by the selection of the modular ratio and the assumed bonding condition. While general modular ratios are available, additional project-specific data assisting with establishing these parameters would be the best option.

Comparison of Backcalculation and Laboratory Testing Results

To assist in evaluating which layer characteristics were appropriate to use in the MEPDG software, the results of the backcalculation (field tests) were compared with results obtained from laboratory testing conducted as part of the LTPP Program.

Bound Materials

As presented in table 75, the average backcalculated static PCC elastic modulus is 6,180,000 psi (42,718,000 kPa) whereas, according to the LTPP Program data, the laboratory measured static modulus for this section in 2006 is reported as 3.6 million psi (24,804,000 kPa). This difference is clearly significant and can be explained in two ways:

- Based on the data available in the LTPP Program database for this section, the number of samples used to obtain this result was not statistically sufficient. Moreover this result was inconsistent with the ones measured 4 years before.

- It has been shown that, when using the AREA method in the case of stabilized base layer existing beneath the PCC slab, the base layer influences the slab's responses during the

FWD testing; therefore, when using the AREA method to backcalculate the PCC stiffness, some unrealistically high values are obtained for the PCC modulus. This is believed to be the case for this study. The very high backcalculated stiffness for the PCC layer was actually somewhat representative of the stiffness of an effective layer consisting of both the stabilized base and the PCC layer.

Unbound Materials

The granular subbase was a relatively thin layer directly under a stiff layer, which made it problematic to determine a modulus value from backcalculation. In addition, the influence of a thin unbound layer compared with the contribution of the underlying subgrade was minimal. Thus, the modulus of the subbase assumed the material's default value of 206,820 kPa (30,000 psi).

The resilient modulus of the subgrade ranged from approximately 9,000 to 19,000 psi (62,046 to 130,986 kPa) for a bulk stress between 8 to 28 psi (55 to 193 kPa), as previously shown in figure 68. Considering the fact that resilient modulus testing was not performed on any samples from test section 05-0218 and recognizing the difficulty in determining the resilient modulus for the unknown confinement condition, the typical resilient modulus suggested by the MEPDG for AASHTO A-2-4 subgrade, 16,500 psi (113,751 kPa), was adopted. The average backcalculated composite *k*-value corresponded to a modulus of approximately 19,000 psi (130,986 kPa). This value corresponded with the laboratory values at the highest bulk stress, which was approximately 15 percent higher than the default material value.

RECOMMENDED REHABILITATION DESIGN INPUTS

The objective of this study was to assess the reliability of using FWD backcalculated values in design based on the comparison between designs with inputs from laboratory tests and designs with inputs obtained from FWD backcalculation. For this case study, rehabilitation designs were analyzed for an HMA overlay, an unbonded JPCP overlay, and a bonded JPCP overlay.

Inputs for the Existing Layers

The general inputs used for the design program were based on the available LTPP Program data (as previously summarized), estimated inputs from standard specifications, and on default values within the program when data were not available from other sources. The primary inputs that were evaluated for this case study were those corresponding to properties obtained from the backcalculation process (primarily PCC elastic modulus and *k*-value).

As previously discussed, the majority of the slabs had cracked by 2004 (see table 60). In terms of the MEPDG design, the percentage of cracked slabs before restoration was then assumed to be 50 percent. Almost all of the transverse cracks were working, so they must be repaired before any overlay was applied. The percent of slabs cracked after the restoration was therefore 0 percent. The climatic modeling was used for the unbound layers. While a "Representative value" is also allowed, testing data and backcalculation results were not available for all of the months or seasons to estimate an appropriate representative value.

Inputs for the HMA Overlay

The overall design level for an HMA overlay over JPCP is level 3 and cannot be changed. The input level for the individual layers can be adjusted, with levels 1 through 3 for the HMA overlay and existing PCC, and levels 2 and 3 for unbound materials. For the HMA overlay analysis, the flexural strength and elastic modulus were entered for the existing PCC as required in level 1, and the unbound layer data were entered as level 3.

From the design runs performed during this case study, the HMA overlay design appeared to be very sensitive to the HMA properties of the overlay. The results presented later in this chapter are based on the following HMA mixture properties: the asphalt binder grade was selected as PG 64-22, the effective binder content was 10 percent, and the air void content was 4 percent. The aggregate gradation is presented in table 78.

Table 78. Aggregate gradation for the new HMA overlay alternative design.

Sieve Size	Value
$^3/_4$ inch (percent retained)	0
$^3/_8$ inch (percent retained)	18
No. 40(percent retained)	33
No. 200 (percent passing)	4

1 inch = 25.4 mm.

Table 79 summarizes the performance criteria used in the assessment of the HMA overlay design. These criteria are recommended by the MEPDG. A reliability level of 90 percent was assumed, as well as a 20-year performance period.

Table 79. Performance criteria for a HMA overlay on a rigid pavement.

Performance Criteria	Limit
Initial IRI (inches/mi)	63
Terminal IRI (inches/mi)	172
Transverse cracking (percent slabs cracked)	15
AC surface down cracking (longitudinal cracking) (ft/mi)	2,000
AC bottom up cracking (alligator cracking) (percent)	25
AC thermal fracture (transverse cracking) (ft/mi)	1,000
Chemically stabilized layer (fatigue fracture)	25
Permanent deformation (AC only) (inches)	0.25
Permanent deformation (total pavement) (inches)	0.75
Reflective cracking (percent)	100

1 inch/mi = 0.0158 m/km.
1ft/mi = 0.19 m/km.
1 inch = 25.4 mm.
AC = asphalt concrete.

Inputs for the Unbonded JPCP Overlay

The same PCC properties of the existing PCC layer were used for the JPCP unbonded overlay design. The properties based on laboratory measurements were as follows: unit weight of 138 lb/ft^3 (2.2 g/cm^3), Poisson's ratio of 0.15, CTE of 5.2×10^{-6}/°F (9.4×10^{-6}/ºC), and an elastic modulus of 3.6 million psi (24,804,000 kPa). The PCC mixture design used for the existing PCC layer was also used for the overlay, such as a water-to-cement ratio of 0.38 and a cement content of 719 lb/ft^3 (11.5 g/cm^3). The existing PCC was not being rubblized or crack-and-seated, so the fracture type was left as "User Defined." In addition, the existing PCC layer was fixed as a level 3 input in an unbonded PCC overlay design.

The PCC slabs were designed to be 15 ft (4.6 m) long and 12 ft (3.7 m) wide to be compatible with the existing PCC layer, and the joints were sealed with silicon and doweled with 1.25-inch (31.8-mm) bars. With respect to the required bond breaker layer (2 inches (50.8 mm) thick, as suggested by the MEPDG), the same HMA properties were employed as were used in the HMA overlay design alternative.

The performance criteria for the unbonded JPCP overlay suggested by the MEPDG are presented in table 80. A 90-percent reliability level and 20-year performance period were used.

Table 80. Performance criteria for a PCC overlay on a rigid pavement.

Performance Criteria	Limit
Initial IRI (inches/mi)	63
Terminal IRI (inches/mi)	172
Transverse cracking (percent slabs cracked)	15
Mean joint faulting (inches)	0.12

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.

Inputs for the Bonded JPCP Overlay

The inputs, as well as the performance criteria for the JPCP bonded overlay, were the same as those used for the unbonded JPCP overlay with the difference that the bonded overlay did not include an asphalt interlayer. In addition, the input level for the existing PCC layer can be based on levels 1 through 3 for a bonded PCC overlay.

REHABILITATION DESIGN RESULTS

Three types of overlays were designed as the rehabilitation for the existing JPCP, namely an HMA overlay, an unbonded JPCP overlay, and a bonded JPCP overlay. To validate the reliability of using FWD backcalculation results in MEPDG designs, two design alternatives were originally planned for each kind of overlay, the first of which would employ the laboratory testing results as inputs, and the second would employ the backcalculated values as inputs. However, the research team later found (from a study on the conversion of layer moduli into the *k*-value, which is part of this report) that the MEPDG software did not appear to always use the entered dynamic *k*-value to represent the stiffness of all the layers beneath the base layer as is stated in the MEPDG documentation. Sometimes, the stiffness of the base layer is also taken into account in the composite *k*-value. Therefore, a third alternative had to be designed to learn how the MEPDG uses the input *k*-value. The details of the following three design alternatives are presented:

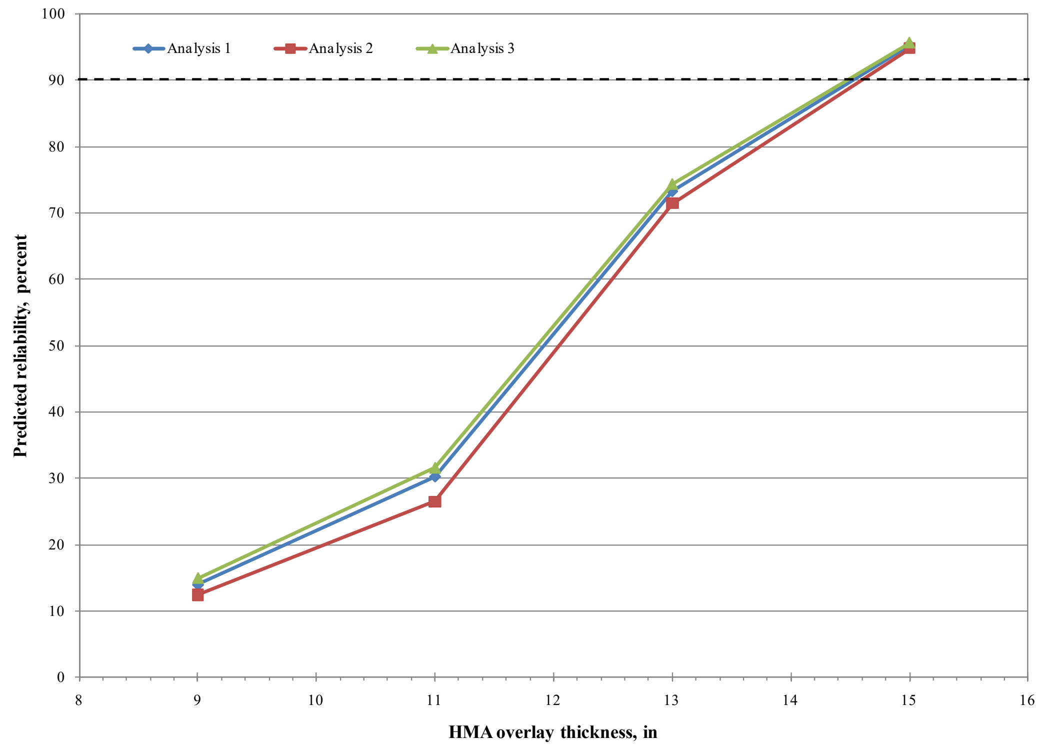
- **Alternative 1**: The first design alternative was a benchmark, where the laboratory-measured or recommended material properties were used as inputs in the MEPDG. For this alternative, the backcalculated dynamic *k*-value was not used to represent the stiffness of the supporting layers.

- **Alternative 2**: In the second design alternative, the backcalculated values, in terms of the static elastic modulus for the existing PCC layer and the dynamic *k*-value for the supporting layers, were used as inputs, assuming that the *k*-value reflected the stiffness of all of the layers beneath the base and the backcalculated elastic modulus reflected only the stiffness of the PCC layer (see table 75). Based on the FWD testing, the average static backcalculated elastic modulus calculated for the existing PCC layer was 6,180,000 psi (42,580,200 kPa). In the bonded PCC and HMA overlay designs, the modulus of rupture for the existing PCC layer is also required in the MEPDG. This value was obtained using the correlation between the elastic modulus and the modulus of rupture provided within the MEPDG. Based on this correlation, a modulus of rupture of 1,060 psi (7,303 kPa) was obtained for an elastic modulus of 6,180,000 psi (42,580,200 kPa). The average backcalculated dynamic *k*-value was 340 psi/inch (100 kPa/mm) and was directly entered in the rehabilitation section of the design.

- **Alternative 3**: The individual bound layer moduli using a modular ratio value of 0.5 was used, with the PCC modulus being 2.4 million psi (16,545,600 kPa) and the lean concrete base modulus being 1.2 million psi (8,272,800 kPa).

HMA Overlay of JPCP

In the MEPDG HMA rehabilitation design, the thickness of the overlay was varied until the thinnest HMA layer was obtained that still satisfied the selected performance criteria. The HMA overlay thickness determined to meet all criteria at the 90-percent reliability level with the assumed inputs was 15 inches (381 mm), as illustrated in figure 80, regardless of the different sets of inputs described for alternatives 1 through 3. The rutting criterion is what drives this excessively thick HMA overlay, which is considered impractical from a constructability standpoint.



1 inch = 25.4 mm.

Figure 80. Graph. Summary of predicted reliabilities for surface rutting of HMA overlay.

Thus, in this case study, the structural capacity of the existing PCC pavement did not control the design, but rather it was controlled by the HMA overlay material properties. Therefore, the final HMA overlay thickness requirement, regardless of the underlying layer properties selection, the same. This implies that changes in the HMA mix design would substantially affect the design thickness of the HMA overlay. However, several variations in mix properties were analyzed (such as varying binder grades, binder content, air void content, etc.), and rutting continued to control the design results. If rutting was not considered, an HMA overlay thickness of 4 inches (102 mm) was obtained that successfully meets the other performance criteria.

The research team also found that the *k*-values reported in the output file were the same whether or not the dynamic *k*-value was used as an input. The reported *k*-value appears to be based on the entered subgrade layer modulus, which was confirmed by running different moduli with and without the dynamic *k*-value entered. Additional design runs were executed with low and high base layer moduli to determine how the *k*-value was calculated. The *k*-values were identical for cases with varying base layer stiffness, indicating the stiffness of the base was not included in the *k*-value calculation.

Design Results for the Unbonded PCC Overlay

The design thickness of the unbonded JPCP overlay was the thinnest slab that still satisfied the selected performance criteria (shown in table 80). However, the use of unbonded JPCP overlays thinner than 7 inches (178 mm) is not recommended by the MEPDG. This is because the unbonded overlay acts independently from the supporting layers, so its minimum allowable thickness should be limited to ensure its structural capacity as well as its functionality. As summarized in table 81, the research team found that a 7-inch-(178-mm)-thick or thicker JPCP unbounded overlay can meet all of the criteria for the first two alternatives; however, because the elastic modulus of the PCC estimated in alternative 3 was very low, the structure failed in transverse cracking and requires a greater thickness. In that case, a 15-inch (381-mm) PCC overlay would be required to satisfy the selected performance criteria.

Table 81. Design thickness of unbonded PCC overlay for three design alternatives.

Alternative	Design Thickness (inches)
1	7
2	7
3	7

1 inch = 25.4 mm.

Additional design runs for unbonded PCC overlays were made to assess the *k*-value to determine whether it was computed similarly as was observed in the HMA overlay assessment. Based on the additional design runs, the mean of the calculated *k*-values agreed well with the entered *k*-value suggesting that the MEPDG uses the manually entered *k*-value for unbonded JPCP overlay designs. However, when no *k*-value was designated, it was not explicit which layers were taken into account in the calculation of the *k*-value. However, a noticeable difference was found between using a low and high existing PCC modulus, which might indicate that the stiffness of the existing PCC was involved in the calculation of the *k*-value.

Design Results for the Bonded PCC Overlay of JPCP

The design thickness of the bonded JPCP overlay was determined by reducing the thickness of the overlay until the performance criteria shown in table 800 were no longer fulfilled. As presented in table 82, the design thickness for all three alternatives is 3 inches (76 mm). Thinner slabs were not recommended by the MEPDG version used in the analysis (Note that the newer version 1.1 of the MEPDG allows a 1.5-inch minimum).

Table 82. Design thickness of bonded PCC overlay for three design alternatives.

Alternative	Design Thickness (inches)
1	3
2	3
3	3

1 inch = 25.4 mm.

The research team found that the correlations between the elastic modulus and modulus of rupture of PCC used by the MEPDG were different for bonded and unbonded overlays. In other words, using the same modulus of rupture for bonded and unbonded cases resulted in different elastic moduli.

As with the HMA and unbonded PCC overlay alternatives, additional design runs were executed to determine the *k*-value calculations. The mean *k*-value for a high and low modulus base material noticeably varied. Therefore, it seems that the modulus of the base layer was considered in the calculation of the *k*-value for bonded JPCP overlay designs, which agreed with the assumptions made for bonded JPCP overlays. It is also apparent that the calculated *k*-values matched the entered dynamic *k*-value.

Evaluation of Design Results

An HMA overlay was designed for rehabilitation of test section 05-0218 in three ways. First, measured laboratory moduli of the pavement layers were used for the design. Second, using available FWD test results, the elastic modulus of PCC and dynamic effective *k*-value were backcalculated and used for the design. Finally, the backcalculated PCC elastic modulus was modified and reduced to account for the contribution of the stabilized base layer. The required thickness was found to be the same for the three alternatives. This is attributed to the HMA overlay requirement being controlled by HMA materials performance and not the underlying structural capacity in this case. In addition, the research team found that it did not appear the dynamic *k*-value was being used when it was used as an input.

An unbonded PCC overlay design was also analyzed using the same three alternatives. A 7-inch (178-mm) PCC overlay met the selected performance criteria over the 20-year design life for alternatives 1 (laboratory-based inputs) and 2 (backcalculated PCC modulus and *k*-value). However, alternative 3 (modified PCC elastic modulus and base modulus) required a much thicker unbonded overlay (15 inches (381 mm)). The modified elastic modulus value was as low as 2.4 million psi (16,536,000 kPa), which did not provide enough structural capacity to adequately carry the future traffic loadings.

