Concrete Pavement Performance in the Southeastern United States

by

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16. Abstract

This report documents an in-depth study of the performance of concrete pavements in the southeastern United States. Information from the Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) database was investigated. Analysis of 36 sections in Alabama, Florida, Georgia, Mississippi, and North and South Carolina showed that the majority of these pavements are providing excellent service well beyond their original design lives. This has important implications for new pavement construction, as well as maintenance and rehabilitation of existing pavements.

For new pavement construction, the results of this study suggest that life cycle cost models should assume better performance and longer service life than existing AASHTO predictions for these pavements. Thus, the economic benefits of constructing concrete pavements where heavy traffic is anticipated or long life is desired may be considerable.

The implication for maintenance and rehabilitation of existing pavements is that concrete pavements may have considerably more remaining structural capacity than time in service or traffic applied to the pavement would suggest. For this reason, expensive and time-consuming reconstruction efforts or thick overlays should not be used unless the evaluation of pavement condition indicates it is warranted. If the pavement is in good structural condition, diamond grinding and other rapid, low cost Concrete Pavement Restoration (CPR) alternatives may extend pavement life considerably and improve serviceability.

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Executive Summary

This report documents an in-depth study of the performance of concrete pavements in the southeastern United States. Information from the Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) database was investigated. Analysis of 36 sections in Alabama, Florida, Georgia, Mississippi, and North and South Carolina showed that the majority of these pavements are providing excellent service well beyond their original design lives. This has important implications for new pavement construction, as well as maintenance and rehabilitation of existing pavements.

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Objectives of this study were:

- To investigate the performance of concrete pavement in the southeastern United States, as documented in the SHRP LTPP database,
- To compare performance to that predicted by the AASHTO 1993 design procedure to determine if these sections were meeting or exceeding expectations,
- To compare actual pavement thickness for each section to design thickness calculated using Portland Cement Association, AASHTO 1993 and AASHTO 1998 procedures to determine which procedure was most appropriate for design in this region, and
- To determine which design features lead to superior concrete pavement performance in this region.

The Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) database was used to evaluate the performance of some of the 41 concrete pavement sections located in the southeastern United States. Jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavement (CRCP) test sections from the General Pavement Studies (GPS) experiments 3 and 5 were considered, ranging in age from 11 to 34 years. Jointed reinforced concrete pavement (JRCP) was dropped from the analysis because of its limited use in this region, and some other sections were lacking traffic or material information. States

considered included Alabama, Florida, Georgia, Mississippi, North Carolina, South Carolina, and Tennessee. However, Tennessee sections did not have any information in the database and were dropped from the study. During this time, the growth of population and traffic in the southeast has been explosive. Therefore, it is probable that many or most of these sections are well beyond their initial design traffic volumes.

For the 36 sections studied and the assumptions documented in the body of the report, this research suggests the following conclusions and recommendations for concrete pavement design and rehabilitation in the Southeast United States:

- JPCP with undoweled joints should not be used, particularly for pavements with 5 million or more ESALs of anticipated traffic.
- Since existing JPCP with undoweled joints may have considerable remaining fatigue capacity, dowel retrofit followed by diamond grinding should be considered to restore serviceability.
- Doweled JPCP 229 to 254 mm thick (9 to 10 inches) often carries as much as 10 to 20 million ESALs over 25 or more years.
- The AASHTO 93 procedure appears to produce reasonable designs for doweled JPCP.
- The AASHTO 98 procedure is unnecessarily conservative for doweled JPCP.
- CRCP 196 to 234 mm thick (7.7 to 9.2 inches) often carries as much as 7 to 19 million ESALs over 21 to 30 years. Thus, performance similar to doweled JPCP may be achieved with a 10 to 15 % reduction in pavement thickness. These designs may also have considerable remaining life.
- Both the AASHTO 93 and 98 procedures are unnecessarily conservative for CRCP. Pavements 20 % thinner than required by either AASHTO method have performed very well.
- Major rehabilitation decisions should be based on pavement condition and not age or cumulative traffic. Pavements 25 to 30 years old that have carried 10 to 25 million ESALs are still performing well, and may be capable of carrying much more traffic.

Section 1
Introduction

Objectives
Objectives of this study were:

- To investigate the performance of concrete pavement in the southeastern United States, as documented in the SHRP LTPP database,
- To compare performance to that predicted by the AASHTO 1993 design procedure (AASHTO, 1993, Huang, 1993) to determine if these sections were meeting or exceeding expectations,
- To compare actual pavement thickness for each section to design thickness calculated using Portland Cement Association (PCA, 1984, Huang, 1993), AASHTO 1993 (AASHTO, 1993, Huang, 1993) and AASHTO 1998 (AASHTO 1998) procedures to determine which procedure was most appropriate for design in this region, and
- To determine which design features lead to superior concrete pavement performance in this region.

Scope
The Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) database (FHWA, undated, LAW PCS, 1999) was used to evaluate the performance of some of the 41 concrete pavement sections located in the southeastern United States. Jointed plain (JPCP) and continuously reinforced (CRCP) test sections from the General Pavement Studies (GPS) experiments 3 and 5 were considered, ranging in age from 11 to 34 years. States considered included Alabama, Florida, Georgia, Mississippi, North Carolina, South Carolina, and Tennessee. However, Tennessee sections did not have any information in the database and were dropped from the study. During this time the growth of population and traffic in the southeast has been explosive. Therefore, it is probable that many or most of these sections are well beyond their initial design traffic volumes.

The pavement designs available and considered by type and state are listed in Table 1-1. Three jointed reinforced concrete pavement (JRCP) sections (GPS 4) were found in the database but were not considered further in this study. The limited sample suggests that this type of pavement is not used much in the region. Two sections, both JPCP, did not have enough data available and could not be studied further. Either traffic or material information was insufficient. Thus, the final study sample comprised 22 JPCP and 14 CRCP sections. Twelve sections from the PCC structural factors (SPS-2) experiment in North Carolina were also investigated. However, these sections are only about 5 years old and no significant distress has been observed, so it is too early to analyze them. A sign showing the location of a typical section is shown in Figure 1-1.

Table 1-1. Pavement Sections Available and Considered

State	JPCP Available	JPCP Considered	JRCP Available	JRCP Considered	CRCP Available	CRCP Considered	Total Available	Total Considered
Alabama	1	1	2	0	2	2	5	3
Florida	7	6					7	6
Georgia	8	8			1	1	9	9
Mississippi	2	2	1	0	5	5	9	7
North Carolina	5	4			3	3	8	7
South Carolina	1	1			3	3	4	4
Tennessee								
Total	24	22	3	0	14	14	41	36

Figure 1-1. Road test section 1-3028, Alabama JPCP, I-59 North, Jefferson County.

