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Performance Evaluation of Cement-Treated Roadway Bases

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PERFORMANCE EVALUATION OF CEMENT-TREATED ROADWAY BASES

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Preface

This report describes a study conducted at the University of Wyoming by Dr. Khaled Ksaibati, associate professor of civil engineering, and Melinda Bowen, graduate student of civil engineering. In this study, the researchers evaluated the field performance of cement treated roadway bases.

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TABLE OF CONTENTS

LIST OF TABLES

Chapter 1

INTRODUCTION

Background

Traffic volumes and loads are increasing on America's roads and highways. These increases have caused a need to build stronger pavement structures by increasing the strength of roadway bases. Adding cement or asphalt to bases accomplishes this objective.

The cost of cement began to rise in the 1970s. As a way to continue to treat the base at a lower cost, transportation agencies began to use fly ash as a partial replacement for the cement. R. Pavlovich performed a laboratory study in 1979 on the low level replacement of fly ash in cement treated base (CTB) and Portland cement concrete. Pavlovich determined that CTB with low fly ash contents should exhibit most of the same attributes as other CTB. The cement/fly ash treated bases (CFATB) did exhibit less freeze-thaw resistance, wet-dry resistance, and lower compressive strength. Despite these risks, the Wyoming Highway Department began using CFATB in the reconstruction of roadway sections on the following classes of primary and secondary roads: roads that were expecting significant volumes of heavy vehicles, or roads that demonstrated a tendency to rut under prevailing traffic conditions (Conklin, 1993, pp. 1-2).

A study in the early 1990s, conducted by Khaled Ksaibati and Travis Conklin, considered the limitations of Pavlovich's work. Laboratory testing was conducted and determined that the source of fly ash generally has an effect on the strength, durability, and resilient modulus. This study also determined that higher replacement levels could be used than those determined in the Pavlovich study. This study also considered the field performance of the CFATB bases on primary and secondary roads in Wyoming by using 29 test sections, which ranged from 4 to 13 years in service. Only two of the sections were more than ten years old. Approximately one-third of the sections were in service for only five years.

Problem Statement

Ksaibati and Conklin's research provided preliminary information on the field performance of primary and secondary highways with CTB and CFATB. No information was gathered on the interstate system. Another limitation of Ksaibati's evaluation was using only the Pavement Serviceability Index (PSI) to determine a section's performance. PSI is a function of pavement roughness and it does not incorporate pavement distresses. After considering this limitation, it was determined that a comprehensive field evaluation of CTB should be performed. The basis of this study included two parts: primary and secondary sections with CTB and CFATB and interstate sections with CTB. The study considered the cracking characteristics for all these roadway types. The primary and secondary roadway sections were used to determine the effect of fly ash in CTB on the pavement performance. The interstate roadway sections were used to determine the pavement performance of CTB when compared with other bases.

Objectives

The main objectives of this research include the following:

- Determine if fly ash has an effect on the field performance of cement treated bases.

- Determine if cement-treated bases perform similarly to other base types.
- Develop pavement performance models for interstate and non-interstate roadways in the state of Wyoming.

The primary and secondary roadway evaluation was performed by collecting pavement condition data through the use of Wyoming Department of Transportation videotapes. This data was used to determine if fly ash level had a significant effect on the performance of the sections.

The interstate evaluation was performed by gathering information from the Wyoming Department of Transportation pavement management system. This data was evaluated to determine if CTB, in general, had a significant effect on the overall performance of test sections. This report documents the design of experiment, data collection, data analysis, and conclusions of evaluating the field performance of cement treated base sections.

Report Organization

Chapter 2 of this report is a literature review of cement-treated bases, plant mix bituminous bases, fly ash, cement/fly ash-treated bases, environmental effects of fly ash, and pavement performance. Chapter 3 includes a discussion of the experimental design. Chapter 4 discusses the data collection. Chapter 5 discusses the statistical analysis of the primary, secondary, and interstate roadways. Finally, Chapter 6 summarizes the research performed, presents conclusions, and offers recommendations for further research.

Chapter 2

LITERATURE REVIEW

Introduction

There are approximately four million miles of roads in the United States. Of these, 2.3 million are surfaced with asphalt or concrete, 1.3 million are surfaced with stone, gravel or soil, and 0.4 million miles are non-surfaced. Of the asphalt and concrete surfaced roads, 96 percent are surfaced with asphalt (Roberts *et al.*, 1996, p. 3).

The typical cross-section for an asphalt-surfaced road is composed of the surface course, base, subbase, and subgrade. The subgrade is the foundation layer of the pavement structure. In most cases, this layer is just the natural earth surface, but also can be a compacted soil. The subbase usually is not used unless frost action is severe, the subgrade is weak, or where a construction working platform is needed. It is composed of granular or stabilized material and is placed on top of the subgrade. The base distributes the stresses caused by traffic loads acting on the surface so that little deformation or displacement of the subgrade occurs. The base must have high stability and density. It may be composed of gravel or crushed rock and may be treated with asphalt, cement, fly ash, or lime. The asphalt surface must be able to withstand the wear and abrasive effects of moving vehicles and must be stable enough to resist shoving and rutting. It can range from one to six inches in depth (Wright, 1996, pp. 453-454).

Cement-Treated Bases

Cement-treated base is a soil cement that is used as a road base. It generally is composed of aggregate, approximately 8 percent Portland cement, and water at 1-2 percent below the optimum moisture content (Huntington, 1995, p. 6). Cement-treated base also is referred to as lean concrete and cement bound granular materials. It generally is mixed in a batch plant according to the strength, durability, and uniformity requirements for the layer being constructed. It is then transported to the site, placed on the subgrade, compacted, and overlain with asphalt or Portland cement concrete (Ksaibati, 1995a, p. 1440).

Cement-treated base is used to increase the strength characteristics of the roadway. Soil cement unconfined compression strengths range from about 50 psi to more than 1500 psi. The strengths for CTB generally are in the upper part of this range (Huntington, 1995, p. 7).