The final rehabilitation design method used was a PCC bonded overlay. A 3-inch (76-mm) bonded overlay was determined to be sufficient to meet the selected design criteria for all three design alternatives. A 76-mm (3-inch) PCC overlay met the design criteria even in the third alternative, which included a lower strength PCC, which can be explained by the fact that the existing pavement and the overlay performed as a monolithic section. In general, before the bonded overlay is placed on top of an existing JPCP, all existing distresses are rehabilitated and a good bond is achieved between the new and the existing pavement. Therefore, in the case of alternative 3, the new overlay (3 inches (76 mm)) and the existing pavement (7.8 inches (198 mm)) combined to produce a composite PCC layer placed on a strong base layer (lean concrete with modulus of 2 million psi (13,788,000 kPa). This section proved to have sufficient structural capacity to carry the loads over the design life with a reliability of 90 percent.

For unbonded overlays, the existing slab acts a base layer. The low modulus defined for the overlay in the case of alternative 3 (2.4 million psi (16,536,000 kPa)) required the slab to have a relatively higher thickness to provide sufficient structural capacity to bear the cumulating fatigue damage. Although the existing PCC slab with a modulus of 2.4 million psi (16,536,000 kPa) provided a strong base layer for the slab, numerous runs of the MEPDG revealed that a 15-inch (381-mm) overlay would be required to meet the transverse cracking criteria of 15 percent with a 90-percent reliability.

SUMMARY

Test section 05-0218, located in Saline County, AR, was selected for rehabilitation case study of a PCC slab on stabilized base layer. The distress survey data available for this section in the LTPP Program database indicated that this section was in poor condition (50 percent of the slabs had working transverse cracking) in 2007.

The most recent FWD test data available for test section 05-0218 was from 2004. Using the LTPP Program database, input parameters required for the overlay designs were compiled. Not all required data were available for test section 05-0218; therefore, data from other sections of project 05-0200 were used. Also, the USGS Web site was used as a reference for data concerning the depth of the water table.

Using the FWD test data, the elastic modulus of PCC and the dynamic effective *k*-value were backcalculated for the section. These values were calculated to be 6,180,000 psi (42,580,200 kPa) and 340 psi/inch (100 kPa/mm), respectively. The backcalculated PCC elastic modulus was later corrected for the effect of the stabilized base layer. The corrected value of the PCC elastic modulus obtained was 2.4 million psi (16,536,000 kPa), which was lower than the measured value of 3.6 million psi (24,804,000 kPa). The lower PCC modulus appears to influence only the required thickness of the unbonded PCC overlay for this case study, as summarized in table 81. With the exception of the unbonded PCC overlay alternative, there was no apparent difference between the use of laboratory-based and backcalculation-based inputs based on the results of the rehabilitation designs.

Other observations made during the design analysis process include the following:

- For the HMA overlay design it seems impossible to draw a definite conclusion about the constituents contributing to the *k*-value being reported in the design output (and assumed to be used in the design calculations). The entered dynamic *k*-value did not appear to be used in the determination of *k*-value. The difference of the calculated *k*-values between a low and high base stiffness was so slight it appears that the base layer stiffness was not considered in the calculated *k*-value.

- With the unbonded JPCP overlay design, a noticeable difference was found in the calculated *k*-value between a low and high existing PCC modulus, which might indicate that the stiffness of the existing PCC was involved in the calculation of the *k*-value.

- The modulus of the base layer appears to be considered in the calculation of the *k*-value for bonded PCC overlay designs, which agreed with the assumptions discussed in the MEPDG. In addition, the calculated *k*-values matched the entered dynamic *k*-value.

- For this case study, a low PCC modulus only influenced the unbonded PCC overlay thickness requirement, more than doubling the required thickness. The PCC modulus did not appear to influence the other overlay design options.

CHAPTER 5. CASE STUDY 5: WHITETOPPING

PROJECT OVERVIEW

In this case study, a rehabilitation alternative that consisted of a JPCP layer over an existing HMA layer (sometimes referred to as whitetopping) was considered for LTPP Program test section 30-0113 (the project also used in case study 1). Project 30-0100 was located on I-15 near Great Falls, MT, and consisted of 12 test sections. Test section 30-0113 was a flexible pavement cross section with an HMA surface and aggregate base layer over subgrade. The original pavement, nominally a 4-inch (102-mm) HMA surface on a 8-inch (203-mm) aggregate base, was constructed in 1997.

TEST SECTION DATA

The structure and all other properties of the section used in the alternatives presented in previous sections of this report were kept identical.

ANALYSIS AND INTERPRETATION OF FWD TESTING DATA

This section presents the data checks and backcalculation analysis of the FWD data used in this case study.

Preprocessing Deflection Data

Before conducting the backcalculation analysis, the available deflection data were screened. The dataset used for this case study was assessed as part of case study 1 and was not reassessed for this case study.

Backcalculation Analysis

Backcalculation of test section 30-0113 outer wheelpath deflection data was performed using linear, static layered elastic analysis. The backcalculation analysis is discussed in case study 1; the use of the backcalculation inputs in the MEPDG software for whitetopping is covered in this case study.

REHABILITATION DESIGN RESULTS

The overall design level for this rehabilitation (PCC over existing HMA) was level 3, which cannot be adjusted within the MEPDG program. The required layer inputs used in the MEPDG are dependent on the overall design level selected for design. The bound layer inputs can be levels 1, 2, or 3, and the unbound layer inputs can be levels 2 or 3.

Design Criteria and General Inputs

The general inputs used for the design program were based on the available LTPP Program data (as previously summarized), estimated inputs from standard specifications, and default values within the program when data were not available from other sources. The primary inputs that were evaluated for this case study were those corresponding to properties obtained from the backcalculation process (aggregate base modulus, subgrade modulus, and *k*-value).

The performance criteria used for the design of this whitetopping alternative are listed in table 83. An overall reliability level of 90 percent was selected for this case study as well as a 20-year analysis period.

Table 83. MEPDG performance criteria for JPCP pavement.

Performance Criteria	Limit
Initial IRI (inches/mi)	63
Terminal IRI (inches/mi)	172
Transverse cracking (percent slabs cracked)	15
Mean joint faulting (inches)	0.12

1 inch/mi = 0.0158 m/km.
1 inch = 2.54 cm.

The MEPDG (version 1.003) was used to design the whitetopping alternative for the LTPP Program test section 30-0113. Nationally calibrated performance models were used. For the concrete layer properties and the pavement design features, level 3 default values were used. Table 84 and table 85 summarize these values.

Table 84. Concrete layer properties used for the whitetopping design.

Concrete Property	Value
Unit weight (lb/ft^3)	150
Poisson's ratio	0.2
CTE ($\times 10^{-6}$/ºF)	5.5
Cementitious material content, (lb/yd^{3})	600
Water to cement	0.42
Aggregate type	Limestone
Curing method	Curing compound
28-day PCC modulus of rupture (psi)	690

1 lb/ft^3 = 0.0160 g/cm^3.
°F = 1.8 × °C + 32.
1 lb/yd^3 = 0.593 kg/m^3.
1 psi = 6.89 kPa.

Table 85. Design features used for the whitetopping design.

Parameter	Value
Dowel diameter (inches)	1
Dowel spacing (inches)	12
Sealant type	Liquid
Joint spacing (ft)	15

1 inch = 25.4 mm.
1 ft = 0.305 m.

Incorporation of Backcalculation Results With the MEPDG Software

This section illustrates the procedure for using the backcalculated results as inputs for unbound and bound layers of the existing pavement structure within the MEPDG software.

Unbound Layers

The JPCP overlay rehabilitation design for an existing flexible pavement requires individual layer inputs, including layer modulus and material properties. The dynamic subgrade *k*-value is an optional input and is entered as part of the "Rehabilitation" screen input (see figure 81). The backcalculation results of the existing pavement were performed using layered elastic backcalculation, so a *k*-value was not determined. A *k*-value can be correlated to the determined subgrade modulus using available correlations, but it should be relatively similar to that determined internally by the MEPDG software. The unbound layer moduli can be entered in the layer input screen, but the adjusted value needs to be used because there is no correction factor input for this rehabilitation design.

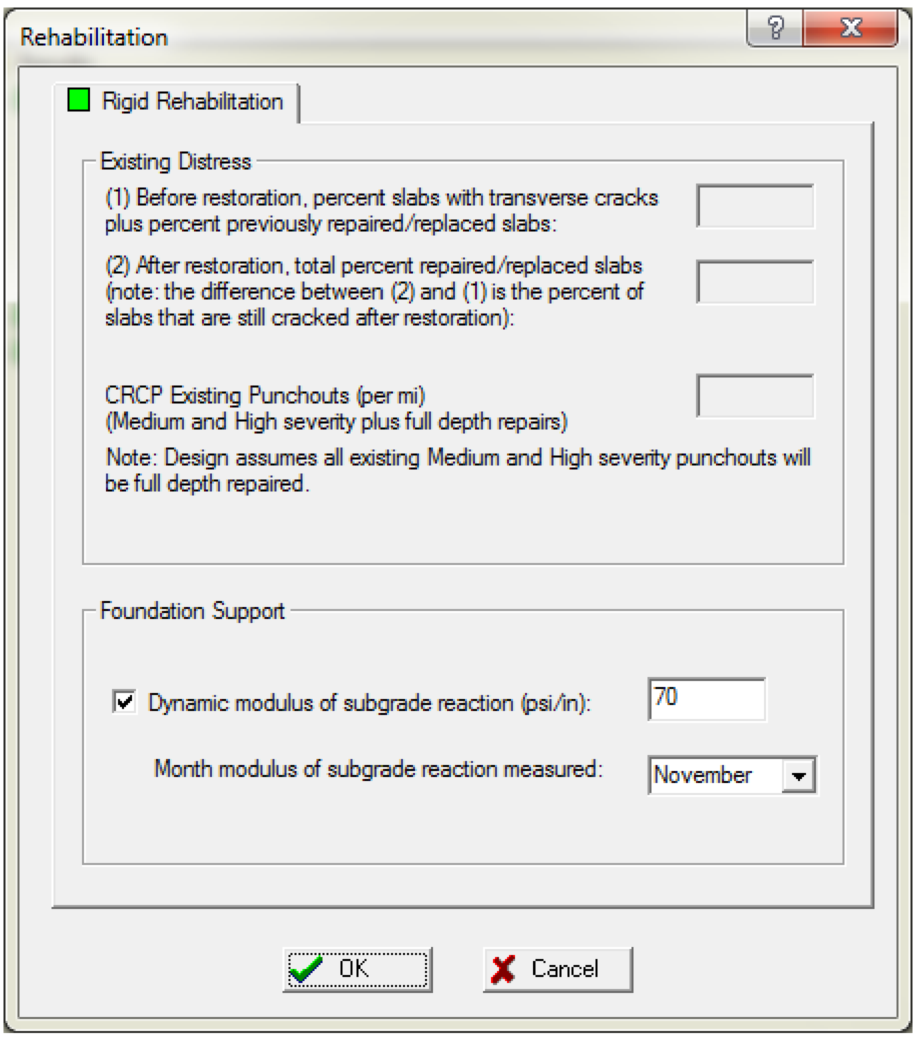


Figure 81. Screen Capture. Rehabilitation screen for JPCP over HMA.

Bound Layers

Although the existing HMA layer modulus was available from backcalculation, the MEPDG software did not use it as an input regardless of level hierarchy selected.

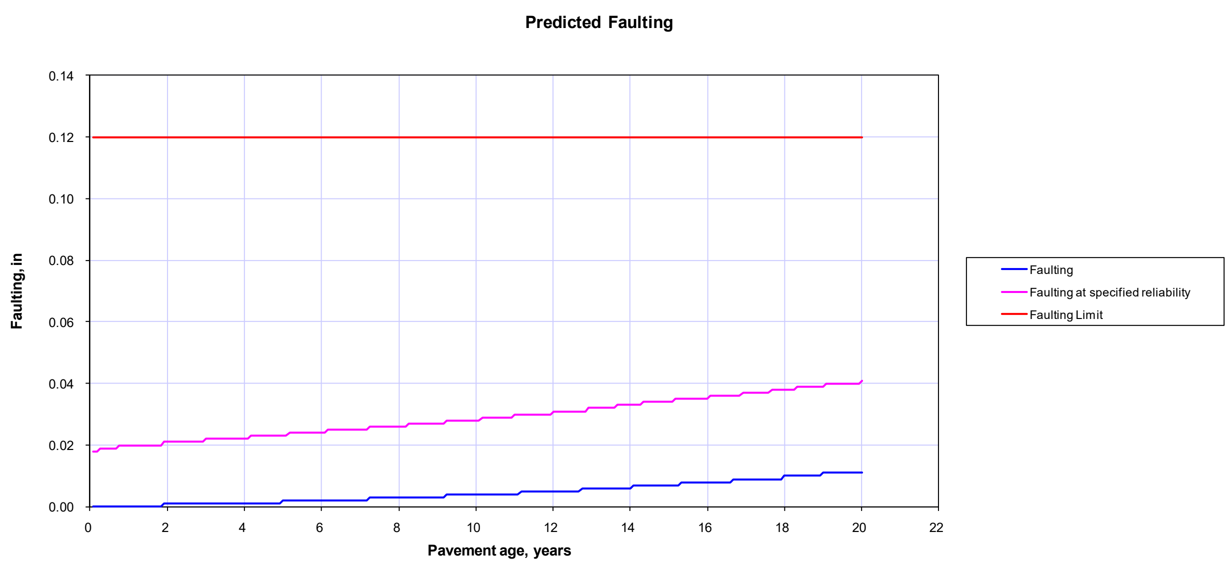
Evaluation of Design Results

Using the properties mentioned in the previous section for the concrete overlay, a 7-inch (178-mm)-thick concrete overlay was determined to be suitable for this section in accordance with performance criteria presented in table 83. The MEPDG results for this analysis are shown in table 86 and in figure 82 through figure 84.

Table 86. Results of the whitetopping analysis design for section 30-113.

Performance Criteria	Distress Target	Reliability Target	Predicted Distress	Predicted Reliability	Acceptable
Terminal IRI (inches/mi)	172	90	85.9	99.84	Pass
Transverse cracking (percent slabs cracked)	15	90	4	95.91	Pass
Mean joint faulting (inches)	0.12	90	0.011	99.999	Pass

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.



1 inch = 25.4 mm.

Figure 82. Graph. Joint faulting versus time for the whitetopping analysis design for test section 30-0113.

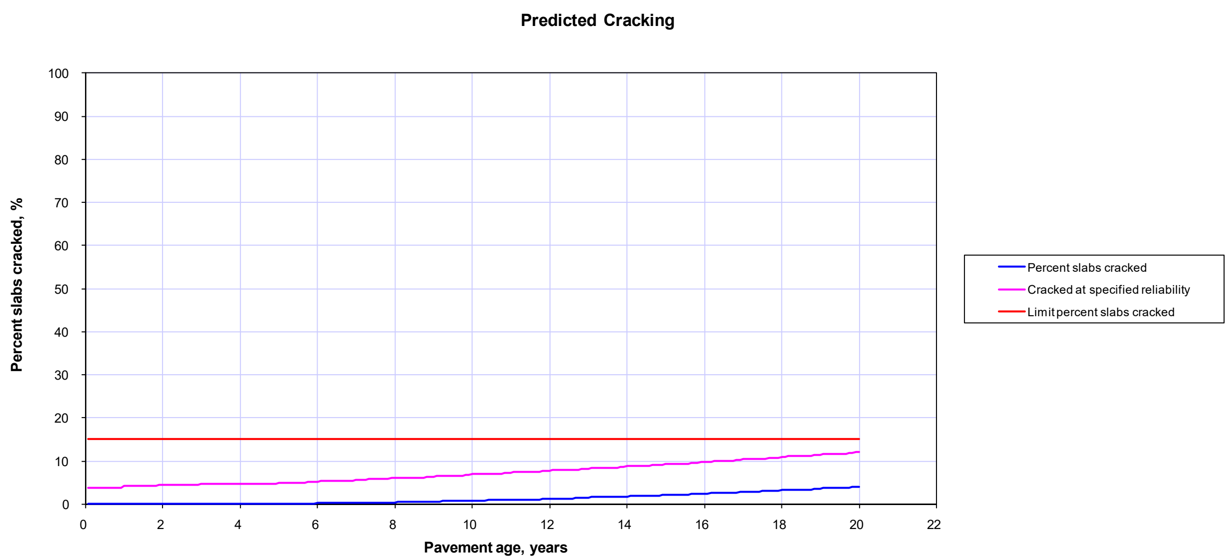
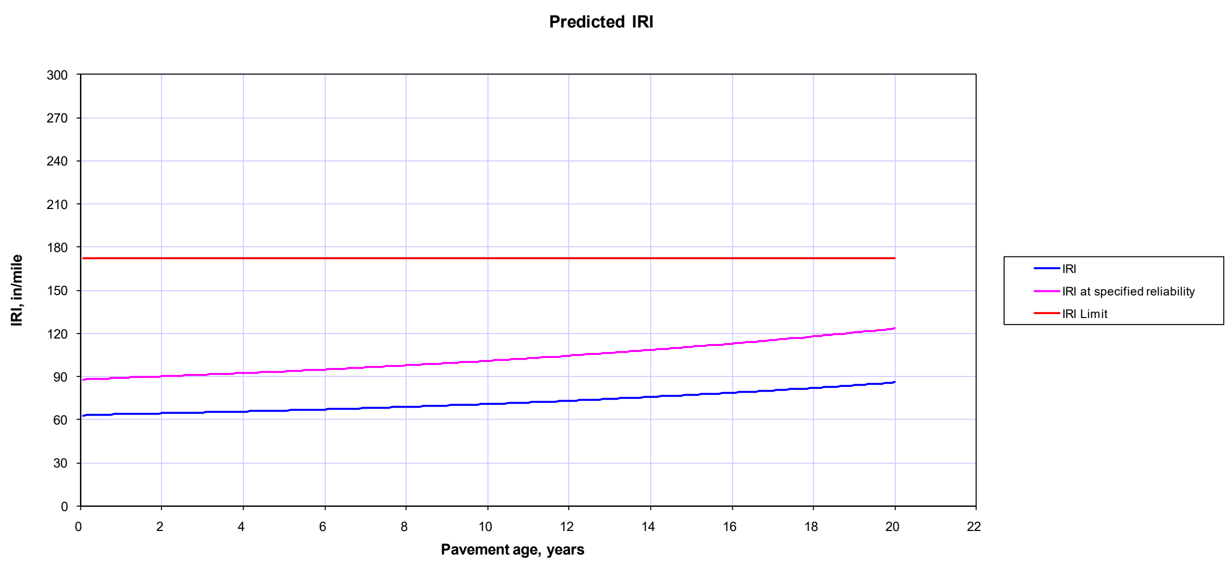


Figure 83. Graph. Transverse cracking versus time for the whitetopping analysis design for test section 30-0113.