Plan of Work

The following tasks constituted the work plan for this project:

a. Conduct a literature review, with special attention to the Federal Highways Administration reports documenting the experimental design and data collection methods and lessons learned on concrete pavement performance.

b. Design an investigation using the database to determine the design features that improve pavement performance in this geographic region.

c. Retrieve and analyze the data from the database.

d. Analyze selected sections using the AASHTO 1993 equations, the PCA method, and the new 1998 AASHTO procedure to determine whether the performance has been better or worse than predicted by design procedures currently in use.

e. Write a final report documenting the results of the study.

Organization

The experimental design and results are presented in the following sections. Background is presented in Section 2 and methodology in Section 3. Section 4 presents project results and discussion, and conclusions and recommendations are provided in Section 5.

Section 2
Background

For the 36 sections considered, the first step was to develop comparisons between actual and predicted performance and analyze the results. Next, the actual pavement thickness for each section was compared to the design thickness obtained using a variety of procedures. This methodology is discussed in section 3.

For the performance comparison, measured pavement International Roughness Index (IRI) was converted to Present Serviceability Index (PSI) and plotted against the 1993 AASHTO design equation. Some design information could be extracted directly from the database using the DataPave 2.0 software, as listed in Table 2-1.

Table 2-1. Design Information from DataPave

Design Parameter	DataPave Information
Pavement thickness	Mean thickness from section report
Pavement roughness	International Roughness Index (IRI) by year
Traffic, in ESALs	Estimated and measured ESALs, by year (not complete)
Soil under pavement	California Bearing Ratio (CBR) or AASHTO Soil Classification
Shoulder type	Tied (concrete) or asphalt
Drainage	Undrained, longitudinal, drainage blanket
Joint type, spacing	Doweled, aggregate interlock

The data fields listed in DataPave 2.0 appear at first glance to provide all necessary information for analysis and design. However, on closer inspection, many data are missing and must be assumed from other parameters. For example, the modulus of subgrade reaction k, an important design input, was not provided for the sections considered. Table 2-2 lists the assumptions made as well as the basis for those assumptions.

Table 2-2. Design Assumptions

Design Assumption	Basis
Total cumulative traffic	Estimated and measured traffic and interpolation
Modulus of subgrade reaction k	Correlations to CBR and AASHTO Soil Classification (PCA 1984, AASHTO 1998)
Concrete flexural strength	Assumed 4.48 MPa (650 psi) (AASHTO 4993)
Concrete modulus of elasticity	Assumed 25.6 Gpa (3.71 million psi), consistent with assumed concrete flexural strength
Standard deviation	Assumed 0.34 based on AASHTO (AASHTO 1993), traffic variation not considered
Reliability	50 % for comparing performance (Figures 1 through 8) and 85 % for determining design requirements (Tables 6 and 7)
Drainage coefficient C_d	0.9 to 1.1, based on drainage type
Joint J	2.6, 2.8, 3.1, 3.2, or 4.1 based on pavement, joint, and shoulder type

Data on AASHTO soil classification and California Bearing Ratio (CBR) is available in many cases, and k may then be estimated using Portland Cement Association (PCA, 1984, Huang, 1993) or AASHTO supplement (AASHTO 1998) charts. In the absence of other data, the layer descriptions provided in the Section Reports may be used for rough estimates along with

AASHTO guide charts (AASHTO 1986, AASHTO 1993). In some cases, curves representing the highest and lowest estimated k values were developed and shown on the same figure. For Alabama and Mississippi, the CBR could be used to find a single value of k. For other states, the AASHTO soil classification was used, which may be correlated to a range of k values.

Using this design information and an assumed initial serviceability index of 4.5, traffic in ESALs corresponding to PSI values of 4.0 to 0 (at 0.5 increments) was calculated and plotted as shown in the figures in Section 4. Some figures represent cases where two different values of k were used. Next, field measured IRI values converted to PSI were plotted, using an equation from Hall and Correa (1999). Although 36 plots were generated during this research, only 8 representative figures are provided in this report.

The cumulative traffic corresponding to the measured IRI values was projected using both estimated and measured traffic by year from the database. Although this information was incomplete and in some cases contradictory, the cumulative traffic is probably reasonable since overestimation and underestimation would tend to cancel out over the 11 to 34 year period. Where both measured and reported traffic were shown for the same year, they often did not agree – one or the other was used, based on which better matched the traffic information from previous and subsequent years. Cumulative traffic ranged from less than half a million to nearly 30 million 80 kN (18 kip) Equivalent Single Axle Loads (ESALs).

Section 3
Methodology

AASHTO Design Equation

The 1993 AASHTO design equation was used to compare design performance under predicted traffic load with actual performance. 1993 AASHTO rigid pavement design equation (AASHTO 1993) is:

$$\log_{10}(W_{18}) = Z_R \cdot S_o + 7.35 \cdot \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624 \cdot 10^7}{(D+1)^{8.46}}}$$
$$+ (4.22 - 0.32 \cdot p_t) \cdot \log_{10}\left[\frac{S'_c \cdot C_d \cdot (D^{0.75} - 1.132)}{215.63 \cdot J \left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right] \qquad (3-1)$$

Where

W_{18} predicted number of 80 kN (18-kip) Equivalent Single Axle Load (ESAL) applications determined from the equation above and then converted to thousand of a Single Axle Load (KESAL).

Z_R standard normal deviate assumed to be equal to 0 for reliability of 50% ("actual" performance) and to 1.037 for reliability of 85% (design performance), where higher reliability requirement demands a thicker design.

S_O combined standard error of the traffic prediction and performance prediction assumed to be equal to 0.35 for rigid pavements.

D thickness (25 mm increments or inches) of pavement slab provided by the database.

ΔPSI difference between the initial design serviceability index, p_o, and the design terminal serviceability index, p_t, which varies from 0 (for $p_o = p_t = 4.5$) to 4.5 (for $p_o = 4.5$, $p_t = 0$)

S'_c modulus of rupture (psi) for Portland cement concrete used on a specific project assumed to be equal to 4.48 MPa (650 psi) for concrete with normal aggregates.

J load transfer coefficient used to adjust for load transfer characteristics of a specific design. The material type (Asphalt or Tied PCC) and presence of Load Transfer Devices for outer shoulder of each section were used to make assumptions. The range of values used was 2.6 – 4.1.

C_d drainage coefficient. For the Pavement's Exposure to Saturation from 5 to 25% applicable to the region of interest, C_d was assumed to be equal to 1.10 for drainage blanket and longitudinal drains, 1.00 in case of unknown drainage systems, and 0.90 for no drainage system.

E_c modulus of elasticity (psi) for Portland Cement Concrete (PCC).

k modulus of sub-grade reaction (psi) was listed as a field in DataPave 2.0. However, no k values were available for the sections of interest. Therefore, k was estimated from California Bearing Ratio and/or from the AASHTO type of soil and adjusted according to the treatment of section's sub-layers.