CTB has a tendency to induce reflective cracks into the asphalt layer. It is believed that shrinkage cracks form at the surface during the early life (a few days to a few years after construction) of the CTB. These form due to a reduction in volume of the CTB layer as the water evaporates after placement. Fatigue cracks form at the bottom of the base as a result of traffic loads. In both of these cases, the crack propagates through the entire depth of the layer in time ranging from a few weeks to a few years (George, 1990, p. 80). The greatest effect on the creation of reflective cracks comes from temperature cycling cracks, which form early on in the roadway service life because the CTB is placed during the warmth of the day. As air temperature drops at night, small cracks begin to form in the cement-treated base. These small cracks will grow and reflect to the asphalt surface after one or two years (Williams, 1986, pp. 339-348).

When the cracks begin to occur, there is relatively little effect on the riding quality of the pavement. However, the cracks can allow for the start of other problems.

The introduction of water could cause pumping to occur and result in deflection or weakening of the subgrade. This eventually becomes detrimental to the performance and useful life of the pavement structure (George, 1990, p. 81).

The Wyoming Department of Transportation uses strict specifications in the construction of cement treated bases. The gradation of the aggregate must meet specifications listed in Table 2.1 (WYDOT, 1996, p. 558-559).

Table 2.1 WYDOT CTB Aggregate Gradation Specifications

Sieve	% Passing, by Mass
37.5 mm	100
25.0 mm	90-100
4.75 mm	50-70
2.36 mm	40-60
75 μm	4-20

Coarse aggregate used in cement-treated bases must be composed of hard, durable particles or fragments of stone where at least 50 percent of the mass retained on the 4.75-mm sieve must have at least one fractured face. The percentage of wear of the coarse aggregate shall be less than 50 percent, as determined by AASHTO T 96 (Los Angeles Abrasion Test). Also, WYDOT's specifications state that materials cannot be used if they degrade when alternately frozen and thawed, or wetted and dried.

The fine aggregate used in CTB must consist of crushed stone, crushed gravel, or natural sand. The fraction of the aggregate that passes the 425-μm should have a liquid limit less than 25 and a plasticity index less than 6. If this aggregate is nonplastic, the liquid limit must be less than 30.

The aggregate is mixed with cement and water in a central mixing plant. The Portland cement used must conform to the requirements set by ASTM C 150. Type I or Type II Portland cement may be used as a base or subbase treatment (WYDOT, 1996, pp. 550-551). Type I cement is the standard Portland cement most commonly used. Type II cement is a modified Portland cement used when moderate sulfate resistance or moderate heat of hydration is required (Somayaji, 1995, p. 51). After mixing, the percentage of moisture should not vary from optimum by more than 2 percent.

The cement treated mixture must be compacted within 60 minutes from the time mixing was started and within 30 minutes after the material was placed on the roadbed. Before the end of the day that the CTB is finished, a bituminous curing material must be applied (WYDOT, 1996, pp. 181-183).

Plant Mix Bituminous Base

Plant mix bituminous bases (PMBB) are roadway bases treated with asphalt. These also have been referred to as black bases and asphalt-treated bases. PMBB increased in popularity in the 1970s for use under pavements. These bases add stability to the pavement structure. They also can be erosion resistant and aid in the reduction of pumping in concrete pavement (Yoder, 1975, p. 368).

According to specifications used by WYDOT, aggregate for the PMBB must contain coarse and fine aggregates that meet the requirements of grading "W." These requirements are summarized in Table 2.2. The aggregate also must be clean, tough, and durable (WYDOT, 1996, pp. 557-558).

Table 2.2 PMBB Aggregate Gradation Requirements.

Sieve	% Passing, by Mass
37.5 mm	100
25.0 mm	90-100
12.5 mm	60-85
4.75 mm	45-65
2.36 mm	33-53
75 μm	3-12

The coarse aggregate may consist of crushed stone or crushed gravel. This aggregate should have less than 50 percent wear when tested in accordance with the Los Angeles Abrasion Test (AASHTO T96). At least 50 percent of the materials retained on the 4.75-mm sieve should have at least one fractured face. The magnesium sulfate soundness loss (tested by AASHTO T 104) must be less than 12 percent.

Fine aggregate in PMBB may consist of crushed stone, crushed gravel, or natural sand. Aggregate that passes the 425-μm sieve must have a liquid limit less than 25 and a plasticity index less than 3. The exception to this is where the aggregate is nonplastic; the liquid limit must be less than 30 (WYDOT, 1996, p.198, 226-227).

Fly Ash

Fly ash is the finely divided residue that results from combustion of ground or powdered coal and is transported from the combustion chamber by exhaust gases (Ahmed, 1992, p. 6). The fly ash particles generally are spherical and range in size from one to 100 microns (Boles, 1986, p. 4). The ash is removed from the exhaust fumes of the plant by an electrostatic precipitator (Ksaibati, 1995b, p. 19).

The chemical composition of fly ash may vary from one plant to another, and even within a plant. There are three types of coal-burning boilers: stoker-fired furnaces, cyclone furnaces, and pulverized coal furnaces. The stoker-fired furnaces create fly ash that generally is not good for use in highway applications. Cyclone furnaces are not widely used, and are not the best option for use with Portland cement concrete. The pulverized coal furnaces usually produce the best quality fly ash in the largest quantities (Boles, 1986, p. 2).

It was found as early as 1914 that Portland cement concrete could benefit from the addition of fly ash (Ahmed, 1992, p. 7). Fly ash can have pozzolanic reactivity. This relates to the ability of fly ash to form cementitious products at ordinary temperatures when combined with alkali and alkaline earth hydroxides in the presence of water. The alkali and alkaline earth hydroxides usually are provided by adding lime or cement to fly ash. Fly ash also can be a self-hardening material. Fly ash has been used in cement and concrete and as a stabilizing agent (when combined with lime or cement) for soils and aggregates in pavement subgrades, bases, and subbases (Mumtaz, 1990, p. 59).