1 inch/mi = 0.0158 m/km.

Figure 84. Graph. IRI versus time for the whitetopping analysis design for test section 30-0113.

An attempt to use a thinner overlay failed because of transverse cracking. Table 87 shows the results obtained for a 6-inch (152-mm) concrete overlay design.

Table 87. Results of the 152-mm (6-inch) whitetopping analysis design for test section 30-0113.

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (inches/mi)	172	90	109.1	95.98	Pass
Transverse cracking (percent slabs cracked)	15	90	32.9	4.37	Fail
Mean joint faulting (inches)	0.12	90	0.008	99.999	Pass

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.

FOUNDATION MODULUS CALCULATION

The procedure used within the MEPDG to calculate the composite dynamic *k*-value for a whitetopping pavement was not specified in the MEPDG documentation. No information was found regarding how the pavement structure was modeled within the MEPDG or which layers of the pavement were used in determining the composite *k*-value. In an attempt to determine what layers were used within the MEPDG to establish the composite *k*-value when analyzing a concrete overlay on an HMA pavement, four additional runs were performed.

The MEPDG includes the following two options for defining the pavement structure:

- Provide the moduli for each layer and allow the MEPDG to calculate a *k*-value.
- Provide the moduli for each layer and provide a *k*-value.

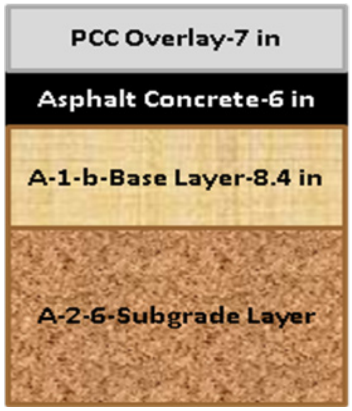
Four additional runs were evaluated to help determine whether the stiffness of the base was included in the composite *k*-value calculation within the MEPDG. The pavement structures used for these four cases are provided in figure 85. These four runs also helped to determine whether the *k*-value input was actually used within the MEPDG. This was in question because there was no valid method available for backcalculating a *k*-value when the FWD testing was performed on an existing HMA pavement. The characteristics of these runs were as follows:

- **Run 1**: *k*-value calculated by the MEPDG; aggregate base modulus of 13,300 psi (91,700 kPa).

- **Run 2**: *k*-value of 70 psi/inch (19 kPa/mm); aggregate base modulus of 13,300 psi (91,700 kPa).

- **Run 3**: *k*-value calculated by the MEPDG; aggregate base modulus of 2 million psi (13,790,000 kPa).

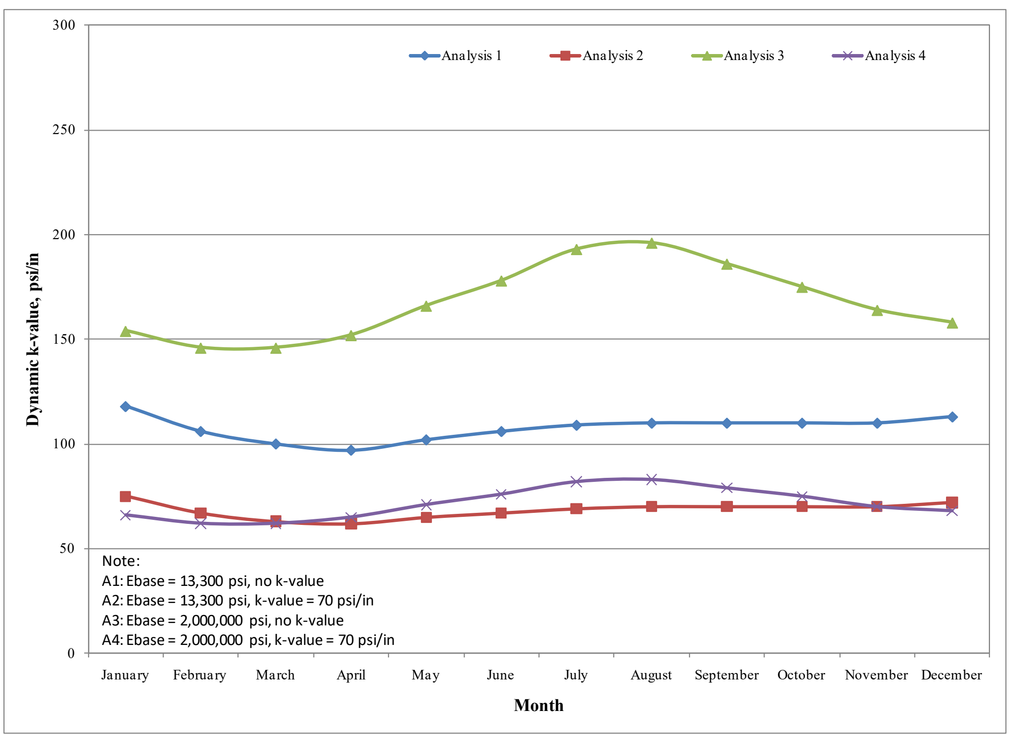
- **Run 4**: *k*-value of 19 kPa/mm (70 psi/inch); Aggregate base modulus of 2 million psi (13,790,000 kPa).



1 inch = 25.4 mm.

Figure 85. Diagram. Pavement structure used for the four runs.

The monthly *k*-values calculated within the MEPDG for each of these runs are summarized in figure 86. Comparing the results of runs 1 and 3 shows that the difference between an aggregate base stiffness of 13,300 psi (91,700 kPa) and 2 million psi (13,790,000 kPa) resulted in *k*-values calculated by the MEPDG that were different. Therefore, it can be concluded that the aggregate base stiffness was considered in the calculation of the composite *k*-value within the MEPDG. A *k*-value of 70 psi/inch (19 kPa/mm) was provided as an input in the MEPDG for runs 2 and 4. The MEPDG output shows that this value is indeed used as the composite *k*-value regardless of the aggregate base stiffness; however, there were slightly different climate adjustments.



1 psi = 6.89 kPa.
1 psi/inch = 0.263 kPa/mm.

Figure 86. Graph. Monthly *k*-values provided within the MEPDG output for the four runs.

Based on the results from these four runs, it appears that the MEPDG changed the original flexible pavement structure into an equivalent structure consisting of a PCC slab with the same properties as the PCC overlay topped with a base layer with the same properties as the existing HMA overlay; all of this was then supported by Winkler springs having a stiffness equal to composite *k*-value established using all layers under the existing HMA. Figure 87 shows schematically how the MEPDG changed the original structure into an equivalent structure when calculating the pavement response.

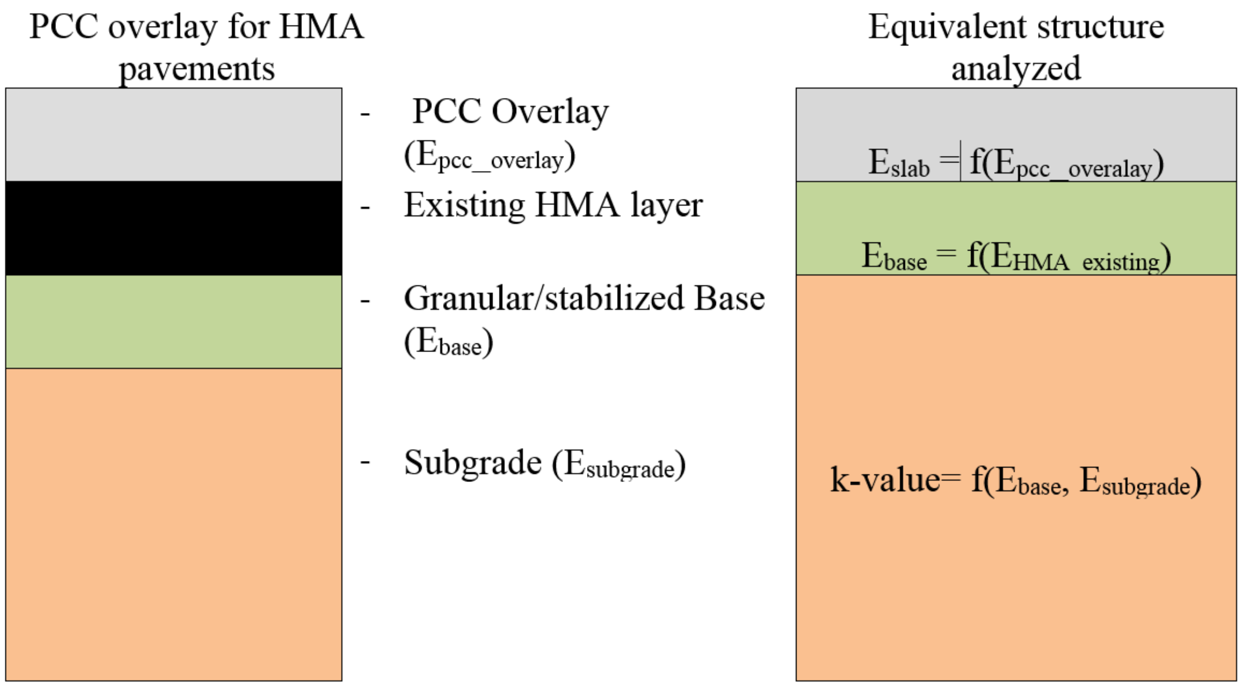


Figure 87. Diagram. Schematic equivalent pavement structure used within the MEPDG to calculate the pavement response for whitetopping.

CHAPTER 6. CASE STUDY 6: COMPOSITE PAVEMENT EVALUATION AND OVERLAY DESIGN

PROJECT OVERVIEW

Project 19-0600 from the LTPP Program SPS-6 database served as the basis for the composite pavement case study. The 19-0600 project was located on I-35 near Des Moines, IA, and test section 19-0659 was selected for the indepth analysis. Test section 19-0659 was a composite pavement cross section with an HMA surface (overlay) on a JRCP and granular base layer over subgrade. This section is representative of the following selection factors:

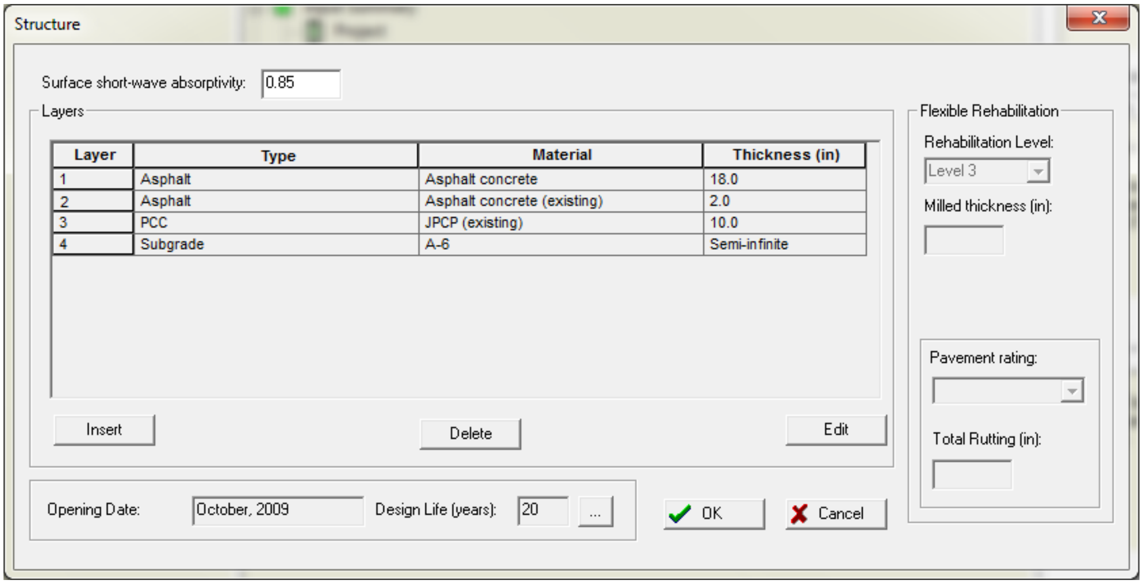
- Composite pavement.
- Wet-freeze climate zone.
- Principal arterial—interstate (rural) functional class.
- “Fair” pavement condition.
- Clay subgrade classification.
- No (or deep) rigid layer.

The original JRCP pavement, nominally a 10-inch (254-mm) PCC slab on a 8-inch (203.2-mm) base with 76.5-ft (23.3-m) transverse joint spacing, was constructed about 1965. A 4-inch (101.3-mm) HMA overlay was placed in 1989. In addition, the LTPP Program database indicates that a number of different maintenance activities were conducted over the years, including asphalt crack sealing in 1990, full-depth transverse joint repair patching in 1992, full-depth patching of HMA pavement in 1999, and skin patching in 2006. Although the existing PCC pavement is a jointed reinforced design, the MEPDG software did not, at the time of this report, differentiate between plain and reinforced pavement in the rehabilitation process.

PAVEMENT CONDITION/PERFORMANCE

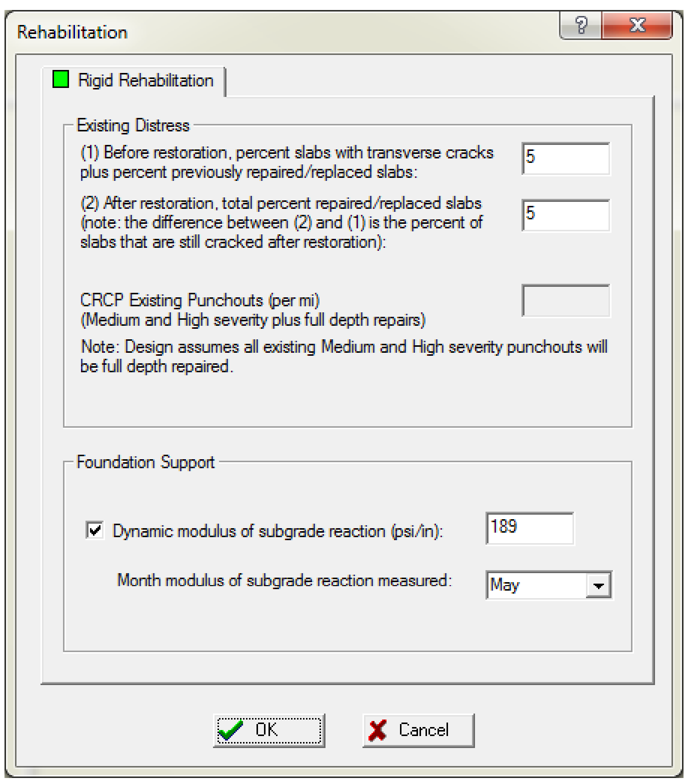
Pavement condition and performance data can be used to customize (or calibrate) the performance models within the MEPDG software for the specific highway agency using the design program. However, the calibration of performance models was beyond the scope of this study. Therefore, the difference in predicted distress and rehabilitation results based on the globally calibrated models were compared for backcalculation-based and laboratory-based inputs.

The inputs used in the pavement rehabilitation design used relatively little distress data. The distress data used for design inputs of HMA overlay rehabilitation of JPCP (including composite pavements) consisted of the percentage of cracked slabs before rehabilitation and the percentage of slabs repaired. The MEPDG design software does not directly analyze JRCP; however, it was assumed that the reinforcement held cracks tight so that they would not reflect through the overlay. Documenting the distress data for the underlying slabs in a composite pavement is problematic for design, even if the existing HMA overlay is intended to be milled off during rehabilitation. In addition, for a composite pavement, reflective cracking is a critical distress that must be considered. The MEPDG reflective cracking model accounts for two modes of crack formation: propagation of existing cracks and propagation of new cracks. While the new cracking was estimated within the program, the documentation was not clear on how the number of existing cracks was determined. There was no distress input for the structure as in many of the other design types (figure 88), but the "Rehabilitation" input screen (figure 89) allows entering the percent of cracked slabs before and after restoration activities. However, based on the results of the design runs, there appears these inputs had little influence.



1 inch = 25.4 mm.

Figure 88. Screen Capture. Structure input screen for HMA over JPCP rehabilitation.