Assuming that initial serviceability level of the constructed section was 4.5 (PSI) and eventually decreased, the interval of serviceability was 4.5 to 0.0. To represent graphically a design model, an increment of serviceability decrease of 0.5 was used. That gave 10 design points to generate a curve.

It was not known which Standard Normal Deviate (Z_R) was considered in the design procedure. Therefore, two values of Z_R were involved in analysis of the design model - one for reliability of 50% and another for 85%. This difference in Z_R values was reflected in the difference in slopes of graphs representing pavement performance.

Another uncertainty in the presentation of the design curve was dependant on the choice of a k value. For some states (Alabama and Mississippi) a CBR value was found in LTPP database. Other states' information was interpreted in terms of AASHTO soil classification. By converting a CBR value into modulus of sub-grade reaction, one gets a single value, while AASHTO soil type provides a range of k values for a given soil type (AASHTO 1998). Since there was no method known to determine which modulus from this range was most descriptive for the section of interest, it was logical to use two extremes of this range. Therefore, two values of modulus of sub-grade reaction appear in calculations: k_{max} and k_{min}. Actual pavement performance would be expected to fall in-between of two design curves based on two k values. Assumptions made above resulted in generating of 2 to 4 design curves of pavement performance. These design curves were analyzed and compared with actual pavement performance.

Because two different k values could be assumed, five of the figures in Appendix A (all but A-4) have two AASHTO curves. In this case the two curves should be considered as the upper and lower limits on a band.

Traffic Estimation

In order to compare the actual traffic data provided by LTPP database with the results of the AASHTO design procedure, it was necessary to present these two sets of data in compatible formats. Since the AASHTO equation is based on the relationship between traffic in the Equivalent Single Axle Load (ESAL) and level of serviceability in Present Serviceability Index (PSI), it was necessary to make sure that LTPP provided equivalent actual data for a section of interest. The program's database has two types of traffic information: estimated and monitored. This information is stored in DataPave 2.0 under TRF_EST_ANL_TOT_LTPP_LN and

TRF_MONITOR_BASIC_INFO respectively. These were used to estimate a cumulative traffic in ESALs.

IRI to PSI Conversion

The serviceability level of a section is described in the database through the International Roughness Index (IRI), so it had to be converted to PSI in order to be compared with AASHTO design model. The IRI value was converted to PSI using a relationship from Hall and Correa (Hall and Correa 1999).

$$PSI = 5 + 0.6046\, x^3 - 2.2217\, x^2 - 0.0434\, x \qquad (3\text{-}2)$$

Where

$$x = \log(1 + SV) \qquad (3\text{-}3)$$

and

$$SV = 2.2704\, IRI^2 \qquad (3\text{-}4)$$

It was established that FHWA LTPP program database presents required data in terms of IRI and two types of traffic data: estimated and monitored. It is also known that AASHTO design model procedure applied to design pavements represents the same data in terms of PSI and predicted traffic load.

Actual performance information was limited by the number of times the roughness of a section was measured. Another factor limiting the analysis was the lack of measured traffic data. A couple of readings were required in order to estimate the value of traffic for a certain year – a year for which PSI was given or any other year. The estimation of traffic for that particular year had to be performed through interpolation and/or extrapolation of given traffic data. A value of the cumulative traffic for the year of interest was a summation of loads for all the years a section was in use. Although some gaps were found in LTPP database, they were filled with estimated values of section's traffic.

To create a graph at least two points were needed. Therefore, if it was impossible to find at least two values for actual serviceability of a section and to closely estimate a traffic load for the same years, it was not possible to generate a graph of actual pavement performance. For those sections there was nothing to compare with design graphs based on the AASHTO design model and no analysis of pavement performance could be done. As a result, 4 of the 42 sections could not be analyzed further: 1-4007, 12-3804, 12-3811, and 37-3008. Following the analytical procedure described in this section, a graph was created for each section included in the study.

Section 4
Results and Discussion

Actual performance versus predicted performance was plotted for the 36 sections for which enough design and traffic data was available. Examples are shown in Figures 4-1 and 4-2, and in Appendix A in Figures A-1 through A-6. For the AASHTO performance prediction, the mean (50 % reliability) was assumed.

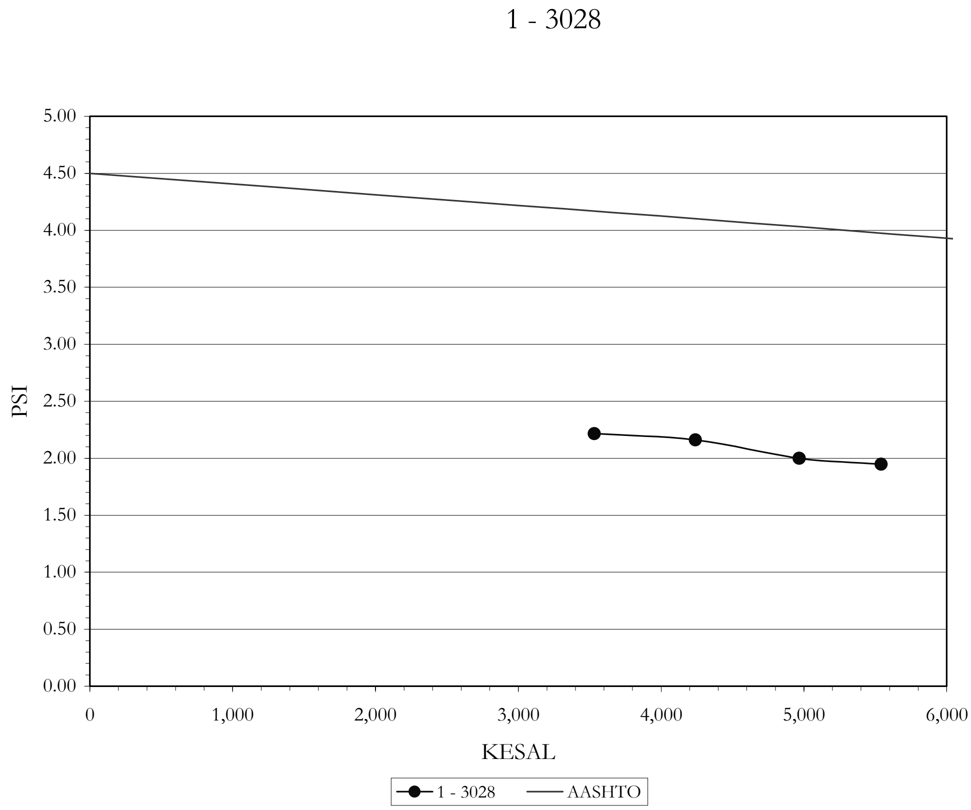

Figure 4-1. Section 1-3028, Alabama JPCP, aggregate interlock joints.