The quality of fly ash is determined by the fineness, the chemical composition, the loss on ignition, and the uniformity of the ash. Fineness is determined by the percent of material retained on the #325 sieve. The loss on ignition is a measure of the unburned coal remaining in the ash. Uniformity is a measure of the variation in ash characteristics from shipment to shipment (Boles, 1986, p. 8).

Fly ash can be classified in one of two ways. The most common system is based on ASTM specification C-618. The fly ash is classified as "Class F" if the combination of silicone dioxide (SiO_2), aluminum oxide (Al_3O_3), and iron oxide (Fe_2O_3) is a minimum of 70 percent. This combination must be a minimum of 50 percent for an ash to be classified as "Class C." For both classifications, the maximum sulfur trioxide (SO_3) content is 5 percent, the maximum moisture content is 3 percent, the maximum loss on ignition (LOI) is 6 percent, and the maximum fines retained on a #325 sieve is 34 percent. Class F fly ash generally has a low content of lime, therefore, it does not have much cementitious reactivity unless lime is added. Class C fly ash has a higher lime content, therefore, it has cementitious properties and pozzolanic properties (Boles, 1986, p. 5).

The other fly ash classification is based on the type of coal burned. Bituminous coal has a higher carbon content, which causes it to burn more completely and produce less fly ash. Bituminous coal is mainly used in the eastern United States. Subbituminous, or lignite, coal has a higher lime content and is mined in the western United States (Halstead, 1990, pp. 96-97).

The Wyoming Department of Transportation has the following specifications concerning the fly ash used on their projects. The fly ash may be either Class C or Class F. If the ash is being used in conjunction with an aggregate source that is determined to be reactive by the WYDOT Materials Program, the specifications set by ASTM C 618

Table 2A must also be met. These specifications consider the loss on ignition, gradation, drying shrinkage, uniformity, the effectiveness in controlling alkali-silica reaction, and the effectiveness in controlling sulfate resistance.

Cement/Fly Ash-Treated Bases

The most attractive factor for using fly ash in cement-treated base is economic. Fly ash is much less expensive than the other base materials it replaces. Since the vast majority of fly ash is wasted in landfills, the majority of the cost associated with the usage of fly ash is the cost of transportation (Ksaibati, 1995b, p. 19).

When adding fly ash to cement treated bases, several WYDOT specifications concerning replacement ratios must be met. If the fly ash being used contains less than 15 percent calcium oxide (CaO), then 1 kg of cement can be replaced with 1.33 kg of fly ash. If the fly ash being used contains more than 15 percent CaO, then 1 kg of cement can be replaced with 1 kg of fly ash. At most, fly ash can substitute for 20 percent of the total cement required (WYDOT, 1996, p. 179). The cement and fly ash must be added to the mixture in a way that allows for the material to be uniformly distributed throughout the aggregates during the mixing process (WYDOT, 1996, 180).

Environmental Effects of Fly Ash

In 1997, over 900 million short tons of coal were consumed in electric utility plants in the United States. In 1988, this value was more than 750 million short tons (Department of Energy, 1998). The increase in coal consumption also has caused an increase in the production of fly ash. Over 60 million short tons of fly ash were produced in 1997. Only 19 million short tons were put to some sort of use. The remainder was disposed in landfills (ACAA, 1997).

Fly ash also may contain toxic trace elements, such as arsenic, selenium, molybdenum, and cadmium. There have been concerns that these trace elements could leach into the surrounding soils, however research has shown that the leaching from highway purposes is of no danger to the surroundings.

To save landfill space, extensive research has been conducted on the use of coal fly ash as a material for use in highway construction (Schroeder, 1994). The Federal Highway Administration, in response to congressional requirements and incentives, and an EPA guideline has conducted much of this research. All states now allow for fly ash to be used in conjunction with PCC on federal-aid projects (Ormsby, 1990, 52).

The production of portland cement releases carbon dioxide (CO_2) into the atmosphere in large proportions. In fact, there is approximately a 1:1 ratio of CO_2 released to portland cement produced. This CO_2 gas is a major contributor to the greenhouse effect and to global warming. Cement production is expected to increase from 1.4 billion tons in 1995 to 2 billion tons in 2010. Since fly ash can be used to replace some of this cement, using fly ash could cause a significant reduction in cement production and thereby reduce the amount of greenhouse gases (Bilodeau, 2000, p. 41).

Pavement Performance

Several factors affect pavement performance. Generally, performance declines later in the pavement life, as a result of continued traffic loads and the environment. Pavements may begin to show signs of performance decline earlier in their service life due to failure to adhere to asphalt mix specifications, inadequate bases, or improper placement.

The functional performance concerns the riding quality, safety and appearance. The functional performance can be measured by determining surface roughness, surface friction, and rut depth. Structural performance concerns the strength of the pavement layers. This can be measured through visible distress and structural adequacy (USDOT, 1992, 3-2).

The visible distresses that can occur in an asphalt-surfaced pavement include the following: cracking, surface defects, surface deformation, surface disintegration, repair deterioration, construction deficiencies, and loss of support (USDOT, 1992, 3-2). The two most common distress types are rutting and low-temperature cracking, commonly called thermal cracking (Roberts *et al.*, 1991). The visible stress information, in addition to severity and quantity of the distress, can be used to develop the pavement condition index (PCI). The PCI is a numerical rating scale that runs from 0 to 100. A score of 0 indicates a pavement that has failed and a score of 100 indicates an excellent pavement (Shahin, 1981, 18).

Determination of Pavement Condition Index

There are three basic steps used to determine the PCI for a pavement section. First, the pavement is divided into inspection units that have an area of 2500 $ft^2 \pm 1000$ ft^2. Then a condition survey is performed on each inspection unit that determines the type of distress, the severity of distress, and the total number of distresses. The procedures for determining different distress types and their severity are described in depth in Shahin, 1981. Some distresses may be measured in linear feet: longitudinal cracking, transverse cracking, edge cracking, lane/shoulder drop off, etc. Other distresses may be measured in square feet: alligator cracking, bleeding, block cracking, polished aggregate, raveling, etc.