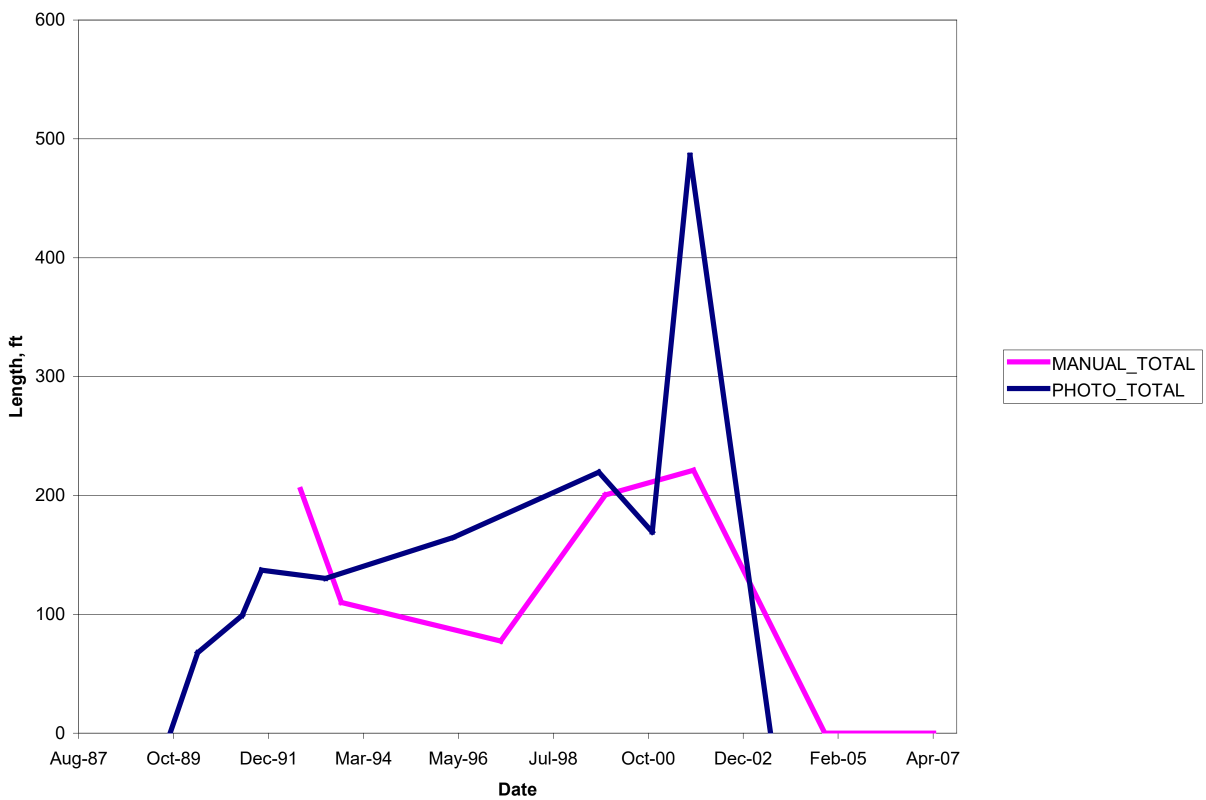
1 psi/inch = 0.263 kPa/mm.

Figure 89. Screen Capture. Existing distress input in "Rehabilitation" screen.

The MEPDG also recommended that the backcalculated PCC modulus be adjusted in relation to the general slab conditions. Again, assessing the general slab conditions was problematic for a composite pavement. Therefore, judgment was required in relating HMA surface distresses and the condition of the pavement before overlay, if available, to the general condition of the underlying PCC during the design process.

Visual distress data were not available in the LTPP Program database for the original JRCP; however, both manual survey data and photographic survey data were available for the HMA surface for several years between 1989 and 2007. As mentioned previously, one of the primary distresses of concern in composite pavements was reflective cracking, but this distress was not specifically identified in the LTPP Program database. Reflective cracking was included in the transverse cracking distress quantity, which also may include thermal cracking of the HMA overlay.

Figure 90 shows the development of transverse cracking since the placement of the HMA overlay in 1989 (based on either manual or photographic surveys). The research team assumed that the absence of cracking in 2005 and 2007 was due to repairs conducted in the interim years. The manual survey recorded 30 cracks in 2001, while the photographic survey recorded 115 cracks; based on the general progression, 30 cracks was more probable. With the section approximately 500 ft (152.5 m) long and a joint spacing of approximately 76.5 ft (23.3 m), there were approximately 6.5 slabs. There would be approximately 4 to 5 cracks (including reflective cracks over the joints) per slab assuming 30 cracks for the section.



1 ft = 0.305 m.

Figure 90. Graph. Summary of transverse cracking distress data for test section 19-0659.

The distresses that were identified in the 2007 data for the HMA surface are summarized in table 88. These distresses included alligator and block cracking, patching, and one location of water bleeding.

Table 88. Summary of test section 19-0659 distress data for 2007.

Distress	Severity	Number of Occurrences	Quantity
Alligator cracking	Medium	—	2,173 ft^2
Alligator cracking	High	—	8 ft^2
Block cracking	Medium	—	2,294 ft^2
Patching	High	12	1,598 ft^2
Water bleeding/pumping	—	1	3 ft

1 ft^2 = 0.093 m^2.
1 ft = 0.305 m.
M = Medium.
H = High.
— Indicates not applicable.

TEST SECTION DATA

The MEPDG design program requires a significant number of inputs. The required design data for test section 19-0659 were obtained from the LTPP Program DataPave database. The required data were not generally complete for any one specific test section within project 19-0600; therefore, results from the entire project were used to obtain the necessary design inputs. While the test sections had slightly different cross sections and maintenance histories, the research team concluded that the data were sufficient for the purposes of this study.

Deflection-Testing Data

Deflection data for test section 19-0659 were available from the LTPP Program database for several years of testing between 1989 and 2007. In addition, laboratory testing data for the PCC layer were available from 2007. Because the 2007 deflection data were the most recent, and there was corresponding PCC testing for that year, the deflection data from 2007 were used in this analysis.

Equipment

Deflection testing was conducted with a Dynatest® FWD (SN 8002-129 for 2007 dataset).

Sensor Configuration

Sensors were located at 0, 8, 12, 18, 24, 36, 48, 60, and -12 inches (0, 203.2, 304.8, 457.2, 609.6, 914.4, 1,219.2, 1,524, and -304.8 mm) for data collected during 2007.

Number of Drops and Load Levels

Four load level targets—6,000, 9,000, 12,000, and 16,000 lb (2,724, 4,086, 5,448, and 7,264 kg)—with four drops at each load level were performed, and the resultant data recorded.

Seating drops were also performed at each load level, but the deflection data were not recorded. Time histories were also available for the fourth drop height.

Test Locations/Lanes and Increments

Testing was conducted mid-lane and in the outer wheelpath at approximately 40- to 80-ft (12.2- to 24.4-m) intervals.

Temperature Measurements

During the deflection testing, temperature measurements were taken at predetermined time intervals using drilled holes in the composite pavement at depths of 0.9, 2.2, 3.8, 8.2, and 12.0 inches (22.9, 55.9, 96.5, 208.3, and 304.8 mm) below the pavement surface.

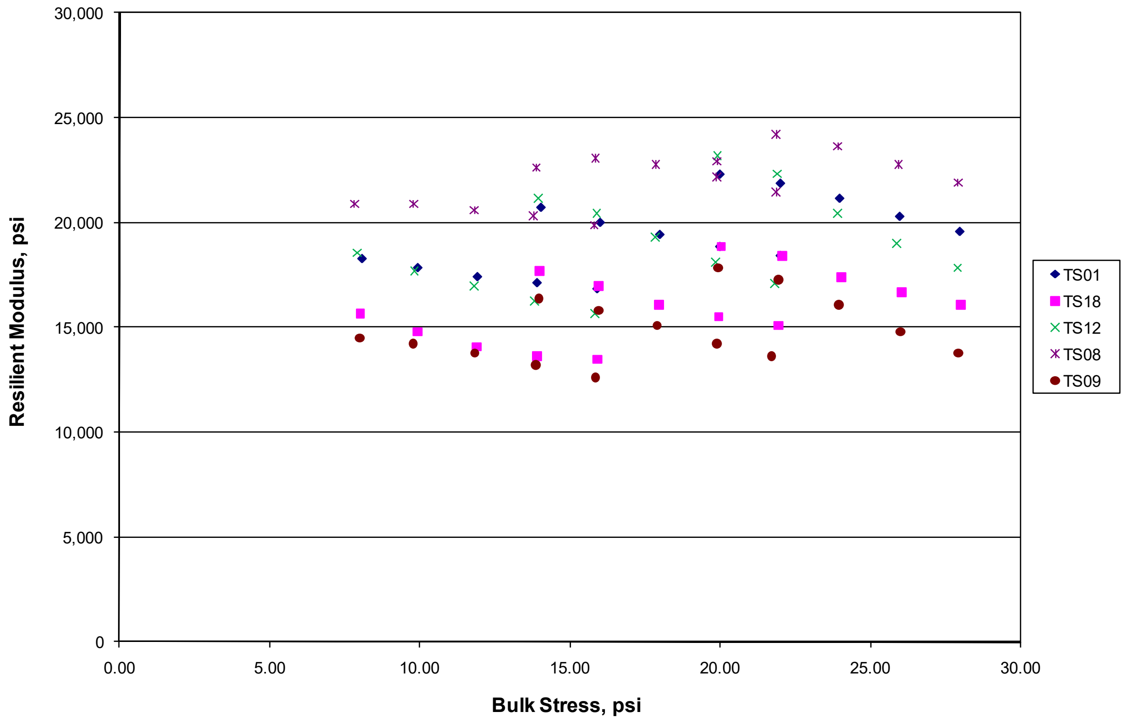
Material Properties Data

This section summarizes the data obtained from the LTPP database for test section 19-0659 regarding its subgrade, granular base, HMA and PCC material properties, traffic, climate, and depth of water table or stiff layer.

Subgrade

Six subgrade samples were retrieved from the 19-0600 project location, including two samples from test section 19-0659, as part of the LTPP Program. The subgrade was generally classified as A-4 to A-6 materials under the AASHTO soil classification system.[2]

Laboratory resilient modulus testing results were available for the six samples of the subgrade materials obtained as part of the LTPP Program data collection; however, one sample's results were significantly different than the remaining samples and were not included in this analysis. Laboratory resilient modulus testing results for the subgrade samples (identified as TS##) evaluated are illustrated in figure 91.



1 psi = 6.89 kPa.

Figure 91. Graph. Summary of LTPP Program laboratory measured resilient modulus for collected subgrade samples.

Additional subgrade properties, including Atterberg limits and sieve analysis, are summarized in table 89 and table 90, respectively.

Table 89. Summary of subgrade Atterberg limits.

Laboratory Test	Average Test Result (Percent)
Liquid limit	29.3
Plastic limit	15.7
Plasticity index	13.7

Table 90. Summary of subgrade sieve analysis.

Sieve Size	Average Percent Passing
1.5 inch	100
1 inch	100
$^{3}/_{4}$ inch	99
$^{1}/_{2}$ inch	99
$^{3}/_{8}$ inch	98
No. 4	96
No. 10	93
No. 40	78
No. 80	61
No. 200	53.4

1 inch = 25.4 mm.

Granular Base

Laboratory resilient modulus testing for the granular base material for project 19-0600 was not available. The base was classified as an uncrushed gravel, and the Atterberg limits generally indicate the material was nonplastic. Aggregate sieve analysis of the base aggregate is summarized in table 91.

PCC Pavement

PCC data available from the LTPP Program database included unit weight, compressive strength, splitting tensile strength, and elastic modulus testing from cores tested in 2007. In addition, the CTE testing was conducted on retrieved cores. The available PCC properties are summarized in table 92 through table 94.

Table 91. Summary of base aggregate sieve analysis.

Sieve Size	Average Percent Passing
1.5 inch	100
1 inch	99
$^{3}/_{4}$ inch	95
$^{1}/_{2}$ inch	83
$^{3}/_{8}$ inch	75
No. 4	60
No. 10	47
No. 40	22
No. 80	13
No. 200	9.9

1 inch = 25.4 mm.

Table 92. Summary of PCC laboratory testing.

Laboratory Test	Average Test Result
Compressive strength (psi)	5,727
Splitting tensile strength (psi)	612
Elastic modulus,(psi)	3,367,000
CTE ($\times 10^{-6}$/°F)	5.8
Poisson's ratio	0.15
Density (lb/ft^3)	136

1 psi = 6.89 kPa.
°F = 1.8 × °C + 32.
1 lb/ft^3 = 0.0160 g/cm^3.

Table 93. Summary of PCC mix design information.

Variable	Value
Cement type	Type I
Cementitious material content (lb/yd^3)	604
Water-to-cement ratio	0.40
Curing method	Curing compound

1 lb/yd^3 = 0.593 kg/m^3.

Table 94. Summary of PCC pavement design features.

Variable	Value
Joint spacing (ft)	76.5
Sealant type	Liquid
Dowel diameter (inches)	1.25
Dowel spacing (inches)	12
Edge support	None

1 ft = 0.305 m.
1 inch = 25.4 mm.

Although the joint spacing was 76.5 ft (23.3 m), the MEPDG design program did not allow joint spacing greater than 25 ft (7.6 m). Even when setting a 25-ft (7.6-m) joint spacing, a program warning was encountered indicating that 20 ft (6.1 m) was the acceptable maximum spacing. (Note that the newer version 1.1 of the design software now allows up to a 30-ft (9.1 m) joint spacing.)

HMA Surface

Information for the existing HMA overlay included aggregate gradation and general mix properties. Asphalt cement grading was obtained from adjacent LTPP General Pavement Studies sections; an AC-20 was used for the last surface layer constructions. The available existing HMA overlay properties are summarized in table 95 and table 96. Resilient modulus testing for the existing HMA overlay was also conducted and is summarized in figure 92.

Table 95. Summary of existing HMA material properties.

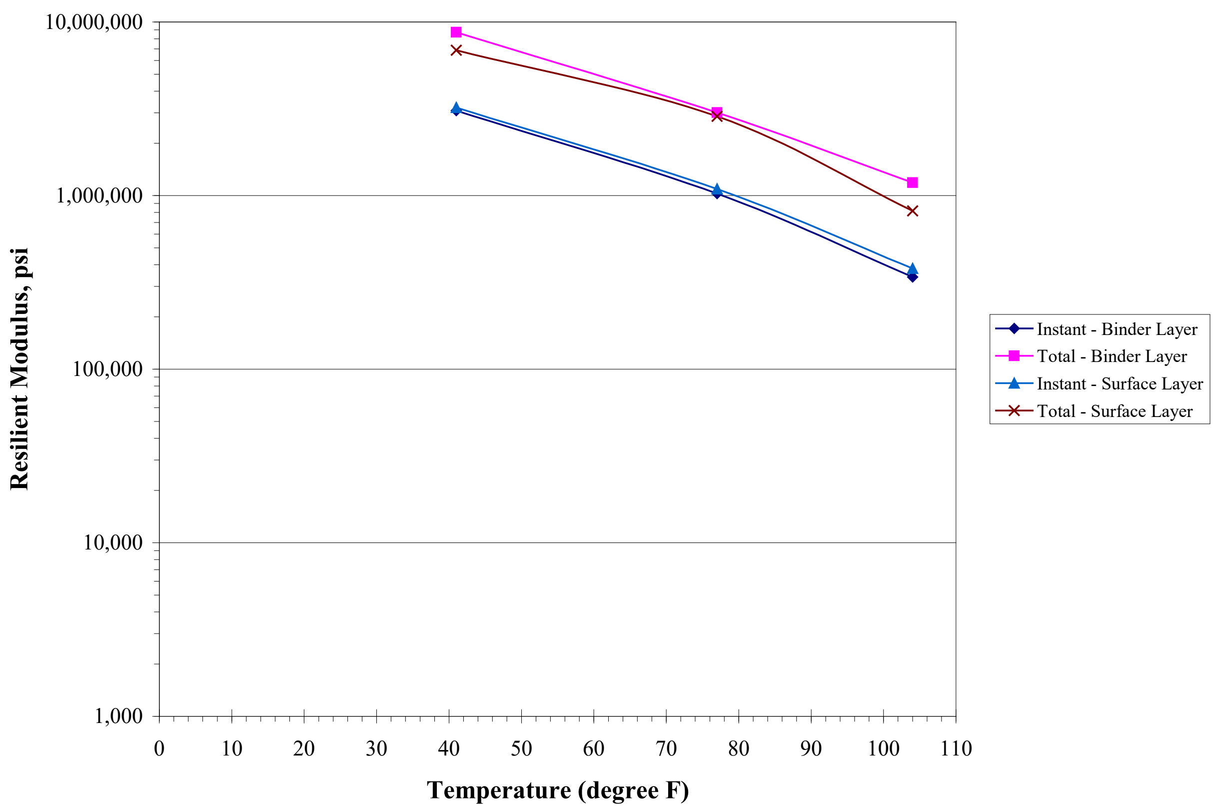
Variable	Value
Effective binder content (percent)	7.1
Air voids (percent)	5.0
Total unit weight (lb/ft^3)	148.0

1 lb/ft^3 = 0.0160 g/cm^3.

Table 96. Summary of existing HMA aggregate sieve analysis.

Sieve Size	Average Percent Passing
$^3/_4$ inch	100
$^3/_8$ inch	69
No. 4	45
No. 200	6.4

1 inch = 25.4 mm.



1 psi = 6.89 kPa.
°F = 1.8 × °C + 32.

Figure 92. Graph. Summary of HMA resilient modulus testing.

Based on the laboratory data, the existing HMA binder and surface layers had relatively similar resilient moduli, with an average instantaneous resilient modulus of approximately 1,460,000 psi (10,059,400 kPa) at 70 °F (21.1 °C) and an average total resilient modulus of approximately 3,880,000 psi (26,733,200 kPa) at 70 °F (21.1 °C), both of which were quite high for HMA.

Depth to Rigid Layer/Water Table

The depth to the water table is another input required for the MEPDG. However, no data were found for project 19-0600. One record was obtained from the LTPP Program database for Iowa, indicating the depth to a rigid layer was greater than 100 ft (30.5 m). Records also suggested a perched water table was possible between April and July at a depth of 12 to 48 inches (304.8 to 1219.2 mm). Historical groundwater levels from the USGS indicated an average of approximately 12 ft (3.7 m) below land surface.[4]

Climate/Environment Data

Climate data were obtained from the updated climate files on the MEPDG Web site.[5] The weather station at Des Moines, IA, was used for this study. The general climatic category for the case study location is wet-freeze.

Traffic Data

Traffic data were obtained from the LTPP Program database. Because evaluating the effect of traffic data on design results was not a primary goal of this study, only basic information was used from the available data (total volume, growth, and vehicle class distribution), with the remaining inputs (such as monthly distribution, hourly distribution, and wheel spacing) kept at their default values in the MEPDG software. A 2.15-percent linear growth was estimated based on traffic data from 1965 to 1993. An initial two-way AADTT volume of 6,179 vehicles per day for 2009 was estimated based on the linear estimation. The vehicle class distribution obtained from the LTPP Program database is summarized in table 97.

Table 97. Traffic distribution by vehicle class.