JPCP Performance Comparison

Sample results for JPCP sections are shown in Figure 4-1 and in Appendix A, Figures A-1 through A-3. Information on the 22 JPCP sections is provided in Table 4-1. The table includes state, section number, and route, age (as of the year 2000), drainage type, joint type, IRI (as of the year 1999), PSI (estimated from IRI), Average Daily Truck Traffic (ADTT, provided for the latest year available), and cumulative traffic in thousands of ESALs (KESALs). JPCP sections ranged in age from 11 to 34 years, were 201 to 284 mm (6.4 to 13.3 inches) thick, and had cumulative traffic of 388 thousand to 27.5 million ESALs.

Table 4-1. JPCP Sections in Study

State Section #	Route #	Age (years)	Slab thickness mm	Slab thickness inches	Type of drainage	Type of joint	IRI (m/km)	PSI (est.)	ADTT	Cumul. KESALs
Alabama										
1-3028-1	59	29	259	10.2	blanket	aggregate	3.7	1.96	2106	5542
Florida										
12-3804-1	75	15	305	12	none	dowels	1.9	3.44	1335	1166
12-3811-1	10	24	239	9.4	long. drains	aggregate	2.5	2.84	1264	12528
12-4000-1										6076
12-4057-1	75	14	338	13.3	blanket	dowels	0.9	4.58	770	11155
12-4059-1	1	11	163	6.4	long. drains	dowels	1.3	4.12	65	1143
12-4109-1	1	11	180	7.1	long. drains	dowels	2.0	3.33	65	1042
12-4138-1	92	26	203	8	none	aggregate	3.2	2.29	151	5659
Georgia										
13-3007-1	5	19	239	9.4	none	dowels	1.7	3.66	280	388
13-3011-1	16	25	262	10.3	blanket	none	1.1	4.36	1060	2216
13-3015-1	16	21	254	10	long. drains	dowels	1.4	4.00	594	3293
13-3016-1	20	23	284	11.2	long. drains	dowels	1.4	4.00		19647
13-3017-1	20	27	251	9.9	blanket	aggregate	1.3	4.12	1549	12752
13-3018-1	20	27	251	9.9	blanket	aggregate	1.2	4.24	1584	18802
13-3019-1	23	19	231	9.1	none	dowels	1.6	3.77	2240	2679
13-3020-1	300	15	254	10	none	dowels	1.4	4.00	698	2626
Mississippi										
28-3018-1	72	16	236	9.3	none	dowels	1.7	3.66	522	2250
28-3019-1	72	16	239	9.4	none	dowels	2.1	3.23	522	2151
North Carolina										
37-3008-1	74	16	201	7.9	none	dowels	2.0	3.33		
37-3011-1	95	23	254	10	unknown	dowels	1.6	3.77		16957
37-3044-1	85	34	229	9	unknown	dowels	2.0	3.33		27517
37-3807-1	52	20	239	9.4	none	none	1.8	3.55		3578
37-3816-1	147	27	236	9.3	none	dowels	2.1	3.23		6897
South Carolina										
45-3012-1	77	19	254	10	none	dowels	1.2	4.24	1400	17247
Tennessee										
None										

Alabama section 1-3028 (Figure 4-1) represents the worst performance in the study, with a current PSI of only about 2.0 after 5.5 million ESALs. Closer inspection of the damage mechanisms reveals considerable joint faulting, which is to be expected since the pavement had aggregate interlock joints. Other sections without doweled joints, such as Florida sections 12-3811 and 12-4138, also performed poorly. These are probably good candidates for dowel retrofit followed by diamond grinding.

In contrast, other sections studied showed much better performance, as documented in Appendix A. Florida section 12-4057 (Figure A-1) is in excellent condition after more than 11 million

ESALs. At 13.3 inches, it is the thickest pavement in the study. It is also one of the newest pavements in the study sample, with a construction date of 1986.

Georgia section 13-3018 (Figure A-2) also has excellent serviceability after 18.8 millions ESALs and 27 years of service. North Carolina section 37-3044 (Figure A-3) has seen the heaviest traffic, 27.5 million ESALs in 23 years, yet still has a serviceability index near 3.5. Its current serviceability is more than twice as high as predicted by the 1993 AASHTO equations.

Overall, of the 22 JPCP sections, all but 3 have a current PSI greater than 3.0, as estimated from IRI. Those three all have aggregate interlock rather than doweled joints. At 3.0 level of serviceability, less than 12 % of users would see a need for rehabilitation or other action (AASHTO 1993). With the exception of Florida, these pavements are 201 to 284 mm (7.9 to 11.2 inches) thick. Florida sections ranged from 163 to 338 mm (6.4 to 13.3 inches), a much wider range than in other states. Most of the performance comparison plots for JPCP gave reasonably good agreement between the AASHTO 93 equation prediction and observed field performance, generally within 0.5 to 1 PSI.

CRCP Performance Comparison

Sample results for CRCP sections are shown in Figure 4-2 and in Appendix A, Figures A-4 through A-6. Information on the 14 CRCP sections is provided in table 4-2. CRCP sections ranged in age from 21 to 30 years, were 196 to 234 mm (7.7 to 9.2 inches) thick, and had cumulative traffic of 2.44 to 22.2 million ESALs. Thus, the variations in age, pavement thickness, and traffic are much smaller for the CRCP sections than the JPCP sections.

Alabama section 1-5008 (Figure 4-2) represents excellent performance, with a current PSI of over 4.5 after 11.4 million ESALs and 24 years. Similar trends are seen for other sections, as represented in Appendix A by Mississippi section 28-5805 (Figure A-4) and North Carolina section 37-5827 (Figure A-5), both of which have serviceability above 4.0 after 25 to 27 years and 17.7 and 3.24 million ESALs, respectively. Section 45-5017 (Figure A-6) from South Carolina represents the only CRCP section that is performing below expectations, and the difference between actual and predicted performance is only about 0.5 PSI.

Overall, all 14 CRCP sections have a current PSI greater than 3.33, and all but three are greater than 4.0. In all but two cases, actual performance exceeds the AASHTO 93 equation prediction, often by a considerable margin. Figures 4-1, A-4, and A-5 are representative of the 12 sections that are exceeding expectations.

Actual and Design Thickness Comparison

The actual pavement thickness for each section is compared to the PCA, AASHTO 1993, and AASHTO 1998 design thickness in Table 4-3 for JPCP and Table 4-4 for CRCP sections. Only sections with heavy traffic (more than 5 million ESALs) were considered. The design thickness in each case was calculated to carry the traffic that had already been imposed on the pavement (5 to 27.5 million ESALs over 11 to 34 years), to a current PSI of 3.0. Note that unlike the performance plots, which did not use a reliability term, a reliability of 85 % was assumed for the AASHTO 93 and 98 procedure design calculations.