The PCI calculation is then determined by calculating the density for distress. Densities are calculated for each individual distress. For distresses measured in square feet, eqn. 2.1 is used. While, distresses measured by linear feet use eqn 2.2.

$$Density = \frac{distress\_amount\_in\_square\_feet}{sample\_unit\_area\_in\_square\_feet} * 100 \qquad \textbf{(Eqn. 2.1)}$$

$$Density = \frac{distress\_amount\_in\_linear\_feet}{sample\_unit\_area\_in\_square\_feet} * 100 \qquad \textbf{(Eqn. 2.2)}$$

These densities then are used to determine the deduct value by using deduct curves like those found in Shahin, 1981. The sum of all deduct values are subtracted from 100. This value is then the PCI.

An example of the calculation process can be seen in Table 2.3.

Table 2.3 PCI Calculation Procedure

Transverse Crack Length	Section Area	Transverse Crack Density	Deduct Value (from Shahin)	PCI
84 feet	2400 feet^2	(84/2400)*100=3.5 ft/ft^2	10	100-10=90

Longitudinal and Transverse Cracking

Longitudinal and transverse cracks are common in pavements with cement-treated bases. They also are common in areas susceptible to wide variations in temperature.

Transverse cracks extend across the pavement, and run perpendicular to the flow of traffic. These cracks generally are caused in one of two ways. The first is due to thermal situations: shrinkage of the pavement surface due to low temperatures, hardening of the asphalt, or the daily temperature cracking. The second reason can be due to problems below the pavement surface, such as reflective cracks from the base material. These cracks generally are not associated with traffic loads (Shahin, 1981, 111).

Longitudinal cracks run parallel to the pavement's centerline and have three main causes. The first two causes are the same as those for transverse cracks. The last cause is a poorly constructed paving lane joint.

Chapter Summary

There are 2.3 million miles of roadway surfaced with asphalt or concrete in the United States. Many of the roadways contain bases that are treated with either cement or asphalt. Fly ash can be used as a partial replacement for cement in CTB. Wyoming has used this replacement in several locations throughout the state. Analysis should be undertaken to determine performance of these roadway sections.

Chapter 3

DESIGN OF EXPERIMENT

Introduction

This research project used field evaluations to study the performance of cement-treated bases. First, primary and secondary roadways were selected and evaluated to determine if the addition of fly ash to cement-treated bases had a significant effect on the pavement performance. Three percentages of fly ash were considered: 0, 20, and 25 percent. The 0 percent fly ash group acted as a control. Then, interstate sections were selected and evaluated to determine if cement-treated bases performed as well as other base types.

Both evaluations used the Pavement Condition Index (PCI) as a way to rank the pavement performance. Finally, statistical analysis was performed on the data sets to determine if differences in performance among these base types were significant.

Evaluation of Non-Interstate Sections

The field evaluation of roadway bases on the primary and secondary system was conducted to evaluate the performance of existing roadway sections with cement-treated base (CTB) with or without fly ash.

A total of 21 sections were included in the study. The length of all sections totaled 148.694 miles. There were 16 sections on the primary system and five sections on the secondary system. All the sections chosen were constructed with a cement-treated base. Some sections also contained fly ash with a varying percentage of cement replacement. The percentages were grouped into three categories: no fly ash, 20 percent fly ash, and 25 percent fly ash. The percentage of fly ash for each section was obtained by searching the WYDOT Materials Program microfilm files. The group containing no fly ash acted as the control and contained eight test sections. The 20 percent fly ash group was composed of 10 sections, while the 25 percent fly ash group was composed of three sections.

The sections ranged in length from 1.560 miles to 12.218 miles. The average section length was 7.081 miles. For the 0 percent fly ash group, the overall mileage was 45.6 miles, with an average of 5.7 miles per section. Also, seven sections were on the primary system and 1 was on the secondary system. For the 20 percent fly ash group, the overall mileage was 75.877 miles, with an average of 7.588 miles per section. Again, seven sections were on the primary system, but three sections were on the secondary system. The 25 percent fly ash group was composed of 27.217 total miles, or 9.072 miles per section on average. Two of these sections were primary roadways, and one was a secondary roadway. This information is summarized in Table 3.1.

Table 3.1 A Summary of the Test Sections' Characteristics

Characteristic	0% Fly Ash	20% Fly Ash	25% Fly Ash	Totals
Total Mileage	45.600	75.877	27.217	148.694
Average Section Length	5.700	7.588	9.072	7.081
Number of Sections	8	10	3	21
Primary Roadways	7	7	2	16
Secondary Roadways	1	3	1	5

The data was collected using videotapes and computer software used by the Materials Program of the Wyoming Department of Transportation. The data collected was used to determine if adding fly ash to CTBs had a significant effect on the performance of test sections. This process is explained in further detail in Chapter 4. The statistical analysis was performed using two methods: regression analysis and analysis of covariance. The procedure and results are presented in Chapter 5.

Evaluation of Interstate Sections

The field evaluation of roadway bases on the interstate system was conducted to compare the performance of existing roadway sections with and without cement-treated base (CTB).

Test sections were selected from the WYDOT pavement management system. The data published in the *WYDOT PAVEMENT MANAGEMENT PROJECT LEVEL*

SUMMARY for 1999 was used. This data included information on the following pavement parameters:

- System: Interstate, Primary, Secondary, or Other Roadway.
- Route: The route number designation given by the Wyoming Department of Transportation.
- Mile post: The location of the roadway section based on mileage.
- District: The WYDOT highway district.
- Section name: A name given to the section based on its location or surrounding landmarks.
- Pavement type: The surface type, for example asphalt or concrete.
- Pavement thickness: The thickness of the surface layer in inches.
- Base type: The material used in the construction of the base, for example CTB or PMBB.
- Base thickness: The thickness of the roadway base layer in inches.
- Annual average daily traffic: The average number of cars expected to travel over that roadway in a 24-hour period.
- Equivalent single axle load: A measure of the traffic loading.
- Last rehabilitation date: The year of the last major construction project on the section.
- Pavement serviceability index: A measure of the road's surface roughness, on a scale of 1-5.
- Rut depth: A measure of the depth of rutting.
- Pavement condition index: A measure of the pavement cracking.