Vehicle Class	Percentage
4	2.8
5	7.4
6	3.8
7	0.6
8	7.1
9	69.3
10	1.1
11	6.3
12	1.4
13	0.2

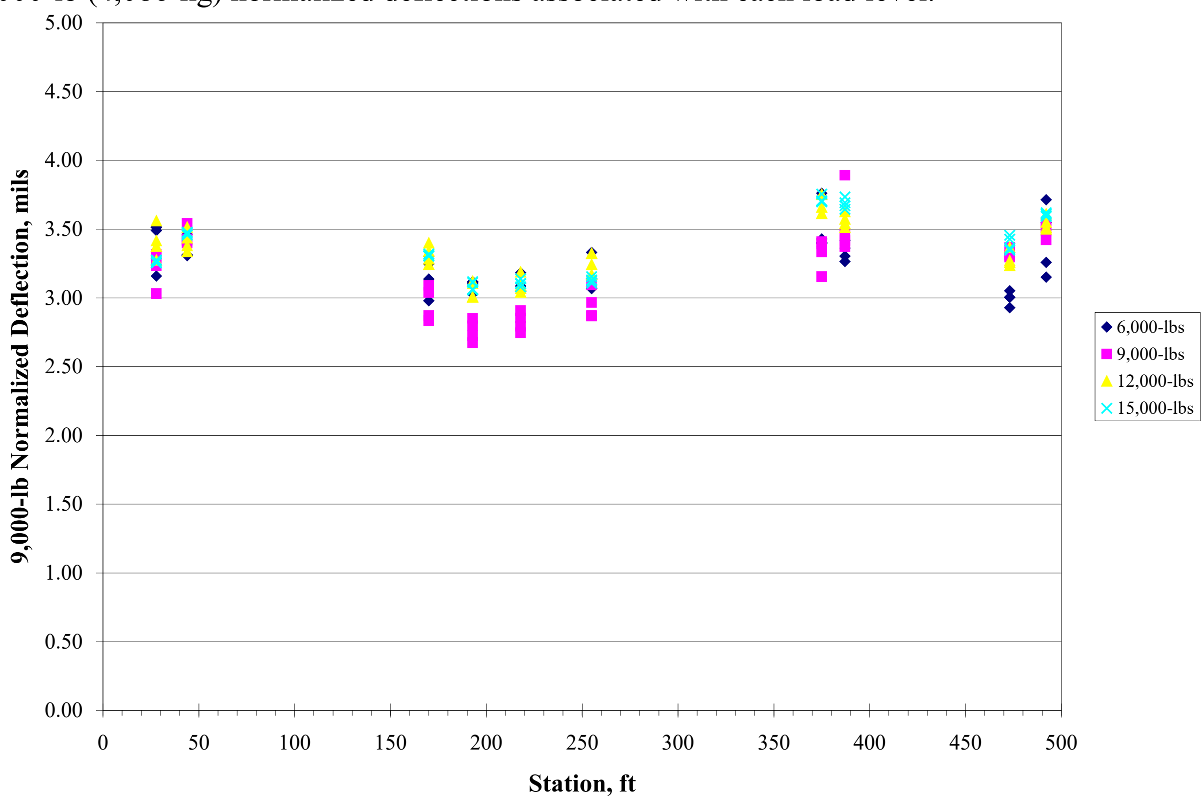
ANALYSIS AND INTERPRETATION OF FWD TESTING DATA

This section presents the data checks and backcalculation analysis of the FWD data, as well as a comparison of the backcalculation results with laboratory testing.

Preprocessing Deflection Data

Before conducting the backcalculation analysis, prescreening of the data was accomplished in a spreadsheet format to identify any outlying or questionable data. An examination of the 2007 data revealed a number of locations with non-decreasing deflections. Although these points were potentially recorded, they were generally not included in a backcalculation analysis. However, for test section 19-0659, 6 of the 10 test locations had non-decreasing basins for one or more of the test drops, and it was generally recommended that 25 percent or fewer of the data points should be removed before questioning the entire dataset. For this case study, only the non-decreasing deflection basin locations were used in estimating the *k*-value and PCC modulus even though this resulted in removal of more than 25 percent of the locations.

Next, a review of the normalized deflections of the entire dataset to a load level of 9,000 lb (4,086 kg) (impulse stiffness modulus can also be used) was performed to evaluate whether the load response was relatively uniform along the tested section or whether the pavement should be divided into smaller sections. The normalized deflection plot for the 2007 dataset is illustrated in figure 93. From this plot, the deflections appear relatively uniform, with only a slight difference between the normalized deflections for the different load levels. Table 98 summarizes the 9,000-lb (4,086-kg) normalized deflections associated with each load level.



1 mil = 0.0254 mm.
1 lb = 0.454 kg.
1 ft = 0.305 m.

Figure 93. Graph. Normalized deflection plot for 2007 mid-lane deflection basins.

Table 98. Summary of normalized deflections.

Statistic	Target Load Level 6,000 (lb)	9,000 (lb)	12,000 (lb)	15,000 (lb)
Average (mil)	3.26	3.18	3.36	3.38
COV	0.06	0.09	0.06	0.07

1 lb = 0.454 kg.
1 mil = 0.0254 mm.

Backcalculation Analysis

Backcalculation of the 19-0659 deflection data was performed using the outer-AREA method with the SHRP five-sensor equations to determine the *k*-value and layer moduli. The outer-AREA method was used to account for compression in the HMA overlay of the composite pavement structure. The analysis consisted of a three-layer system: HMA overlay, PCC slab, and spring foundation.

Backcalculation Results

The average results from the backcalculation are summarized in table 99. The variability in the *k*-value was relatively low, and while the variability in the bound layers was higher, it was still acceptable. In this case, the foundation represented the granular base and subgrade. The determined *k*-value using the outer-AREA method would also include the effects of a rigid layer, if present.

Table 99. Summary of average backcalculation results.

Statistic	Dynamic *k*-value (psi/inch)	Dynamic PCC Modulus (psi)*	HMA Overlay Modulus (psi)*
Average	189	8,525,000	852,500
COV	0.10	0.23	0.23

*At temperature during testing and assuming preliminary modular ratio (β) of 10 (see figure 122 in volume I).
1 psi = 6.89 kPa.
1 psi/inch = 0.263 kPa/mm.

The outer-AREA method was first used to calculate an effective modulus of a single plate with an equivalent thickness equaling the combined thickness of the two layers. The HMA overlay and PCC slab moduli were then determined following the relationship developed by Ioannides and Khazanovich for a two-plate system (the layers are assumed to be bonded).[15] The determination of the individual layer moduli requires the assumption of a modular ratio (β) of the PCC and HMA moduli. A typical modular ratio for a dense-graded HMA is 10 and was used in the initial backcalculation results.[16] However, a range of modular ratios are summarized in with corresponding HMA and PCC moduli. As shown in Table 100, the HMA modulus was more sensitive to the modular ratio selection, increasing by 100 percent from a ratio of 10 to a ratio of 4, while the PCC modulus decreased only by 25 percent over that range.

Table 100. Summary of influence of modular ratio selection.

Modular Ratio	Backcalculated HMA Modulus (psi)	Backcalculated PCC Modulus (psi)
10	852,500	8,525,000
9	928,600	8,357,000
8	1,020,000	8,158,000
7	1,131,000	7,919,000
6	1,271,000	7,625,000
5	1,451,000	7,255,000
4	1,694,000	6,777,000

1 psi = 6.89 kPa.

Backcalculation Modeling Issues and Recommendations

The existing pavement structure was considered a four-layer system (five-layer system if a rigid layer were present). The aggregate base layer was included in the determined *k*-value, as was the presence of any rigid layer. In addition, an effective modulus for an equivalent single plate was first determined, and the individual bound layer moduli were then determined using a modular ratio. The closed-form solutions for composite pavements at the time of this report were not well suited for more than two bound layers and multiple unbound layers.

Comparison of Backcalculation and Laboratory Testing Results

To assist in evaluating which layer characteristics were appropriate to use in the MEPDG software, the results of the backcalculation (field tests) were compared with results obtained from laboratory testing conducted as part of the LTPP Program. While it was outside of the scope of this study to develop new correlations or conversion factors between the two, it was still beneficial to evaluate these relationships and how they may influence MEPDG input selection.

Unbound Materials

The first general argument when comparing field with laboratory results of unbound materials is the stress state of the material is different for the two conditions. To compare laboratory results with those obtained from FWD testing (or vice versa), the resilient modulus was estimated for the stress conditions at the time of FWD testing. The stress conditions accounted for the overburden pressure of the pavement and the stress due to loading. Laboratory testing was only available for the subgrade material for 19-0600, so only the subgrade comparison was discussed in this case study.

Load stresses were determined using ILLISLAB, the load applied during deflection testing, and initial backcalculated layer properties. Overburden stresses were estimated using layer densities, moisture contents, thicknesses, and at-rest earth pressure coefficient (K_0). K_0 for fine-grained unbound layers was estimated using the equation in figure 94.

$$K_0 = \frac{\nu}{1-\nu}$$

Figure 94. Equation. Estimation for at-rest earth pressure coefficients for fine-grained unbound layers.

Where:

K_0 = At-rest earth pressure coefficient.
ν = Poisson's ratio.

To compare the resilient modulus at the stress conditions of FWD testing, the constitutive model in figure 95 (contained in the MEPDG) was used.(6)

$$M_R = k_1 \times p_a \times \left(\frac{\theta}{p_a}\right)^{k_2} \times \left(\frac{\tau_{Oct}}{p_a} + 1\right)^{k_3}$$

Figure 95. Equation. Constitutive model for determining resilient modulus.

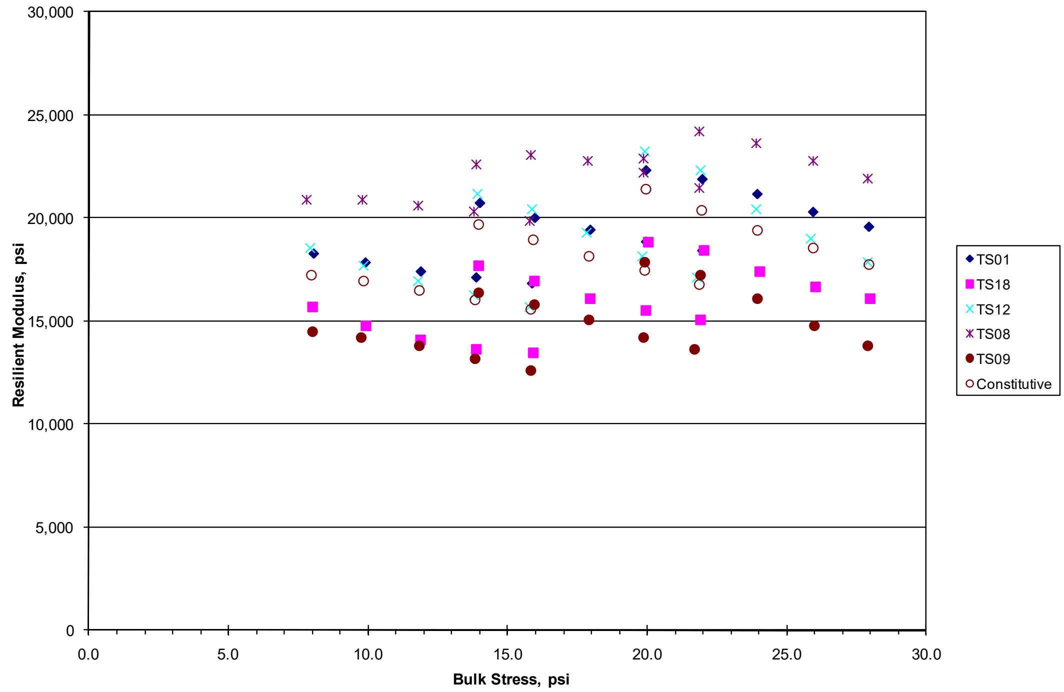
Where:

M_R = Resilient modulus, psi (kPa).
k_1, k_2, k_3 = Regression constants.
p_a = Atmospheric pressure, psi (kPa).
θ = Bulk stress, psi (kPa).
τ_{Oct} = Octahedral shear stress, psi (kPa).

The constitutive model coefficients determined for the subgrade materials from available laboratory results for the 19-0600 project are summarized in table 101; the model is illustrated in figure 96, where TS** designates the respective samples. Figure 97 illustrates the fit of the constitutive model with the measured properties.

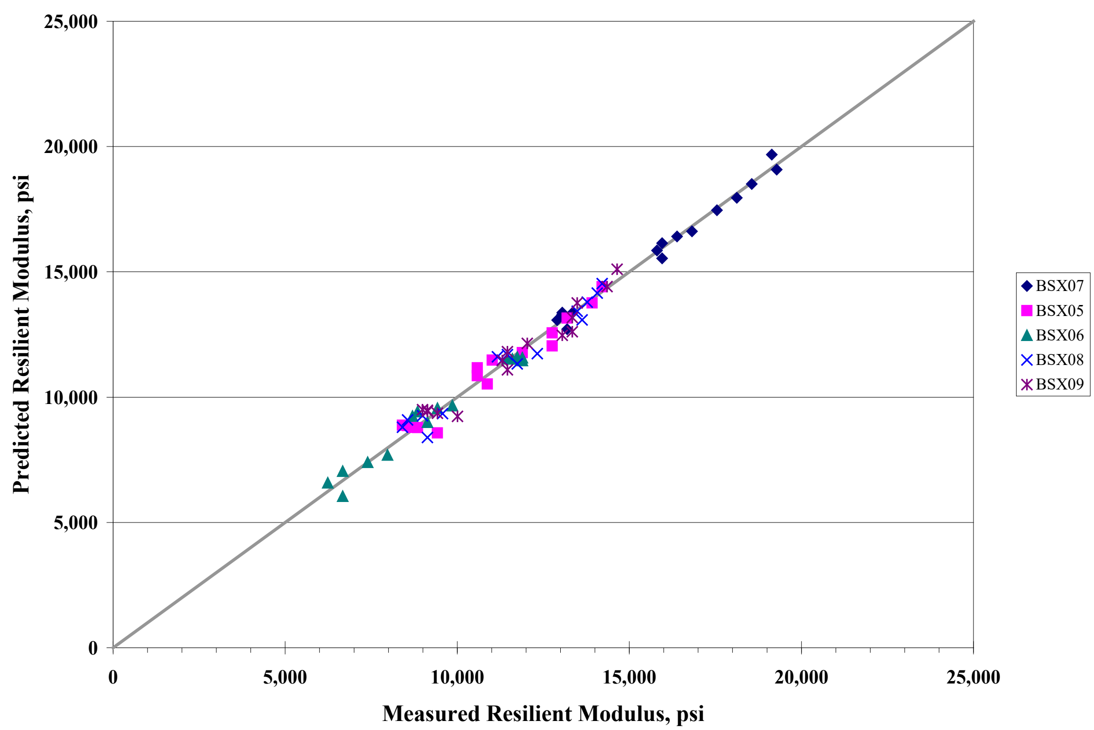
Table 101. Summary of estimated subgrade constitutive regression constants.

Regression Constant			R^2
k_1	k_2	k_3	
1,475.3	0.236	-1.22	0.985



1 psi = 6.89 kPa.

Figure 96. Graph. Estimated constitutive model for subgrade resilient modulus.



1 psi = 6.89 kPa.

Figure 97. Graph. Comparison of subgrade laboratory resilient modulus with predicted resilient modulus.

Although moisture condition influenced the resulting resilient modulus, laboratory resilient modulus testing was conducted at the in situ conditions, and only a general comparison of values is intended here, so the specific effects of moisture were not considered in this case study.

The backcalculation procedures for rigid pavements provided a k-value, whereas the laboratory testing of material samples resulted in the determination of a resilient modulus value. One available correlation for k-value and resilient modulus is shown in figure 98.[(2)]

$$M_R = 26(k\text{-}value)^{1.284}$$

Figure 98. Equation. Correlation for *k*-value and resilient modulus.

Where:

M_R = Resilient modulus, psi.
k-value = Modulus of subgrade support, psi /inch.

However, the k-value determined using the outer-AREA backcalculation was a dynamic k-value and included the contribution of the unbound base layer as well as the subgrade (and bedrock, if present). A factor of 0.5 is commonly applied to convert a dynamic k-value to a static k-value. So, for this section the dynamic k-value corresponded to a static composite k-value of approximately 95 psi /inch (26 kPa/mm) and a resilient modulus of 9,000 psi (62,010 kPa). These values were considered somewhat low for a composite material that included the effects of both the subgrade and the aggregate base materials.

Determination of individual layer properties from the composite k-value was a more difficult task. One possible solution was to use the PCA's forward design procedure for determining the influence of an aggregate base layer on a subgrade k-value for use in slab thickness design.[(13)] In this case, the improved k-value was used to back out a subgrade k-value for the thickness of the aggregate layer. Using this method, the research team found that the subgrade static k-value was approximately 62 psi/inch (17 kPa/mm), which corresponds to a subgrade resilient modulus of approximately 5,300 psi (36,517 kPa). A modulus value for the base layer would then need to be assumed (values of 25,000 to 35,000 psi (172,250 to 241,150 kPa)) were used in the design analyses, as discussed later). Another possible solution could be obtained using layered elastic analysis to match the measured deflections, similar to the conversion for new rigid pavement design within the MEPDG software. This method would also require some assumptions for the base layer modulus to determine the subgrade modulus. This latter approach was not investigated for this case study, however.

The estimated subgrade resilient modulus based on laboratory testing and using the constitutive model at the stress conditions for a 9,000-lb (4,086-kg) FWD load was 14,900 psi (102,661 kPa). This was much higher than the estimated subgrade modulus and the composite modulus based on the backcalculated k-value. However, it was quite close to the default design input for the material type (14,000 psi (96,460 kPa)).

Using the k-value-to-resilient modulus correlation in figure 95, a dynamic-to-static (composite value) k-value correction of 0.74 (instead of the 0.5 correction) would provide more consistent values in this case. However, the correlation equation for *k*-value to resilient modulus may as likely be the source of the observed difference.

To assess the significance of determining appropriate correction factors, several combinations of *k*-value and layer modulus were evaluated, as discussed later in this case study.

Bound Materials

This section presents results of the comparison of the backcalculation results from both PCC and HMA overlay layers with laboratory test results.