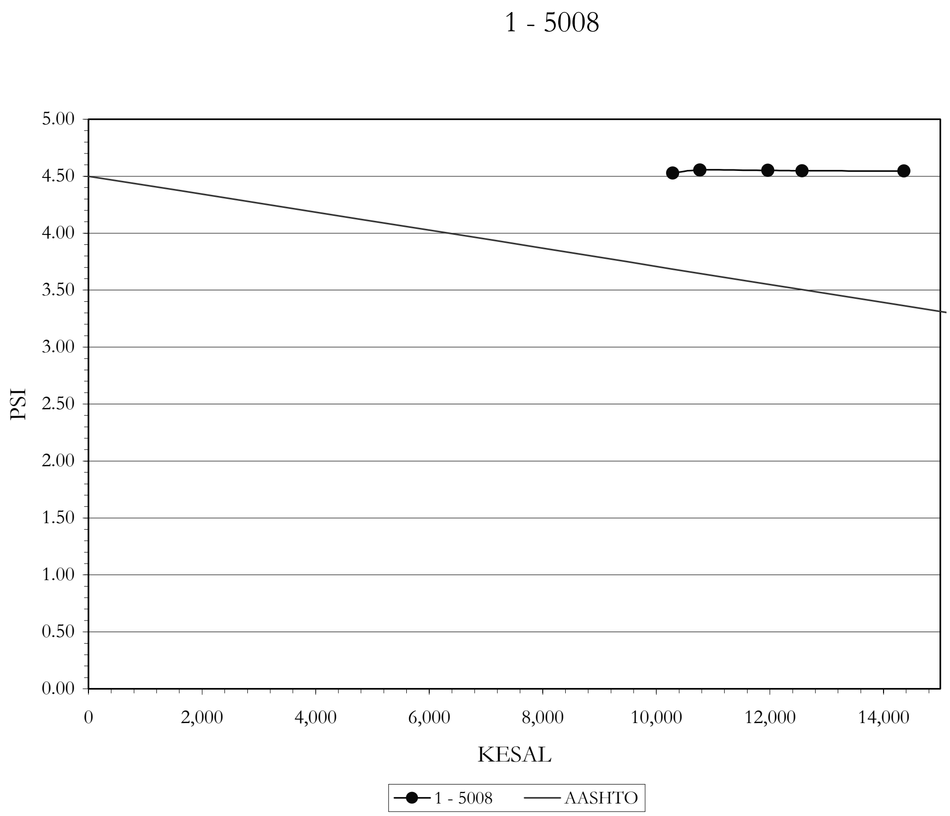

Figure 4-2. Section 1-5008, Alabama CRCP

For example, Alabama section 1-3028 has an actual thickness of 259 mm (10.2 inches). For an ADTT of 2106, the PCA design procedure predicts a thickness of 221 mm (8.7 inches), the AASHTO 98 procedure a thickness of 250 mm (9.83 inches) and the AASHTO 93 procedure a thickness of 252 mm (9.91 inches). Dividing the design values by the actual thickness gives ratios of 0.85, 0.96, and 0.97 respectively between the PCA, AASHTO 98, and AASHTO 93 designs and actual thickness, as shown in Table 6. For cases where the AASHTO soil classification was used to predict k, minimum and maximum design thicknesses corresponding to maximum and minimum k were developed.

In most cases the ratio for the PCA method is low. There are several possible explanations. The ADTT used may not be consistent with the cumulative number of ESALs. Also, the PCA design method uses fatigue and erosion failure criteria, which would be consistent with a PSI much less than 3.0. As a result, it is probably not possible to draw conclusions from the PCA comparison.

In contrast, the AASHTO 98 designs exceed actual thickness by 8 to 27 % for JPCP, and 20 to 28 % for CRCP, on average. AASHTO 93 designs are closer to actual thickness for JCPC (3 % less to 5 % more, on average), but 19 to 22 % high for CRCP, on average.

Table 4-2. CRCP Sections in Study

State Section #	Route #	Age (years)	Slab thickness mm	inches	Type of drainage	IRI (m/km)	PSI (est.)	ADTT	Cumul. KESALs
Alabama									
1-3998-1	20	28	208	8.2	None	1.3	4.12		11365
1-5008-1	20	24	234	9.2	None	0.9	4.58	2836	14358
Florida									
none									
Georgia									
13-5023-1	95	26	213	8.4	None	1.4	4.00	1995	22243
Mississippi									
28-3099-1	20	30	203	8	None	1.4	4.00	1831	4522
28-5006-1	78	21	208	8.2	None	1.4	4.00	1833	6189
28-5025-1	84	22	211	8.3	None	1.2	4.24	329	2443
28-5803-1	78	21	201	7.9	None	1.7	3.66	1837	7249
28-5805-1	10	25	208	8.2	None	1.2	4.24	2307	17734
North Carolina									
37-5826-1	77	23	203	8	Unknown	1.2	4.24		13322
37-5827-1	29	27	206	8.1	None	1.0	4.47		3245
37-5037-1	40	28	198	7.8	Unknown	1.1	4.36		12601
South Carolina									
45-5017-1	77	21	226	8.9	None	2.0	3.33	684	8353
45-5034-1	20	25	211	8.3	None	1.6	3.77	927	10179
45-5035-1	20	25	196	7.7	None	1.2	4.24	759	7647
Tennessee									
none									

Table 4-3. Design thickness comparison for JPCP

	PCA		AASHTO 98		AASHTO 93	
Section	min	max	Min	max	min	max
AL 1-3028	0.85	0.85	0.96	0.96	0.97	0.97
FL 12-3811	0.84	0.95	1.22	1.32	0.96	1.02
FL 12-4057	0.47	0.51	0.88	0.98	0.59	0.65
FL 12-4138	0.81	0.88	1.26	1.30	1.16	1.20
GA 13-3016	0.64	0.78	1.06	1.16	0.86	0.91
GA 13-3017	0.95	1.20	0.98	1.12	1.00	1.05
GA 13-3018	0.90	1.05	1.13	1.25	1.04	1.09
NC 37-3011	0.90	1.10	0.70	1.48	0.83	1.14
NC 37-3044	0.83	1.00	1.46	1.73	1.29	1.35
SC 45-3012	0.65	0.75	1.16	1.44	0.97	1.08
Averages:	0.79	0.91	1.08	1.27	0.97	1.05
Standard Deviation:	0.14	0.19	0.20	0.22	0.18	0.18

For JPCP, the AASHTO 93 designs are thinner than the actual pavement for only three sections, if both minimum and maximum values for k are considered. Only two of these would also have thinner designs under the AASHTO 98 Supplement procedure.