- Average transverse joint fault height in inches: A measure of the height difference between slabs on a PCC pavement.
- Average surface friction: A measure of the friction value on the road.

The files were reviewed extensively to first identify appropriate test sections with CTB. Because nearly all the interstate sections with CTB had asphalt surface types (54 of 57 sections), only sections containing asphalt pavement were selected. This allowed for 54 sections with CTB and asphalt surfaces to be included in the study. Sections with other base types were then considered. An analysis of plant mix bituminous bases (PMBB) with asphalt surfaces resulted in 38 sections being included in the study. Only two crushed base sections had asphalt surfaces, which did not allow for adequate data to provide statistically significant results. Therefore, crushed base sections were not included in this study. There also were not enough sections with asphalt treated permeable base, untreated permeable base, or cracked and seated PCCP base to include in the study.

The test sections varied in length. The lengths for the CTB sections ranged from 1.54 to 15.7 miles with an average of 7.61 miles. The lengths for the PMBB sections ranged from 3.76 to 22.9 miles with an average of 6.95 miles. In total, 92 interstate sections were selected. Of these, 38 sections were PMBB with asphalt surfaces and 54 were CTB with asphalt surfaces. Of the CTB sections, 16 were on I-25, 18 were on I-80, and 20 were on I-90. Of the PMBB sections, 18 were on I-25, 15 were on I-80, and five were on I-90. The test section locations are summarized in Table 3.2.

Table 3.2 Summary of Interstate Test Sections

Interstate	Surface Type		Totals
	CTB*	PMBB**	
I-25	16	18	34
I-80	18	15	33
I-90	20	5	25
All	54	38	92

*CTB: Cement Treated Base
**PMBB: Plant Mix Bituminous Base

The WYDOT pavement management system was used for the collection of pavement performance data, as explained in Chapter 4. The full statistical analysis can be found in Chapter 5 and consists of regression analysis and analysis of covariance.

Chapter Summary

This chapter presented the design of experiment for the interstate and non-interstate sections. Information from the WYDOT pavement management system and other files were used to obtain the physical attributes of the test sections. The test sections were then selected. The data was then collected as explained in Chapter 4. The procedure for the statistical analysis and the results obtained can be found in Chapter 5.

Chapter 4

DATA COLLECTION

Non-Interstate Sections

After the non-interstate test sections were selected, detailed information was gathered on each site. The physical characteristics and pavement conditions can be found in Appendices B and C.

Some of the basic information for each section was obtained from the Wyoming Department of Transportation Pavement Management System. This data included the following information for each section: pavement type, pavement thickness, base type, base thickness, average annual daily traffic, equivalent single axle load, and last rehabilitation date.

All the sections studied were asphalt pavement. The pavement thicknesses ranged from 2 to 4 inches. The cement-treated bases thicknesses ranged from 6 to 12 inches. For each percentage group of fly ash, there is at least one section with a 6-inch base and at least one section with a 12-inch base. The average annual daily traffic ranged from 143 to 1235 vehicles. The daily equivalent single axle load values ranged from 10 to 200. The date of the last rehabilitation gives an indication of how long the pavement was in service. The oldest pavement section was constructed in 1988. The most recent section was built in 1995.

Data on the pavement performance was collected through the use of Wyoming Department of Transportation videotapes and equipment. The selection of sample units used the systematic sampling method outlined in the standards summarized in Shahin, 1981. First, each test section was divided into individual sample units by the method summarized by Shahin, 1981. This method suggests using sample units of 2500 ± 1000 ft^2. For this reason, sample units were used that were 12 feet wide (the average lane width) and 200 feet long. The selection of sample units used the systematic sampling method outlined in the standards summarized in Shahin, 1981. The number of sample units selected for each section varied based on the total section length. A sampling interval was then determined by dividing the total number of sample units in each section by the number of sample units to be studied. The first 200-foot sample was chosen randomly. Each 200-foot sample afterward was selected by skipping 200-foot samples in the number of the sampling interval.

Each of these sample units was then cued on a set of four videotapes. The videos were controlled by a computer program that also stored the road and milepost information for every road on the Wyoming State highway system. Four television screens showed the following views: left wheelpath, right wheelpath, forward, and milepost (or side of the road). The videos were run in slow motion. Each time a sign of pavement deterioration showed on the screens, the tapes were stopped, and the severity and type of deformation was recorded. This information was used to determine the cracking data and calculate the Pavement Condition Index for each sample unit. The Pavement Condition Indices for all sample units were averaged to obtain the PCI for each roadway section. This process was continued for each of the 21 sections and repeated to obtain the data for three years. Information was gathered for the 1997, 1995, and 1993 videotapes for each section.

Interstate Sections

After the interstate test sections were selected, detailed information was gathered on each site. The surface thicknesses for the CTB sections ranged between 4 and 11 inches. The base thicknesses for the same sections ranged from 6 to 13 inches with the vast majority of sections (51 of 54) having a 6-inch base. There is variability between the AADT and ESAL values for each interstate. These values are summarized in Table 4.1.

Table 4.1 A Summary of the Physical Characteristics of CTB Sections

Interstate	AADT	Daily ESAL	Range of Surface Thickness	Range of Base Thickness
I-25	2738	581	4-7 inches	6 inches
I-80	4797	2172	4-11 inches	6 inches
I-90	2210	396	4-7 inches	6-13 inches

The surface thicknesses for the PMBB sections ranged between four and 19 inches while the base thicknesses for the same sections ranged from 3 to 12 inches. The average AADT values for all PMBB sections for each road are summarized in Table 4.2.