PCC Layer

For the PCC surface, the splitting tensile strength, compressive strength, and elastic modulus testing results were available for cores retrieved for section 19-0600. The testing values were converted to elastic modulus and flexural strength estimates using the respective correlation equations summarized in figure 99, with the calculations summarized in table 102.[6]

$$E_{PCC} = 33\rho^{3/2}(f'_c)^{1/2}$$
$$MR = 9.5 * (f'_c)^{1/2}$$
$$MR = 1.02 * (f'_t) + 117$$
$$MR = 43.5\left(\frac{E_{pcc}}{10^6}\right) + 488.5$$

Figure 99. Equation. Various strength and elastic modulus correlation equations.

Where:

E_{pcc} = Elastic modulus, psi (kPa).
MR = Flexural strength, psi (kPa).
ρ = Density of PCC (2.18 g/cm^3 (136 lb/ft^3) from LTPP Program data).
f'_c = Compressive strength, psi (kPa).
f_t = Splitting tensile strength, psi (kPa).

Table 102. Summary of PCC laboratory testing correlations.

Laboratory Test	Average Test Result (psi)	Estimated Flexural Strength (psi)	Estimated Elastic Modulus (psi)
Compressive strength	5,727	719	3,961,000
Splitting tensile strength	612	742	5,828,000
Elastic modulus	3,367,000	635	N/A

N/A = Not applicable.
1 psi = 6.89 kPa.

While the estimated elastic modulus from the compressive strength testing was fairly similar to the measured elastic modulus, the estimated elastic modulus from the splitting tensile strength was quite a bit higher.

The elastic modulus obtained from backcalculation is a dynamic value, one that is based on a rapid, relatively low stress level. As such, it cannot be compared directly with the elastic modulus obtained through laboratory testing. Typically, an adjustment factor of 0.8 has been historically recommended for converting a backcalculated PCC elastic modulus value to a static elastic modulus value.(6) The static PCC modulus values are summarized in table 103 for varying modular ratios, showing that the moduli were still greater than those based on laboratory testing. Adjustment factors ranged from approximately 0.39 to 0.86 depending on modular ratio and laboratory testing result.

Table 103. Summary of influence of modular ratio selection.

Modular Ratio	Backcalculated Dynamic PCC Modulus (psi)*	Approximate Equivalent Static PCC Modulus (psi)
10	8,525,000	6,820,000
9	8,357,000	6,686,000
8	8,158,000	6,527,000
7	7,919,000	6,335,000
6	7,625,000	6,100,000
5	7,255,000	5,804,000
4	6,777,000	5,422,000

*Values taken from table 100.
1 psi = 6.89 kPa.

HMA Overlay Layer

The estimated HMA modulus at 70 °F (21.1 °C) based on laboratory testing was approximately 3,880,000 psi (26,733,200 kPa) (total resilient modulus), which was extremely high for an HMA material. The estimate of the laboratory-based HMA modulus for a temperature of 87 °F (30.6 °C) (average temperature at the time of FWD testing) was 2.2 million psi (15,158,000 kPa), which was significantly greater than the backcalculated value (852,500 psi (5,873,725 kPa) using a modular ratio of 10). Alternatively, the backcalculated HMA modulus can be adjusted for temperature based on the equation in figure 100.(17)

$$ATAF = 10^{slope*(T_r - T_m)}$$

Figure 100. Equation. Asphalt temperature adjustment factor.

Where:

ATAF = Asphalt temperature adjustment factor.
slope = Slope of the log modulus versus temperature equation (-0.021 for mid-lane).
T_r = Reference HMA mid-depth temperature °F (°C).
T_m = HMA mid-depth temperature at time of testing °F (°C).

A correction factor of approximately 1.66 was estimated using the last equation in figure 99 for a reference temperature of 70 °F (21.1 °C). The temperature-corrected HMA moduli for a range of modular ratios are summarized in Table 104. Summary of temperature corrected HMA modulus.

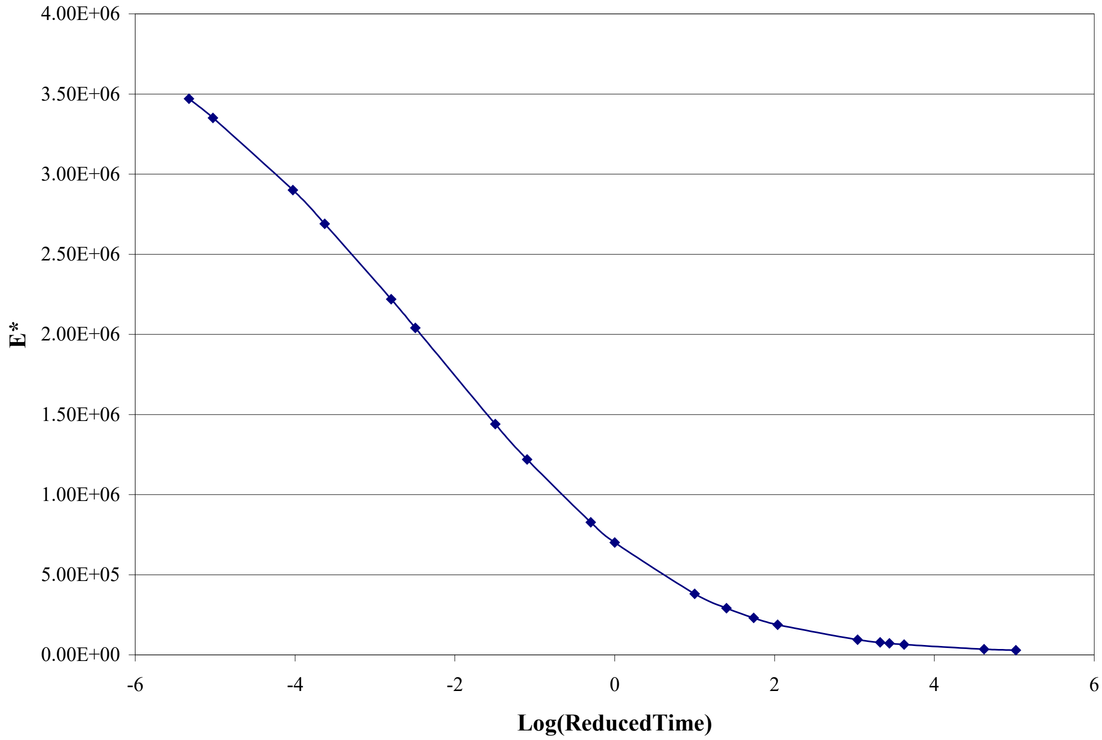
Table 104. Summary of temperature corrected HMA modulus.

Modular Ratio	Backcalculated HMA Modulus at 87 °F* (psi)	Temperature Corrected HMA Modulus at 70 °F (psi)
10	852,500	1,415,000
9	928,600	1,541,000
8	1,020,000	1,693,000
7	1,131,000	1,878,000
6	1,271,000	2,109,000
5	1,451,000	2,409,000
4	1,694,000	2,812,000

*Taken from table 100.
1 psi = 6.89 kPa.
°F = 1.8 × °C + 32.

An additional consideration in comparing laboratory and deflection testing is the frequency of loading. Laboratory resilient modulus testing is conducted with a 0.1-s load pulse, whereas FWD testing is conducted with a 0.03-s load pulse (approximately). Although dynamic modulus testing data were not available, reviewing the approach for shifting the HMA modulus due to frequency and temperature was worthwhile. Based on the available HMA properties, recommended MEPDG default values, and predictive equations, the estimated dynamic modulus master curve using the MEPDG software is illustrated in figure 101.

Based on the results of the predictive equations incorporated in the MEPDG and case study inputs, a dynamic modulus of 1,136,000 psi (7,827,040 kPa) was estimated for the conditions during FWD testing (87 °F (30.6 °C) and 0.03-s loading time). A dynamic modulus of 10,955,100 kPa (1,590,000 psi) was estimated at 70 °F (21.1 °C) and 0.1-s loading time (resilient modulus testing conditions). The predictive model results appear to be more reasonable than the resilient modulus testing for this section but still suggest a relatively stiff HMA material.



1 psi = 6.89 kPa.

Figure 101. Graph. Illustration of HMA dynamic modulus master curve from MEPDG design software.

Because the laboratory resilient modulus testing and the predictive equations indicated a stiffer HMA, a modular ratio less than 10 appeared appropriate for use in the backcalculation analysis; a modular ratio of 7 was selected for this case study.

REHABILITATION DESIGN RESULTS

For this case study, a HMA overlay rehabilitation design was selected. The design of an HMA overlay of a composite pavement was performed in a similar manner as an HMA overlay of a PCC pavement, except that the existing HMA surface layer was included in the design analysis. It was assumed that 2 inches (50.8 mm) of the existing HMA overlay would be milled to address surface deterioration, and that additional pre-overlay repairs (patching and crack sealing) would be performed as necessary. The milling depth was not an available input, so the entered thickness accounted for the milling. While for this case study, the remaining HMA thickness was relatively thin (2 inches (50.8 mm)), not entirely removing the existing HMA during rehabilitation is often done.

The overall design level for this rehabilitation (HMA over JPCP) was level 3, which cannot be changed in the MEPDG. However, a different design level can be assigned for the individual structural layers, which would dictate which design inputs are used. For this case study, a level 1 analysis was required for the PCC strength input to use the elastic modulus input, whereas the existing HMA layer inputs current at the time of this report did not allow the use of backcalculation data, regardless of design level (levels 1, 2, and 3 were available). Levels 2 and 3 were available for the unbound layers material inputs. For this case, the subgrade layer was assigned a level 2, and the aggregate base layer, when used in the cross section, was assigned a level 3.

Design Criteria and General Inputs

The general inputs used for the design program were based on the available LTPP Program data (as previously summarized), estimated inputs from standard specifications, and on default values within the program when data were not available from other sources. The primary inputs that were evaluated for this case study were those corresponding to properties obtained from the backcalculation process (PCC elastic modulus and *k*-value).

A 90-percent reliability level was selected for this analysis. The general, site, and analysis parameter inputs are summarized in table 105 and table 106.

Table 105. Summary of general and site project information inputs.

Variable	Value
Existing pavement construction	July 1999 (HMA overlay) 1965 (original PCC)
Overlay construction	August 2009
Traffic open	August 2009
Design life	20 years
Type of design in MEPDG	HMA overlay over JPCP
Location	Des Moines, IA
Project ID	19-0600
Section ID	19-0659
Date (of Analysis Setup)	June 2009
Station/milepost format	ft
Station/milepost begin	0
Station/milepost end	499
Traffic Direction	South

1 ft = 0.305 m.

Table 106. Summary of analysis parameter inputs.

Variable	Value
Initial IRI (inches/mi)	63
Terminal IRI (inches/mi)	172
Transverse cracking (percent slabs)	15
HMA surface down cracking (long cracking) (ft/mi)	2,000
HMA bottom up cracking, alligator cracking, (percent)	25
HMA thermal fatigue, (transverse cracking) (ft/mi)	1,000
Chemically stabilized layer fatigue fracture (percent)	N/A
Permanent deformation—HMA only (inches)	0.25
Permanent deformation—total pavement (inches)	0.75

1 inch/mi = 0.0158 m/km.
1 inch = 25.4 mm.
1ft/mi = 0.19 m/km.
N/A = Not applicable.

For the new HMA overlay material properties, input values selected are summarized in table 107 and table 108.

Table 107. Summary of new HMA material properties.

Variable	Value
Asphalt grading	PG 58-28
Asphalt content (percent)	11.5
Air voids (percent)	6.8
Total unit weight (lb/ft^3)	148

1 lb/ft^3 = 0.0160 g/cm^3.

Table 108. Summary of new HMA aggregate sieve analysis.

Sieve Size	Average Percent Passing
$^3/_4$ inch	100
$^3/_8$ inch	69
No. 4	45
No. 200	6.4

1 inch = 25.4 mm.

Incorporation of Backcalculation Results with the MEPDG Software

The performance life of HMA over PCC is typically governed by material and functional factors, not structural failure of the underlying PCC pavement. The key distresses in the MEPDG program for HMA overlays of PCC pavements included rutting and reflective cracking. The structural evaluation for HMA over PCC pavements was mainly a design check to ensure that the stresses were within the tolerable limits. Therefore, the overall design results were not anticipated to vary significantly because of variation in the pavement layer strength characteristics. Several analyses were run with the MEPDG design software (summarized in table 109) to evaluate the influence of the unbound and bound layer inputs, as discussed in the following sections.

Unbound Layers

The HMA overlay rehabilitation design for a composite pavement required individual layer inputs, including layer modulus and material properties. The dynamic subgrade *k*-value was an optional input and was entered as part of the "Rehabilitation" screen input. Although the resilient modulus was entered for the subgrade layer in the corresponding layer structure screen, the subgrade modulus and material properties were supposedly used only in determining the ICM climatic adjustment factors. The determined climatic adjustment factors were then applied to the input dynamic subgrade *k*-value. If the *k*-value was not entered, the subgrade layer information was used to compute a *k*-value internally.

Table 109. Summary of subgrade properties used in design runs.

Analysis Run Number	Backcalculated Dynamic *k* (psi/inch)	Subgrade Modulus (psi)	Base Modulus (psi)	PCC Modulus Correction Factor	PCC Modulus Condition Factor, C_{BD}	Average PCC Modulus (psi)	Estimated Flexural Strength (psi)
1[a]	189	14,000	—	0.8	0.7	4,434,000	681
2[b]	—	14,900	—	-	—	4,385,000	679
3[c]	189	9,000	—	0.8	0.7	4,434,000	681
4[d]	189	14,000	25,000	0.8	0.7	4,434,000	681
5[e]	125	14,000	25,000	0.8	0.7	4,434,000	681
6[f]	125	5,300	25,000	0.8	0.7	4,434,000	681
7[g]	189	14,000	35,000	0.8	0.7	4,434,000	681
8[h]	189	14,000	—	1.0	1.0	7,919,000	833
9[i]	189	14,000	—	0.8	1.0	6,335,000	764
10[j]	189	14,000	—	0.8	0.6	3,801,000	654
11[j]	189	14,000	—	0.8	0.5	3,167,000	626

[a]Analysis 1: Initial backcalculation-based subgrade and PCC input selections.
[b]Analysis 2: Laboratory-based subgrade and PCC input selections.
[c]Analysis 3: Backcalculated dynamic *k*-value and correlation of subgrade modulus to backcalculated (composite) *k*-value.
[d]Analysis 4: Inclusion of base layer without adjusting backcalculated *k*-value input.
[e]Analysis 5: Inclusion of base layer with adjusting dynamic *k*-value (estimation of separated subgrade *k*-value).
[f]Analysis 6: Inclusion of base layer with adjusting dynamic *k*-value and correlation of subgrade modulus to estimated subgrade *k*-value.
[g]Analysis 7: Inclusion of base layer with higher modulus value than analysis 4.
[h]Analysis 8: Unadjusted (dynamic) backcalculated PCC modulus.
[i]Analysis 9: Static backcalculated PCC modulus without additional condition adjustment.
[j]Analyses 10 and 11: Additional sensitivities of PCC condition factor.
1 psi = 6.89 kPa.
1 psi/inch = 0.263 kPa/mm.
— Indicates not applicable.

The unbound layer inputs can be entered either as level 2 or level 3. The layer modulus can be entered, along with selecting the ICM modeling or indicating the value as “Seasonal input” or “Representative value,” as shown in figure 102. With these two cases, the user determines an appropriate design value for use throughout the analysis period, which accounts for climatic effects. Because FWD testing data were not available throughout the year, the ICM model was selected.

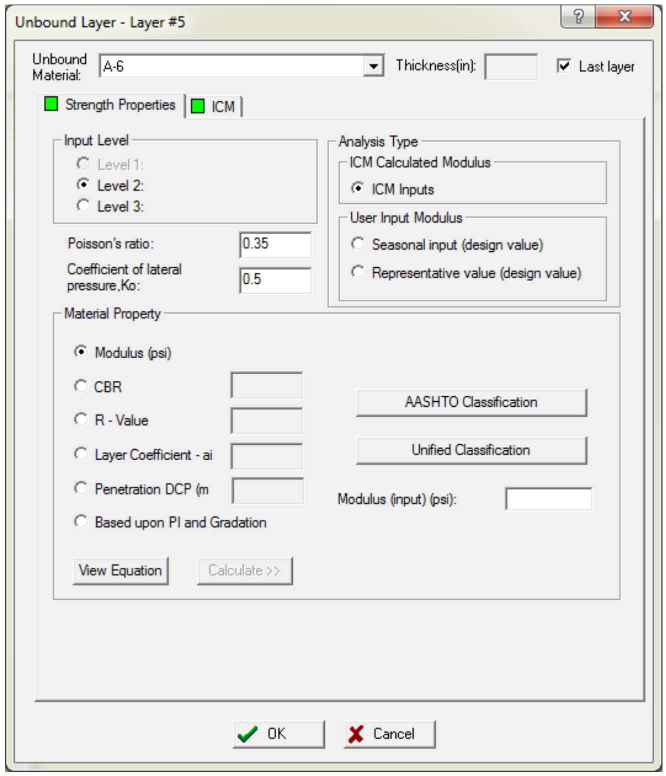


Figure 102. Screen Capture. Unbound layer input screen.

The HMA overlay rehabilitation design incorporated both the flexible pavement and rigid pavement design models. The MEPDG stated that the flexible pavement model was used to determine the stresses and strains for the HMA overlay performance models and the rigid pavement model was used to determine the stresses for the PCC performance models. However, the layer modeling for the rigid pavement design was not entirely clear. In the new rigid pavement design, a procedure was performed within the MEPDG program to convert the input layer moduli, for layers below the base course (bound or unbound), to an effective (or composite) *k*-value. Thus, the pavement layer system was converted to a three-layer system: PCC slab, base layer (bound or unbound), and subgrade (which included all layers below the base). A deflection basin was developed using layered elastic analysis, which was then used to backcalculate a *k*-value that represents all layers below the base layer for the equivalent pavement section. However, for the HMA overlay design analysis, the supplemental documentation indicated that the PCC layer stresses were determined using an equivalent PCC layer that incorporated the HMA overlay, PCC layer, and base course (that is, a two-layer system was used). (Note: the two-layer system was used in MEPDG version 1.0, and it was changed to a three-layered system in version 1.1.)