Table 4-4. Design thickness comparison for CRCP

	PCA		AASHTO 98		AASHTO 93	
Section	min	max	Min	max	min	max
AL 1-3998	0.88	0.88	1.23	1.23	1.15	1.15
AL 1-5008	0.84	0.84	1.39	1.39	1.09	1.09
GA 13-5023	0.88	1.00	1.42	1.56	1.30	1.37
MS 28-5006	0.94	0.94	1.07	1.07	1.08	1.08
MS 28-5803	0.94	0.94	1.18	1.18	1.24	1.24
MS 28-5805	1.06	1.06	1.38	1.38	1.38	1.38
NC 37-5826	0.94	0.94	1.39	1.39	1.15	1.15
NC 37-5037	1.13	1.38	1.33	1.36	1.31	1.37
SC 45-5017	0.83	0.89	0.79	1.07	1.03	1.11
SC 45-5034	0.88	1.00	1.13	1.21	1.15	1.23
SC 45-5035	0.94	1.06	0.91	1.22	1.17	1.26
Averages:	0.93	0.99	1.20	1.28	1.19	1.22
Standard Deviation:	0.09	0.14	0.20	0.14	0.10	0.11

Section 5
Conclusions and Recommendations

Pavement Design and Rehabilitation

For the 36 sections studied and the assumptions documented in the body of the report, this research suggests the following conclusions and recommendations for concrete pavement design and rehabilitation in the Southeast United States:

- JPCP with undoweled joints should not be used, particularly for pavements with 5 or more million ESALs of anticipated traffic.
- Since existing JPCP with undoweled joints may have considerable remaining fatigue capacity, dowel retrofit followed by diamond grinding should be considered to restore serviceability.
- Doweled JPCP 229 to 254 mm thick (9 to 10 inches) often carries as much as 10 to 20 million ESALs over 25 or more years.
- The AASHTO 93 procedure appears to produce reasonable designs for doweled JPCP.
- The AASHTO 98 procedure is unnecessarily conservative for doweled JPCP.
- CRCP 196 to 234 mm thick (7.7 to 9.2 inches) often carries as much as 7 to 19 million ESALs over 21 to 30 years. Thus, performance similar to doweled JPCP may be achieved with a 10 to 15 % reduction in pavement thickness. These designs may also have considerable remaining life.
- Both the AASHTO 93 and 98 procedures are unnecessarily conservative for CRCP. Pavements 20 % thinner than required by either AASHTO method have performed very well.
- Major rehabilitation decisions should be based on pavement condition and not age or cumulative traffic. Pavements 25 to 30 years old that have carried 10 to 25 million ESALs are still performing well, and may be capable of carrying much more traffic.

This report documents an in-depth study of the performance of concrete pavements in the southeastern United States. Information from the Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) database was investigated. Analysis of 36 sections in Alabama, Florida, Georgia, Mississippi, and North and South Carolina showed that the majority of these pavements are providing excellent service well beyond their original design lives. This has important implications for new pavement construction, as well as maintenance and rehabilitation of existing pavements.

For new pavement construction, the results of this study suggest that life cycle cost models should assume better performance and longer service life than existing AASHTO predictions for these pavements. Thus, the economic benefits of constructing concrete pavements where heavy traffic is anticipated or long life is desired may be considerable.

The implication for maintenance and rehabilitation of existing pavements is that concrete pavements may have considerably more remaining structural capacity than time in service or traffic applied to the pavement would suggest. For this reason, expensive and time-consuming reconstruction efforts or thick overlays should not be used unless the evaluation of pavement condition indicates it is warranted. If the pavement is in good structural condition, diamond grinding and other rapid, low cost Concrete Pavement Restoration (CPR) alternatives may extend pavement life considerably and improve serviceability.

Effect of Assumptions

The AASHTO 1993 design equation (Equation 3-1) is more sensitive to some variables than others. The assumptions made in this study are reasonable and consistent with the available data. Modulus of subgrade reaction k is one of the more difficult parameters to estimate, but has little effect on the performance prediction. This may be seen in the figures in Appendix A – the two curves are close together even though the higher k values are four times the lower. Overall, the performance prediction is relatively insensitive to material parameters (k, $S'_{c,}$ and E_c). The model is more sensitive to load transfer and drainage coefficients (J and C_d) but these can be estimated accurately with the information from DataPave.

Recommendations for Future Research

The results of this study suggest a number of potential avenues for further work:

- The methodology of this study could easily be extended to other regions, or to the entire United States. This could be used to evaluate the performance of JPCP and CRCP in a variety of climate regimes, and not merely the wet-no freeze regime prevalent in the southeast. These results could provide valuable input for the development of the 2002 AASHTO Pavement Design Guide and related documents.
- Alternatively, it would be possible to pursue this research in greater depth, using nondestructive testing and forensic analysis. Regional and state-specific analyses could be carried out, with core testing for concrete properties and study of falling-weight deflectometer results.
- There are many other topics in the area of pavement performance that could be investigated, using the expertise that UAB has acquired with DataPave 2.0.

Section 6
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Section 7
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Appendix A
Performance Comparison Figures

Performance comparison figures for six sections are provided below. In cases where two AASHTO lines are provided, they represent higher and lower values of k, respectively, and should be considered as the upper and lower limits on a band.

12 - 4057

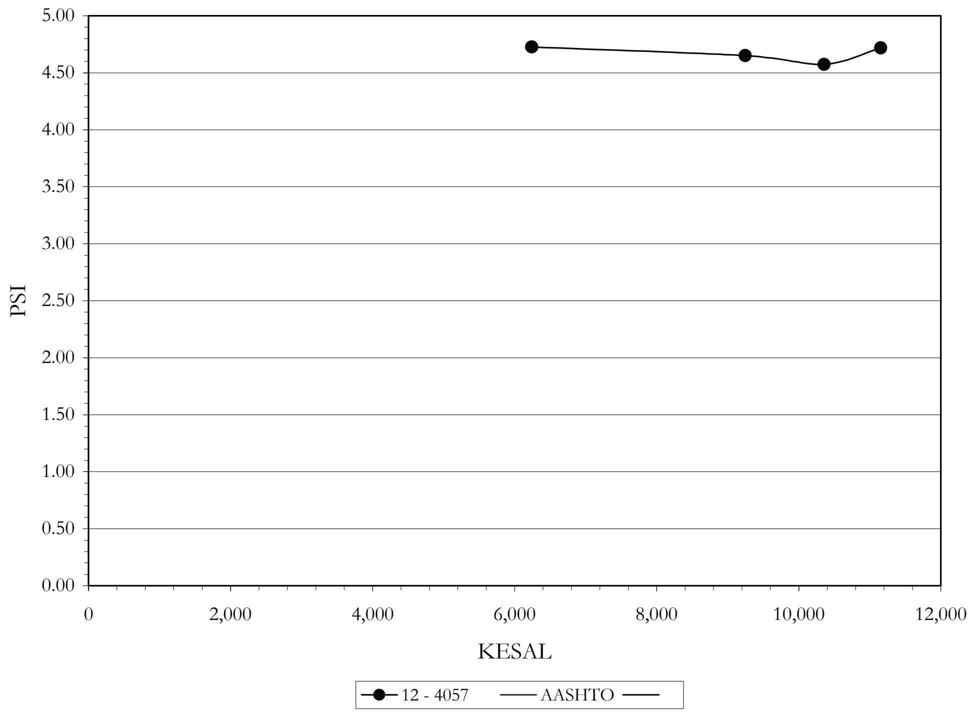

Figure A-1. Section 12-4057, Florida JPCP, doweled joints.