Table 4.2 The Physical Characteristics of the PMBB Sections

Interstate	AADT	Daily ESAL	Range of Surface Thickness	Range of Base Thickness
I-25	1907	455	4-7 inches	6-12 inches
I-80	4535	2246	4-19 inches	5-6 inches
I-90	2556	494	4-10 inches	6-12 inches

Two factors related to pavement age also were included in this study. First, the number of years each section was in service before WYDOT collected the performance data was determined. Second, the difference in AADT and ESAL values for each road also were considered. After an examination of all test sections, it was determined that the data for sections in service for greater than 15 years was not completely reliable. This resulted in the removal of four CTB sections and one PMBB section.

Current field conditions of the test sections were measured by the Wyoming Department of Transportation or were contracted to a consultant by WYDOT. The pavement condition index was measured over the entire section length. This value is measured on a scale from 1 to 100 with a value of 100 corresponding to a roadway with no surface cracks. The cracks were located and the severity was recorded through the use of video distress analysis. Videotapes were taken of the entire roadway and WYDOT materials personnel viewed the tapes. The videotapes were run at a slower speed and the person viewing the tapes entered all cracks and their severity into the computer. The computer then calculated the PCI for the entire roadway section.

Chapter Summary

This chapter presented the data collection for the non-interstate and interstate sections. For the non-interstate sections, field performance data was collected by using videotapes and equipment provided by the Materials Program at WYDOT. For the interstate sections field performance data was obtained from the WYDOT pavement management system. Statistical analysis of this data is presented fully in Chapter 5.

Chapter 5

DATA ANALYSIS

General Statistical Terminology

Following data collection, statistical analysis was performed on the interstate, primary, and secondary road sections. Analysis was performed using the statistical techniques of regression and analysis of variance. This chapter describes the statistical analysis used to evaluate the field performance data. The analysis was performed using MINITAB for Windows computer program, release 12.1, by Minitab, Inc.

Regression Analysis

Regression analysis is a statistical method that uses the relation between two or more quantitative variables so that one variable can be predicted from the other(s). A mathematical equation is used to approximate this relationship.

Simple linear regression is used to describe the statistical relationship between an independent variable (X) and a dependent variable (Y). The term "simple" refers to the fact that there is only one independent variable. The term "linear" implies that the relationship is a straight line. Non-linearity is addressed in multiple regression. The general equation for a simple linear regression model can be seen in Eqn. 5.1.

$$Y = \beta_0 + \beta_1 X + \varepsilon \qquad \textbf{(Eqn. 5.1)}$$

Where: Y: is the dependent variable

β_0: is the value of the Y-intercept

β_1: is the slope of the equation

X: is the independent variable

ε: is the random error term

In Eqn. 5.1, the error term is assumed to have a normal distribution with a constant variance for all observations of X values.

Multiple linear regression is the extension of simple linear regression where several independent variables can be considered. Non-linearity also can be considered through the use of this model by setting one of the independent variables equal to another variable in the model raised to a given power. The equation for a multiple linear regression model can be seen in Eqn. 5.2.

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_{p-1} X_{p-1} + \varepsilon \qquad \textbf{(Eqn. 5.2)}$$

In Eqn. 5.2, the variables and parameters generally are the same as in Eqn. 5.1. There are more independent variables considered in this model as denoted by the X_{p-1} term.

The coefficient of determination, R^2, becomes important in these regression models. This is the proportion of variability within a data set, which is explained by a regression model. The value of R^2 can vary between zero and one. A value of zero implies that none of the variability in the dependent variable is explained by the independent variable. A value of one implies that all data points fall on the line or plane formed by the prediction equation. The value of R^2 will increase as variables are added to the model, therefore, it is important to determine if the change in R^2 from one model to another model containing additional variables is statistically significant. Often, the R^2 value is reduced by a small amount based on the number of data points and the number of independent variables. This reduced value usually is referred to as adjusted R^2. (Neter, 1996, pp. 1-43 and 217-259)

Analysis of Variance

Analysis of Variance (ANOVA) is a statistical method used to evaluate if the means of several groups of data are statistically equal or unequal. This method compares the means of two or more data groups and variation in the data. Several different types of ANOVA exist, depending on the design of the study being performed.

One-way ANOVA looks at the relationship between the mean of the dependent variable and the categories of the categorical independent variable. Two-way ANOVA looks at the relationship between the mean of the dependent variable and the categories of two categorically independent variables. Analysis of covariance (ANCOVA) may be required when one of the independent variables is quantitative, rather than categorical.

The F-test statistic or the P-value can be used to make the decision about equality of the groups. The null hypothesis used is $H_0: \mu_1 = \mu_2 = \cdots = \mu_g$, where g denotes the number of groups or categories. The alternative hypothesis states that at least two of the means are unequal. When using ANOVA or ANCOVA, a confidence (or significance) level must be determined. For the purposes of this report, a confidence level of 95 percent will be assumed. This results in a significance (α) value of 0.05. A P-value smaller than α or a large F-test statistic value would cause a rejection in the null hypothesis. (Agresti, 1997, pp. 438-526)

Statistical Analysis of CFATB and CTB Field Performance

The objective of the non-interstate statistical analysis was to determine if fly ash had an effect on the Pavement Condition Index (PCI). This analysis was undertaken using several steps. Cement-treated base containing fly ash was designated as CFATB.

Cement-treated base with no additional fly ash was designated as CTB. First, regression analysis was used to determine performance models for PCI, transverse crack density, longitudinal crack density, and distance between cracks for each of the fly ash percentages. Then, these curves were compared using Analysis of Covariance. Finally, overall regression models were developed.

Regression Analysis

Several different sets of regression analysis were performed. The results from each set were used to aid in the development of the next set.

In the first analysis, the dependent variable was PCI, and the independent variable was number of years (age) the pavement was in service at the time the PCI measurement was taken. The regression equation was determined first for the test sections containing 0 percent fly ash, then for those containing 20 percent fly ash, and finally for those sections containing 25 percent fly ash. Multiple regressions were performed simultaneously, and the better of the linear and quadratic model is presented. For the data containing 0 percent fly ash, the estimated prediction model is shown in Eqn. 5.3.

$$PCI = 99.7 - 3.18Age \qquad \textbf{(Eqn. 5.3)}$$

Where: PCI: is the pavement condition index

Age: is the number of years in service

This equation had an R^2 value of 0.802. This model seems reasonable because a new pavement would be expected to have a PCI of 99.7, which is quite close to 100. Also, the PCI is estimated to decrease by 3.18 for each additional year the pavement is in service.