In the AREA-based backcalculation procedure, the determined *k*-value was the support value below the slab (or bound base layer, if present). Therefore, the backcalculated *k*-value was a composite value inclusive of the effects of the unbound aggregate layer. However, it was not clear in the program documentation how the properties of the unbound base layer and the subgrade *k*-value were effectively separated.

Several different analyses were conducted to evaluate the unbound base and subgrade *k*-value issue. The combinations in analyses 1 through 8 in table 109 were used to evaluate the subgrade and base properties used in the rehabilitation design. One method was to use the backcalculated *k*-value directly, ignoring the presence of the aggregate layer (analyses 1 and 3). A second method was to use the backcalculated *k*-value and to include an aggregate layer without adjustment of the *k*-value (analysis 4). A third analysis was conducted using a *k*-value adjusted for the thickness of the aggregate layer (analyses 5 and 6). As discussed in the laboratory comparison, in this latter analysis, the aggregate contribution to the *k*-value was backed out using the PCA's design method to arrive at what was theoretically the subgrade *k*-value.(13) A final analysis was run using only the estimated subgrade modulus from laboratory testing (analysis 2).

The flexible pavement design model was based on layered elastic analysis. What was unclear in the documentation was whether the subgrade modulus used in the layered elastic model was estimated from the input dynamic *k*-value or whether the layer modulus entered as part of the inputs was used. If the entered layer modulus was used, this value likely should be correlated with the backcalculated *k*-value in some manner.

Bound Layers

A modular ratio of 7 was selected to determine the backcalculation values for this case study based on the following considerations:

- Although the results from the resilient modulus testing appeared unreasonably high, it suggested the HMA was a stiffer mix.
- The resulting HMA modulus was in line with results of the predictive equations.
- The resulting PCC modulus was reasonable even though it was still greater than laboratory testing results.

For a composite pavement, the existing PCC layer was the only bound layer for which backcalculated data were used as an input—the MEPDG software at the time of this report did not use backcalculation inputs for the existing HMA overlay. The input for the existing PCC based on backcalculation was the elastic modulus; the PCC flexural strength was estimated using the equations in figure 98 and figure 99 and the backcalculated PCC modulus.

As previously described, the MEPDG suggested the backcalculated PCC modulus should be adjusted by a factor of 0.80 to correct from a dynamic value to a static value. The MEPDG further suggested the PCC modulus should be adjusted based on the overall condition of the pavement (because testing is generally conducted only at intact locations). Table 110 summarizes the condition factors suggested in the MEPDG.

Table 110. Condition factor values to adjust intact slab moduli.[6]

Pavement Condition	Condition Factor, C_{BD}
Good	0.42 to 0.75
Moderate	0.22 to 0.42
Severe	0.042 to 0.22

Because of the difficulty of assessing the overall condition of the PCC layer in a composite pavement, the general assessment needed to be based on judgment of the backcalculation results, surface distresses influenced by the underlying PCC (such as reflective cracking), information of the pavement before overlay, and other available data sources (such as cores). Considering these factors, the underlying PCC was likely in the "Good" condition. As summarized in table 109, a range of backcalculated PCC modulus adjustments was evaluated, with the assumption of 0.70 as a baseline input (analyses 1 and 8–11).

The rehabilitation design also considered the percentage of cracked slabs before rehabilitation and the percentage of slabs repaired as part of the rehabilitation. This input for a composite pavement was rather vague. For this analysis, it was assumed that all of the deteriorated cracks were repaired and that the existing reinforcement and dowel bars still provided adequate load transfer.

Evaluation of Design Results

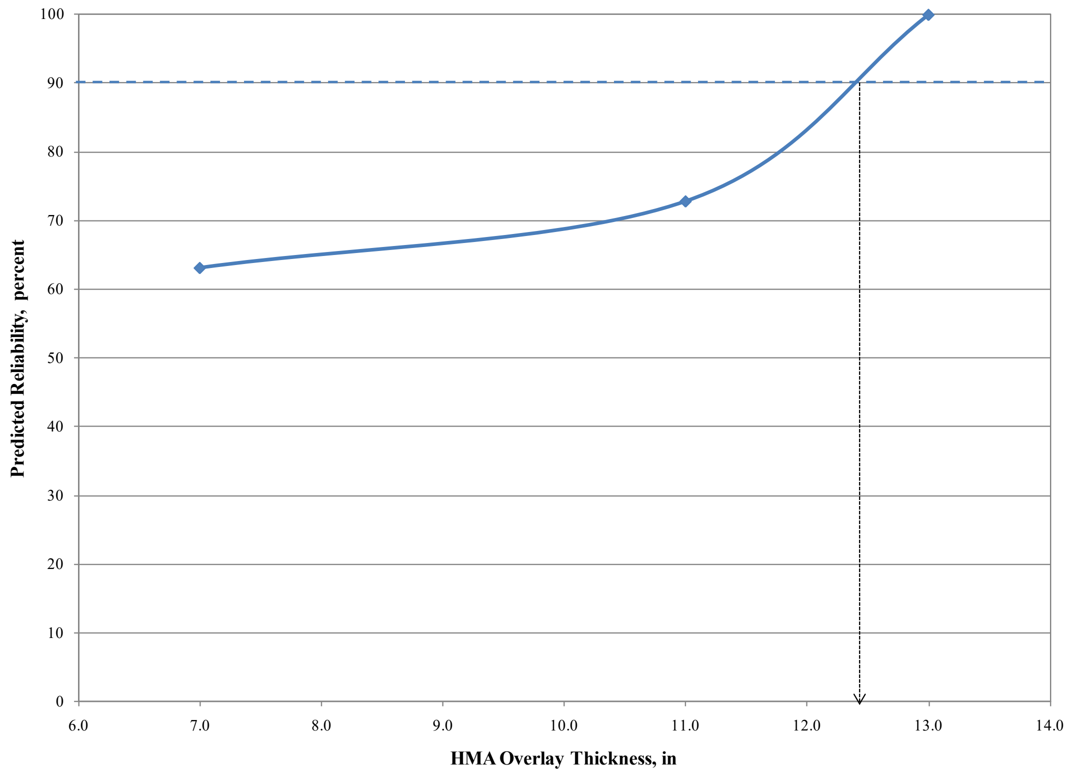
The various analysis scenarios summarized in table 109 were run with the MEPDG software (version 1.0) to evaluate the influence of varying the backcalculation-based inputs. The nationally calibrated performance models were used. Analysis 1 was used as a baseline to compare the results of the various input combinations. That is, the HMA overlay thickness requirement was determined for the baseline inputs and then the change in distress predictions and reliabilities using the same overlay thickness (but varied inputs) were compared.

The required HMA overlay thickness for analysis 1 was controlled primarily by top-down cracking (summarized in table 111), requiring an HMA overlay thickness of approximately 12.5 inches (318 mm) to achieve a 90-percent reliability or less than 2,000 ft/mi (378 m/km) of top-down cracking. This is shown in figure 103, which plots predicted distress as a function of overlay thickness on the left-hand scale and design reliability as a function of the overlay thickness on the right-hand scale. Figure 104 illustrates the stress prediction from the design program for the 12.5-inch (318 mm) overlay. Analysis 2 represented the use of laboratory-based inputs, and the distress predictions were essentially identical to analysis 1, summarized in table 111. This was not too surprising because the PCC inputs for each analysis differed by less than 10 percent, and the same subgrade material properties were used. What is worth noting is that the results were the same despite removing the dynamic *k*-value. The *k*-values were only 4 percent different in the Cracking Summary worksheet of the design summary output file, which was not expected with the difference of *k*-value input.

Table 111. Summary of design results for 378-mm (12.5-inch) HMA overlay.

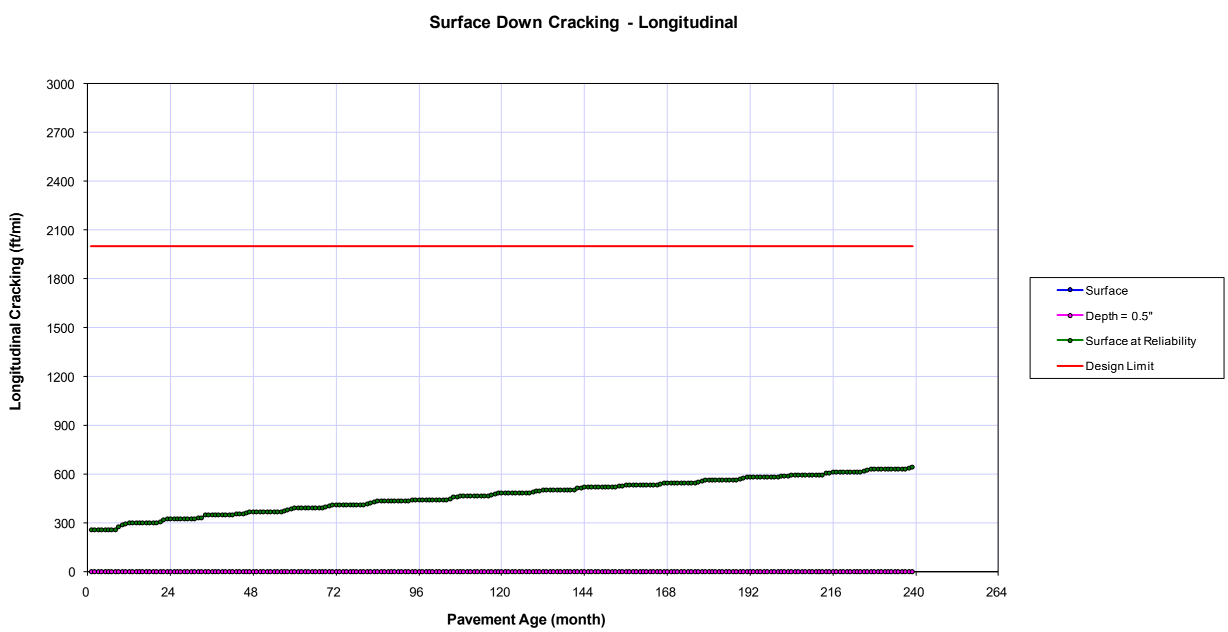
Variable	Backcalculation-Based Quantity (Analysis 1)	Backcalculation-Based Quantity at Reliability (Analysis 1)	Laboratory-Based Quantity (Analysis 2)	Laboratory-Based Quantity at Reliability (Analysis 2)
Terminal IRI (inch/mi)	99.6	99.48	99.6	99.48
Transverse cracking (percent slabs)	0	99.999	0	99.999
HMA surface down cracking (long cracking) (ft/mi)	2.6	99.999	2.7	99.999
HMA bottom up cracking, alligator cracking (percent)	0	99.999	0	99.999
HMA thermal fatigue, (transverse cracking) (ft/mi)	1	99.999	1	99.999
Chemically stabilized layer fatigue fracture (percent)	N/A	N/A	N/A	N/A
Permanent deformation—HMA only (inches)	0.14	98.95	0.14	98.96
Permanent deformation—total pavement (inches)	0.14	99.999	0.14	99.999

1 inch/mi = 0.0158 m/km.
1 ft/mi = 0.19 m/km.
1 inch = 25.4 mm.
N/A = Not applicable.



1 inch = 25.4 mm.

Figure 103. Graph. Summary of overlay thickness determination for analysis 1 using MEPDG design program.



1 ft/mi = 0.19 m/km.

Figure 104. Graph. Top-down cracking distress prediction from MEPDG design program for analysis 1.

A 12.5-inch (318-mm) HMA overlay is not a thickness that would be constructed by highway agencies in this application. To reduce the overlay thickness, one could assume a higher level of allowable top-down cracking distress and plan to perform periodic crack sealing. Alternately, the HMA mixture properties could be reexamined to develop a mixture more resistant to cracking. In addition, as mentioned previously, the performance models were calibrated for the local conditions. For the given inputs and ignoring the top-down cracking distress, HMA surface deformation became the next distress to control the design.

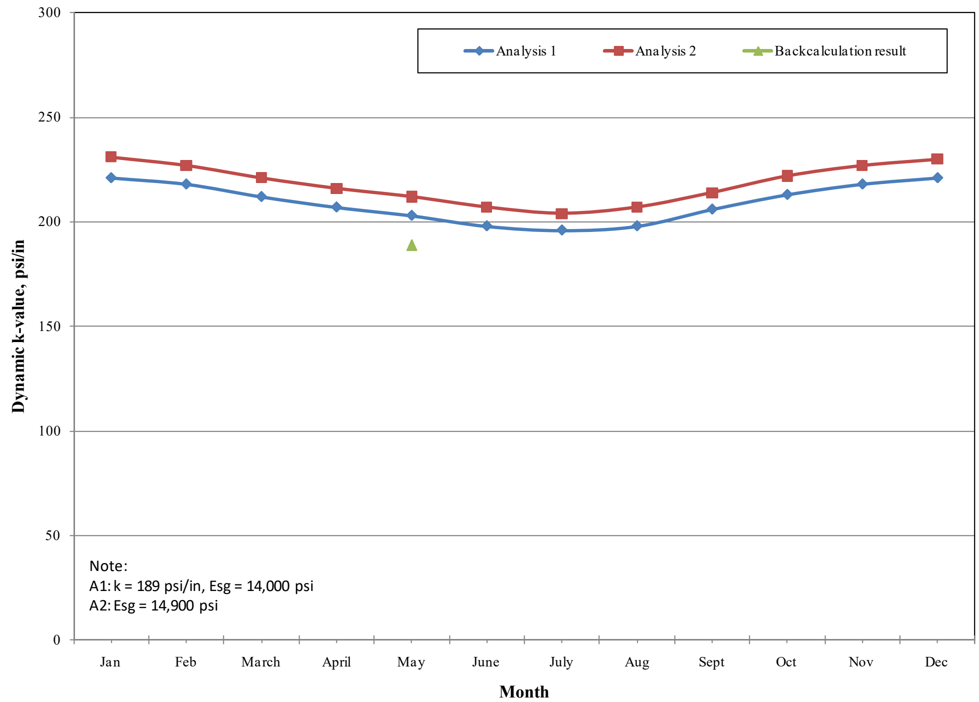
The additional design runs focused on the selection of appropriate layer adjustments and are discussed in the following sections.

Unbound Layers

Although the unbound layer inputs for the range evaluated did not have a significant influence on the final overlay thickness, the following observations were made during the course of assessing the various inputs:

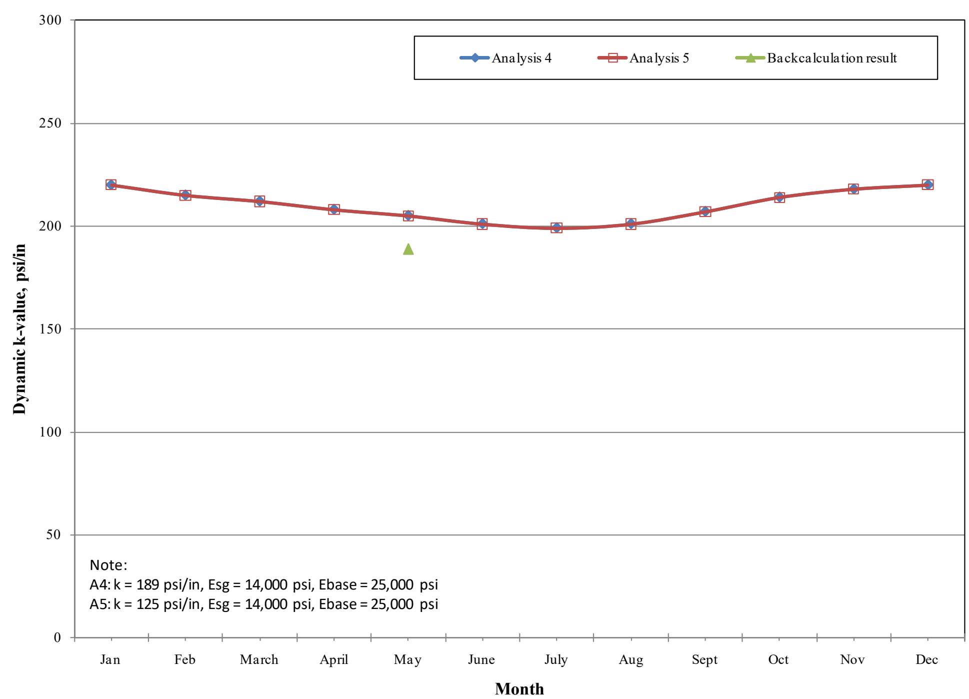
- The input dynamic *k*-value was not reflected in the summarized results. Although the dynamic *k*-value obtained from backcalculation can be entered for rehabilitation design, it was unclear whether that value was used in the design calculations. The *k*-values reported in the Cracking Summary worksheet of the MEPDG results summary did not correspond with the entered value, and changing the dynamic *k*-value input (or even not entering a value at all) produced no change to the summarized *k*-values (illustrated in figure 105 through and figure 107) or the distress quantities. However, changing the subgrade layer modulus value did change the reported *k*-values, as shown in figure 108. It did appear the design program calculated a seasonally adjusted *k*-value (figure 109) based on the input dynamic *k*-value (values reported in the "MonthlySeasonPattern.txt" file), but it did not appear these values carried through to the design results.

  Although the procedure was not undertaken to verify the values, it was possible the *k*-values reported in the design summary were those determined using the procedure discussed for the new design (that is, the use of layered elastic analysis to calculate a deflection basin based on the layer moduli and backcalculation of a *k*-value for the equivalent cross section). Alternatively, the ICM modeling may be incorporating an initial adjustment factor.