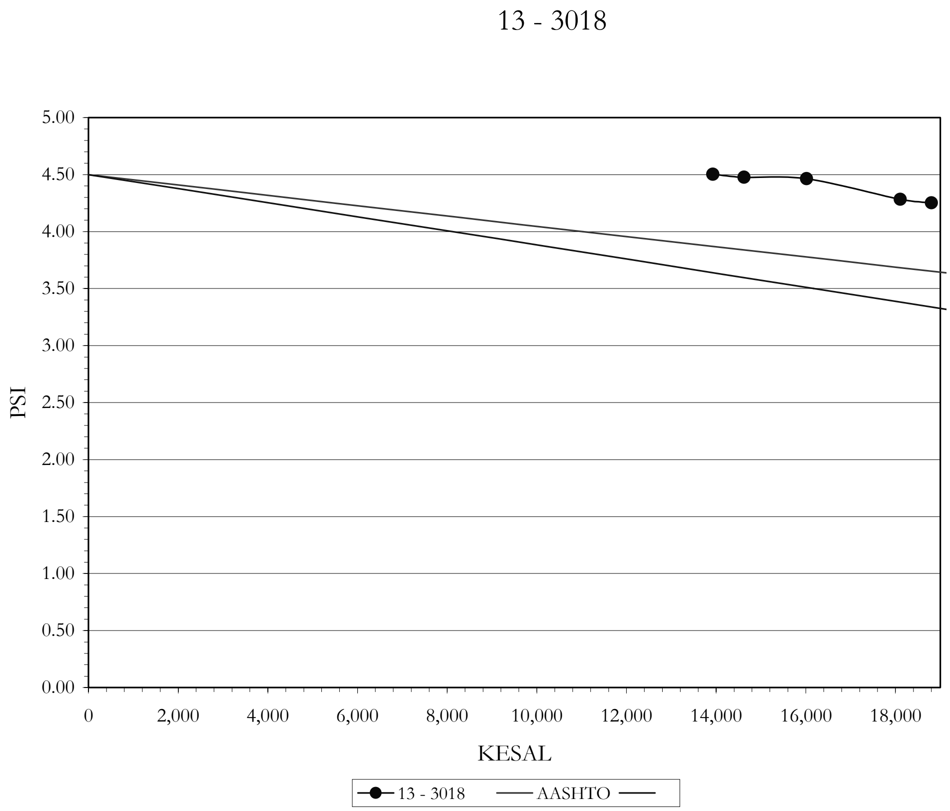

Figure A-2. Section 13-3018, Georgia JPCP, doweled joints.

37-3044

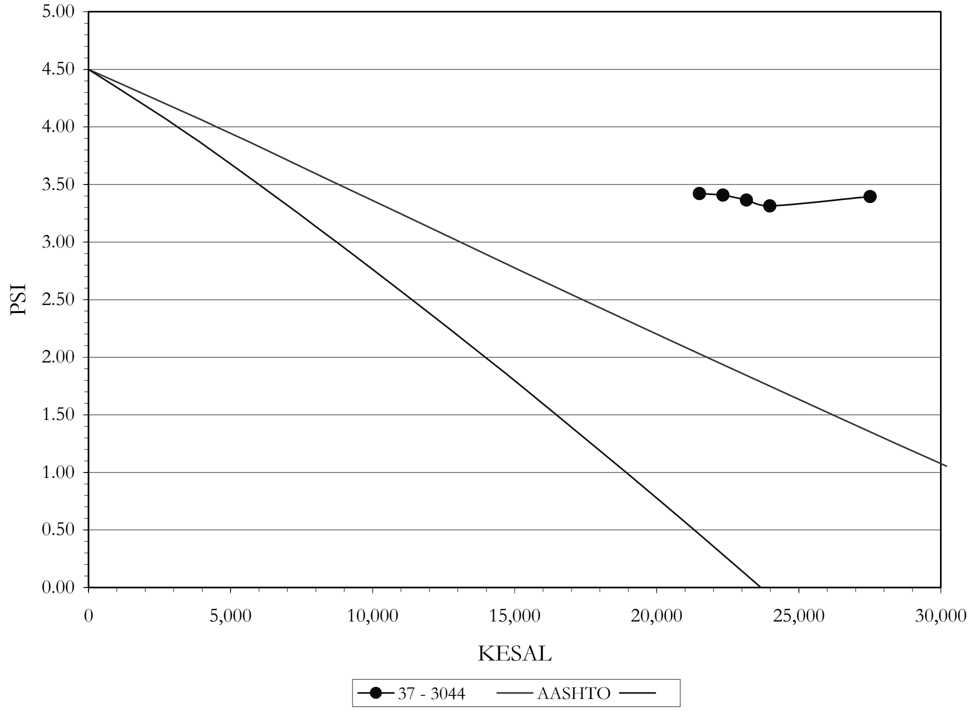

Figure A-3. Section 37-3044, North Carolina JPCP, doweled joints.