For CTBs with 20 percent fly ash, the resulting prediction model can be seen in Eqn. 5.4.

$$PCI = 99.7 - 3Age + 0.3Age^2 \qquad \textbf{(Eqn. 5.4)}$$

This equation had an R^2 value of 0.800. Again, a new pavement would be expected to have a PCI of 99.7, which is close to 100. The PCI is expected to decrease and eventually increase. The point where the PCI begins to increase is at 5.97 years, and is found in the range of data. The slight increase after year six may be due to a single data point at year four, which is pulling the curve downward. There also is higher variability in this set of data than for the sections containing 0 percent fly ash.

For the case where there is 25 percent fly ash, the resulting prediction equation can be seen in Eqn. 5.5.

$$PCI = 100 - 5.6Age + 0.6Age^2 \qquad \textbf{(Eqn. 5.5)}$$

The R^2 value for this equation was 0.985. This equation produces an expected PCI of 100 when the pavement is new. The PCI decreases as age increases until the age reaches 5.01 years. For this data set, this minimum value is outside the data range, and is of no concern.

The analysis was repeated using transverse crack density as the dependent variable. The resulting regression models can be seen in Table 5.1

Table 5.1 Transverse Crack Density Regression Models

Fly Ash Content	Equation	R^2
0%	Transverse Density=0.15+1.10*Age	0.770
20%	Transverse Density=0.26+1.21*Age-0.11*Age^2	0.766
25%	Transverse Density=0.04+2.77*Age-0.45*Age^2	0.977

In all these equations, the transverse crack density increases as the pavement ages. The 0 percent fly ash equation was linear. The 20 and 25 percent fly ash equations were quadratic.

The analysis was repeated using longitudinal crack density as the dependent variable. The resulting regression models can be seen in Table 5.2

Table 5.2 Longitudinal Crack Density Regression Models

Fly Ash Content	Equation	R^2
0%	Longitudinal Dens.=0.04-0.21*Age+0.16*Age^2	0.692
20%	Longitudinal Dens.=0.21+0.32*Age	0.424
25%	Longitudinal Dens=-0.19+0.60*Age	0.656

In all these models, the longitudinal crack density increases as the pavement ages. However, the R^2 values of the longitudinal cracking models are less than the other models developed in this study. This indicates that factors other than the base type have significant influence on longitudinal cracking.

Finally, the analysis was repeated using the distance between transverse cracks as the dependent variable. The resulting regression models can be seen in Table 5.3.

Table 5.3 Distance Between Cracks Regression Models

Fly Ash Content	Equation	R^2
0%	$Distance=195-98.6*Age+14.3*Age^2$	0.906
20%	$Distance=185-68.6*Age+6.85*Age^2$	0.862
25%	$Distance=191-130.9*Age+22.7*Age^2$	0.928

In all these equations, the distance between cracks decreases as pavement ages. All equations were quadratic and decrease and then increase within the data range.

ANCOVA for Comparing Fly Ash Percentage

Analysis of Covariance was performed to compare the means of data from the 0, 20, and 25 percent fly ash curves presented earlier. This method assumes that the slopes of each fly ash group are equal with respect to age. This method was selected because age is a qualitative variable related to the dependent variable. The objective of this section was to determine if the percentage of fly ash has a significant effect on the PCI.

The ANCOVA was performed using the General Linear Model in MINITAB statistical software. The resulting P-value for the percentage of fly ash was 0.244. This results in a failure to reject the null hypothesis of equal means. This implies that the performance of the CFATB and the CTB were equivalent. The P-value for age was 0.000, which means that age does have an effect on PCI. These results are summarized in Table 5.4. This analysis was repeated for transverse crack density, longitudinal crack density, and the distance between cracks. The P-values for fly ash percentage and age for each of these analyses can be seen in Table 5.4.

Table 5.4 ANCOVA Results for Fly Ash Percentage and Age for All Dependent Variables.

Dependent Variable	Factor Considered	P-value
Pavement Condition Index	Fly Ash Percentage	0.244
	Age	0.000
Transverse Crack Density	Fly Ash Percentage	0.057
	Age	0.000
Longitudinal Crack Density	Fly Ash Percentage	0.492
	Age	0.000
Distance Between Cracks	Fly Ash Percentage	0.157
	Age	0.000

In each of these analyses, the percentage of fly ash P-value was greater than 0.05. This causes a failure to reject the null hypothesis of equal means. This means that fly ash percentage does not have an effect on PCI, transverse crack density, longitudinal crack density, or distance between cracks. Similarly, the P-value for age was 0.000 in each of the analyses. This causes a rejection of the null hypothesis of no effect for age. This indicates that age has a significant effect on PCI, transverse crack density, longitudinal crack density, and the distance between cracks.

Overall Equation

The ANCOVA and regression analysis results both were considered and a final regression model was developed for the use of making predictions for Wyoming non-interstate roadway sections. The estimated pooled regression models produced for the PCI can be seen in Eqn. 5.6.

$$PCI = 99.6 - 4.1Age + 0.4Age^2 \quad \textbf{(Eqn. 5.6)}$$

Where: PCI: Pavement Condition Index

Age: number of years in service

The R^2 value for this model was 0.782. This means that 78.2 percent of the variability in PCI can be explained through the use of this model.

Similarly, overall regression models were produced for transverse crack density, longitudinal crack density, and the distance between cracks. These models and their R^2 values can be seen in Table 5.5.