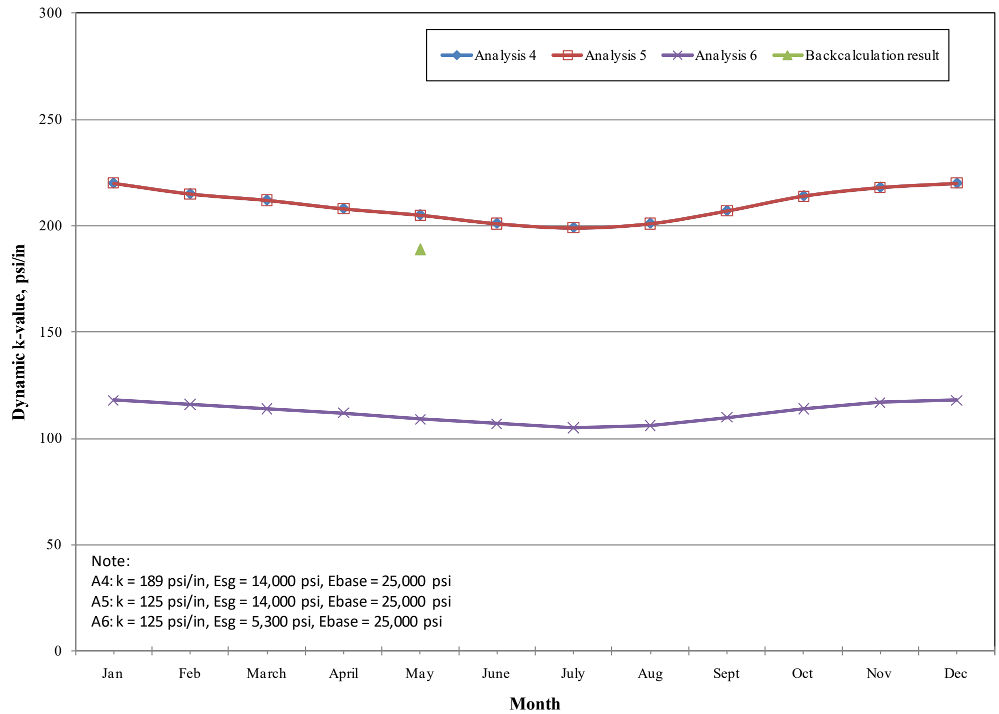
1 psi/inch = 0.263 kPa/mm.

Figure 105. Graph. Summary of *k*-values with and without entering dynamic *k*-value (analyses 1 and 2).



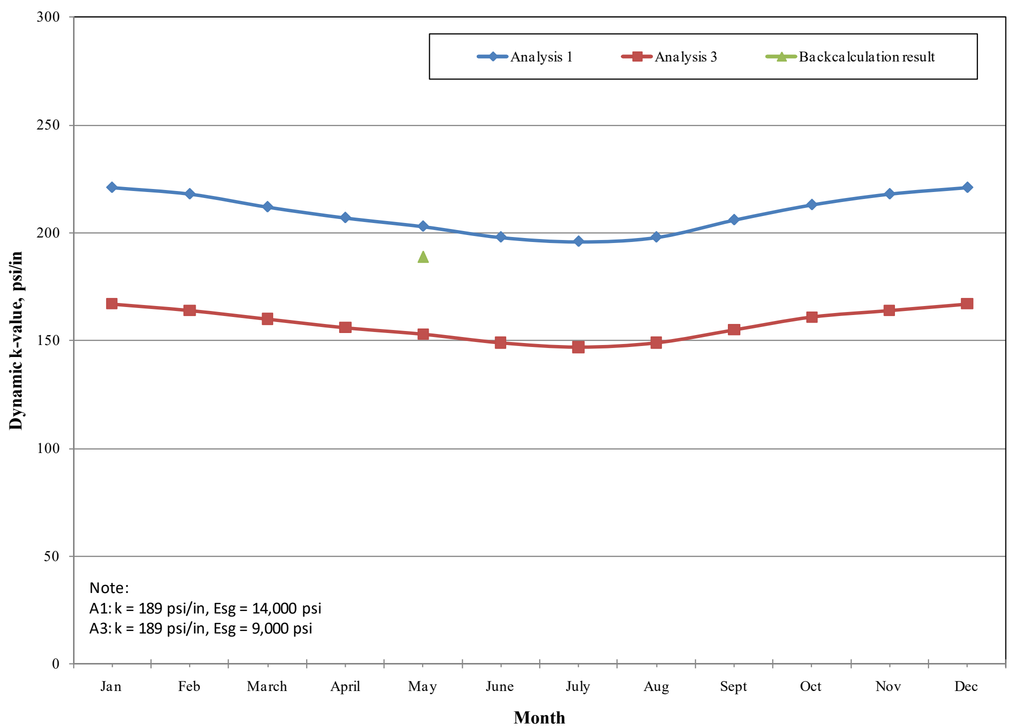
1 psi/inch = 0.263 kPa/mm.

Figure 106. Graph. Summary of *k*-values with changing input dynamic *k*-value (analyses 4 and 5).



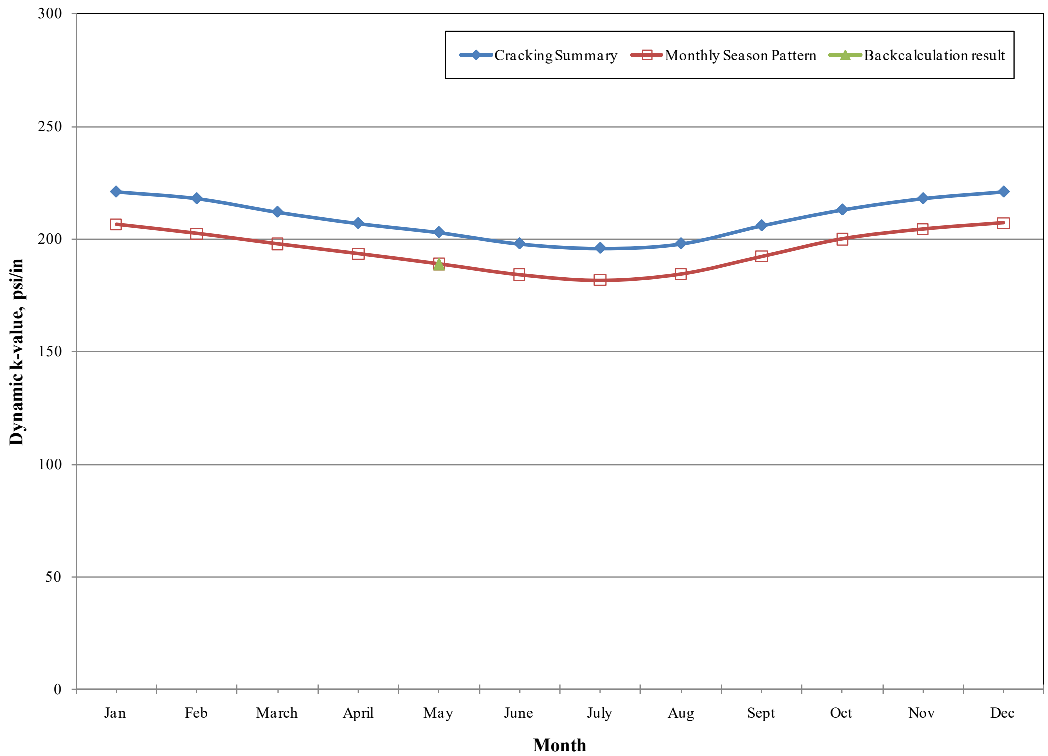
1 psi/inch = 0.263 kPa/mm.

Figure 107. Graph. Summary of *k*-values with base layer included (analyses 4–6).



1 psi/inch = 0.263 kPa/mm.

Figure 108. Graph. Summary of *k*-values with changing subgrade modulus (analyses 1 and 3).

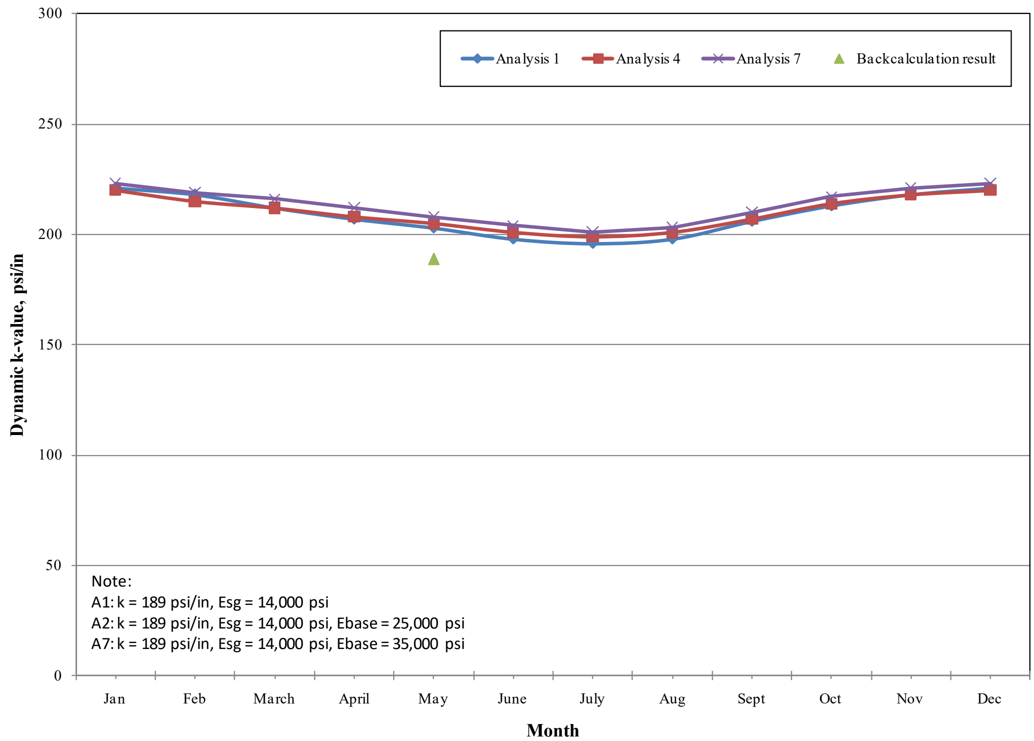


1 psi/inch = 0.263 kPa/mm.

Figure 109. Graph. Comparison of calculated *k*-value data sources.

- The use of a "Representative value" for the subgrade layer (instead of using the ICM model) did not change the *k*-values reported in the results summary. It did change the layer modulus values reported in the "Layers Modulus" worksheet of the output summary file to the value entered as the representative value; however, it did not change the overall required thickness results.

- The rigid pavement model used for the design, in terms of subgrade and base layering, was not entirely clear on how to address a base layer. The supporting documentation indicated a base layer was used in the equivalent pavement structure but it was incorporated into the equivalent slab for rehabilitation design. The issue in this case study was whether the existing base layer needed to be separated from the dynamic *k*-value in some manner and what pavement layer model was entered into the design program. Including a base layer (without adjustment to the subgrade inputs) did not result in a significant change in the reported *k*-values, as summarized in figure 110. The base layer modulus also appears to have had very little influence, but only two relatively similar modulus values were evaluated.



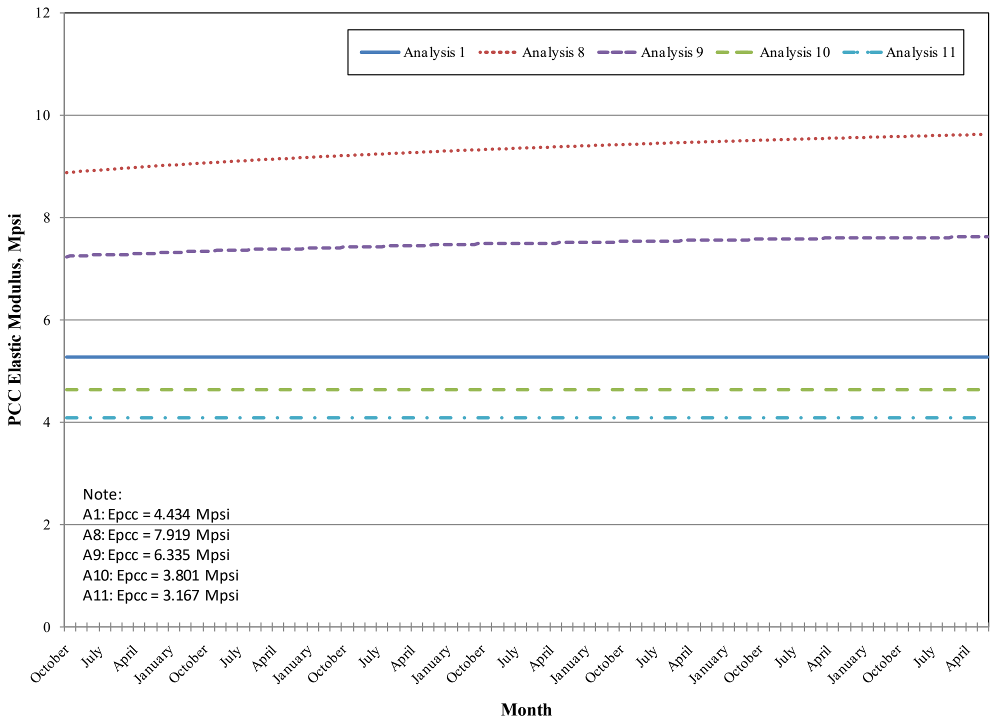
1 psi/inch = 0.263 kPa/mm.

Figure 110. Graph. Influence of base layer on *k*-value.

Bound Layers

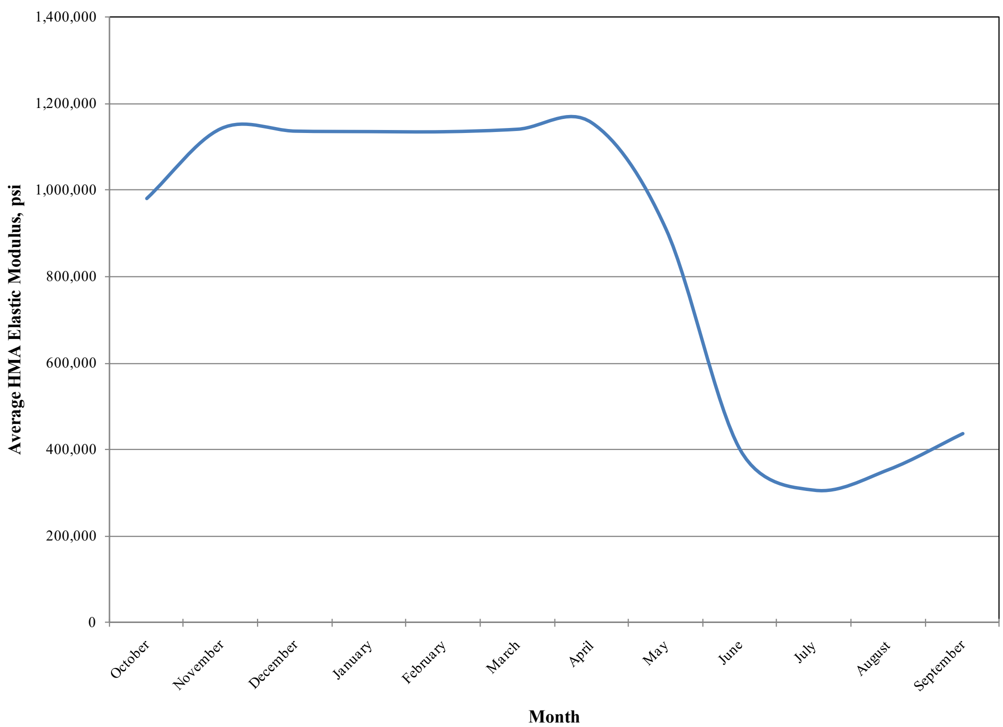
It appears that the rigid analysis used a dynamic PCC modulus even though the design input was the static PCC properties. The PCC strength properties contained in the results summary (Cracking Summary worksheet) were approximately 20 percent higher than the static values entered (the Layer Modulus results showed the input PCC modulus value); even though the documentation was not clear on this, it was assumed the reported values were dynamic. Two of five cases (the higher strength cases; analyses 8 and 9) had increasing PCC strength properties while three remained constant over time, as shown in figure 111. The MEPDG documentation discussed the trend of PCC strength gain over time, but it was unclear from this limited analysis why it apparently showed up in two of the cases. The differences in strength had no influence on the overall design results.

The HMA modulus results are illustrated in figure 112. The values for May were in the general range of the backcalculated HMA modulus; however, the modular ratio was adjusted based on resilient modulus testing and initial dynamic modulus estimates obtained from the design program. Therefore, the user needs to select the modular ratio during the backcalculation process using the best available information.



1 Mpsi = 6,890 MPa.

Figure 111. Graph. PCC modulus over time.



1 psi = 6.89 kPa.

Figure 112. Graph. HMA modulus over time.

SUMMARY

In this case study, the selection of layer property inputs based on backcalculation values had minimal influence on the overall design results, and results were the same as laboratory-based inputs. However, based on the analyses conducted, the research team makes the following recommendations:

- Continue to use the established dynamic backcalculation adjustment factor (0.8 for the PCC modulus) until new ones are developed or an agency develops more specific values. The MEPDG program requires entry of (dynamic/static) PCC elastic modulus values. Additional adjustment of the PCC modulus based on the overall pavement condition can be made, but it does not appear that this influence has an effect unless the pavement is in very poor condition.

- Continue to use the established modular ratios, but a preferred approach would be to employ specific testing data, if available, to determine project-specific ratios.(16)

- Correlate the subgrade modulus input to the static backcalculated *k*-value. Both the flexible and rigid design analyses in the HMA over JPCP rehabilitation appear to use the input subgrade elastic modulus (rather than the input dynamic *k*-value), so this value should be based on the determined support conditions.

- Do not include an aggregate base layer and then determine a corresponding dynamic subgrade *k*-value because this procedure does not appear to have a significant effect on the design results. The addition of a layer would also seem to suggest a change in the climatic adjustment, but it does not appear to be significant, particularly with regard to the overall design results. This conclusion is based on a limited number of runs for this specific case study and should be investigated further for other conditions.

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