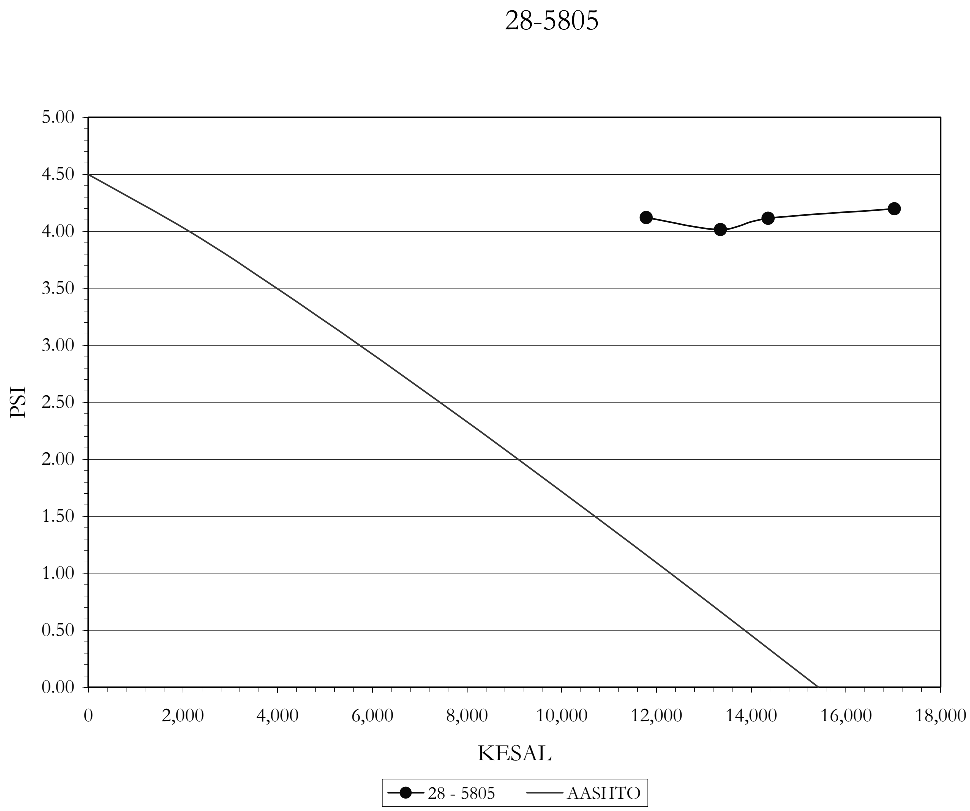

Figure A-4. Section 28-5805, Mississippi CRCP.

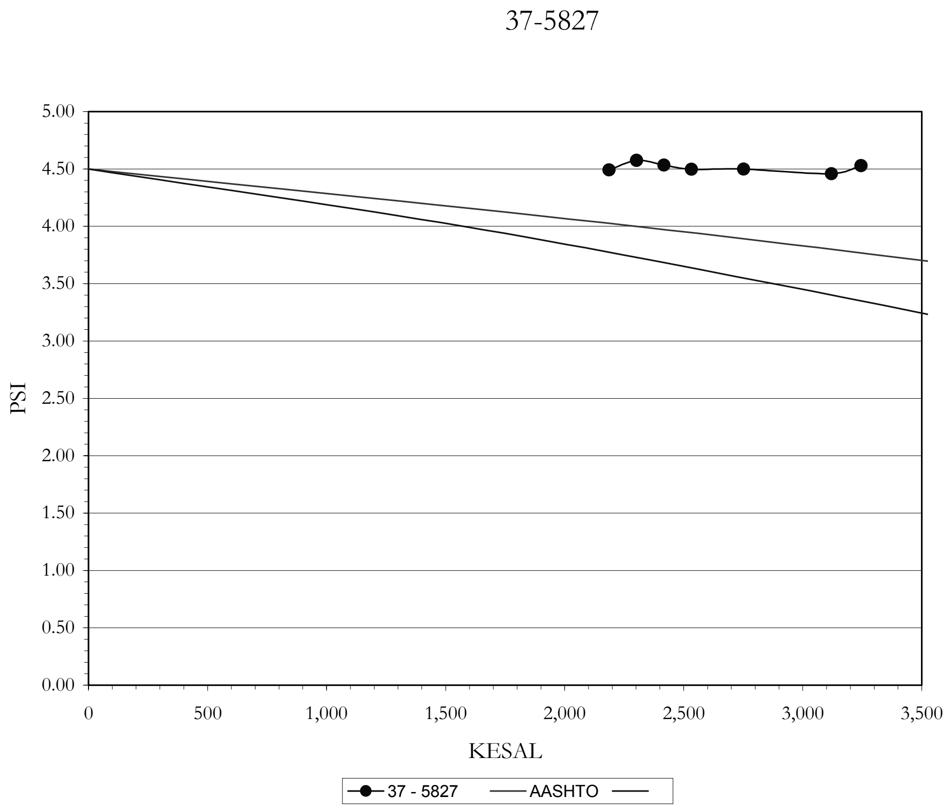

Figure A-5. Section 37-5827, North Carolina CRCP.

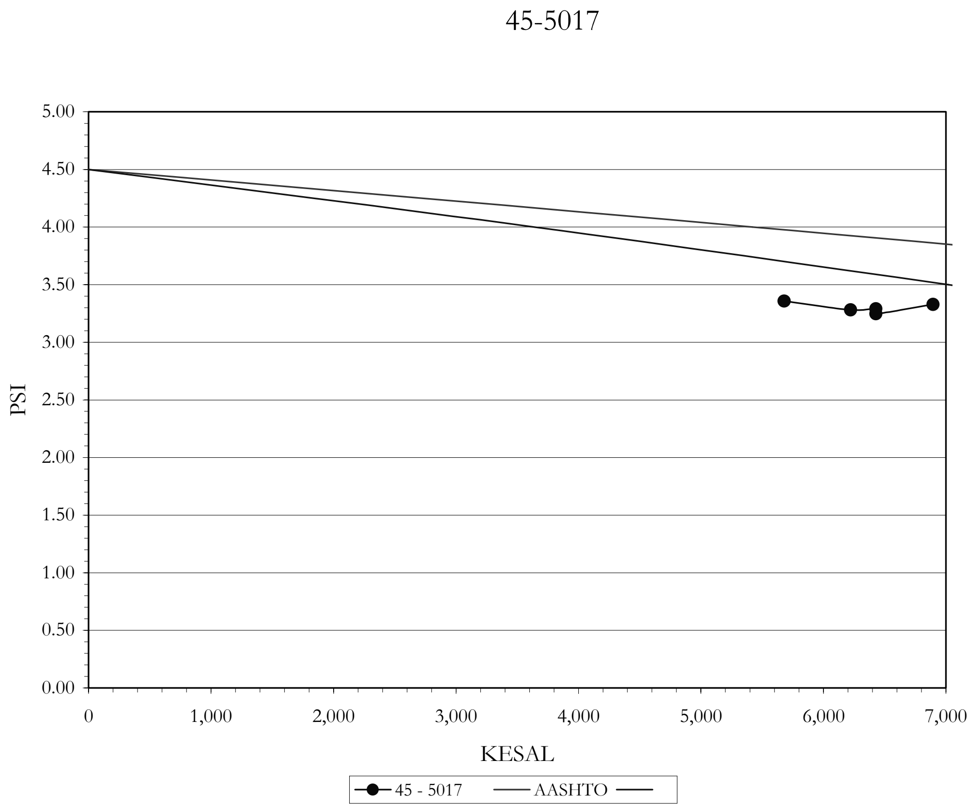

Figure A-6. Section 45-5017, South Carolina CRCP.