Table 5.5 Final Regression Models for Non-Interstate Sections

Dependent Variable	Model	R^2
PCI	$PCI=99.64-4.10*Age+0.36*Age^2$	0.782
Transverse Crack Density	Transverse Density= $0.18+1.48*Age-0.14*Age^2$	0.753
Longitudinal Crack Density	Longitudinal Density= 0.03+0.36*Age	0.495
Distance Between Cracks	Dist. Between Cracks= $186.2-74.47*Age+7.81*Age^2$	0.863

Statistical Analysis of Interstate Sections

Since it was determined that fly ash percentage did not have statistically significant effects on non-interstate performance, all CTB sections with and without fly ash were pooled and compared to non-CTB sections. The objective of the interstate analysis was to determine if CTB and PMBB performed the same. The statistical analysis of the field performance of the interstate sections with cement-treated base (CTB) and plant mix bituminous base (PMBB) was undertaken using three main steps. First, regression analysis was used to develop performance curves to predict the pavement condition index (PCI) based on the physical attributes of each test section. Second, results from the regression analysis were used to perform ANCOVA. This method was used to compare the means for CTB and PMBB sections. Finally, the results from the initial regression and the ANCOVA were used to develop an overall model using regression analysis that best represented the data.

Regression Analysis

Several different sets of regression analysis were performed. Results from each set were used to aid in the development of the next set.

In the first analysis, the dependent variable was PCI, and the independent variable was years in service (age). Age was determined based on years the service was in pavement at the time data was collected. For I-80, data was collected in 1999. For I-25 and I-90, data was collected in 1998. The regression model was determined first for the test sections containing CTB then, for those containing PMBB. The CTB regression resulted in a prediction equation as seen in Eqn. 5.7.

$$PCI = 99.1 - 0.86Age \qquad \textbf{(Eqn. 5.7)}$$

Where: PCI: is the pavement condition index

Age: is the number of years in service

The R^2 value in this instance was 0.505. The PMBB regression resulted in the prediction equation as seen in Eqn. 5.8.

$$PCI = 101 - 1.0Age \qquad \textbf{(Eqn. 5.8)}$$

The R^2 value for this equation was 0.579. These R^2 values are fairly low. Due to the spread of data, therefore, it was determined that a quadratic model would be more appropriate.

The quadratic model used PCI as the dependent variable and used years in service (age) and age squared as the independent variables. The prediction equation was determined first for those test sections containing CTB and then for those containing PMBB. The regression for the CTB sections resulted in the prediction equation, Eqn. 5.9.

$$PCI = 101 - 2.04Age + 0.08Age^2 \qquad \textbf{(Eqn. 5.9)}$$

The R^2 value for this equation was 0.594. The regression for the PMBB sections resulted in a prediction model, Eqn. 5.10.

$$PCI = 100 - 0.67Age - 0.03Age^2 \qquad \textbf{(Eqn. 5.10)}$$

The R^2 value for this equation was 0.585. Several concerns were raised from this regression. First, the CTB fitted line decreases and then reaches a point where it increases again. In fact, this minimum point occurs at a value of 12.93 years, which is included within the data range. This means that after the pavement becomes approximately 13 years old, its condition begins to improve. This is nearly impossible, unless rehabilitation occurs. Secondly, MINITAB reported a possible lack of fit of the data for the PMBB model. Thirdly, although the overall regression model was significant, the P-values for the slopes in the PMBB model were not significant. Finally, a large R^2 change (0.089) occurred between the linear model and the quadratic model for CTB, but the change in R^2 of the PMBB model was only (0.006). Due to these problems, it was determined that the traffic loading should be considered.

In Wyoming, the three Interstates have varying traffic conditions. Interstate 80 has the highest volumes and the highest equivalent single axle loads. It is followed by Interstate 25, and finally Interstate 90. The differing conditions were considered by developing separate equations for I-25, I-80, and I-90. This consideration caused the sample sizes then to be considered small samples. The models developed and their R^2 values can be seen in Table 5.6.

Table 5.6 Performance Models for I-25, I-80, and I-90.

Base Type	Interstate	Equation	R^2
CTB	I-25	$PCI=100-2.1*Age+0.08*Age^2$	0.705
PMBB	I-25	$PCI=99-0.74*Age-0.03*Age^2$	0.549
CTB	I-80	$PCI=100-0.76*Age+0.01*Age^2$	0.731
PMBB	I-80	$PCI=101-1.08*Age+0.07*Age^2$	0.269
CTB	I-90	$PCI=100-0.05*Age-0.32*Age^2$	0.905
PMBB	I-90	$PCI=100-0.33*Age-0.03Age^2$	0.829

The lowest R^2 value corresponds to the I-80 PMBB group. Only 26.9 percent of the variation in the PCI can be explained by Age and Age^2. The highest R^2 value corresponds to the I-90 CTB group where 90.5 percent of the variation in the PCI can be explained by Age and Age^2. All the equations have Y-intercept values within ±1.1 of

100. This is logical because a brand new pavement (one that has been in service for zero years) would be expected to have a PCI of 100.

As can be seen from the regression models presented earlier in Table 5.6, some of the curves decrease and then begin to increase within the data range. The I-25 CTB prediction line has a minimum value at 13.2 years. This problem may have resulted due to the large number of sections (4) that have been in service for 15 years. These four data points have a range of 11 PCI points. The other reason for the minimum value occurring at 13.2 years in service is due to an outlying value. This point has a PCI of 79 after only nine years in service. The I-80 PMBB prediction line has a minimum value at 7.34 years. Part of the reason for this could be due to a disproportionate amount of data that is has not been in service for long. Seventeen sections have been in service for 0-5 years, whereas only seven sections have been in service for 6-16 years. Two observations were considered to be outliers and both were on sections that were in service for greater than five years.

These regression results will then be considered in the Analysis of Covariance to determine if base type is significant.

Analysis of Covariance

Analysis of Covariance was used because age is a qualitative variable. The objective of this section was to determine if base type has a significant effect on the PCI. This analysis also was performed in steps.

First, the ANCOVA was performed using only treatment (CTB or PMBB) and PCI. The P-value resulting from this analysis was 0.474. Since the P-value is greater than

0.05, the null hypothesis of equal means is not rejected. This means that CTB and PMBB have equivalent pavement conditions, regardless of age.

Next, the ANCOVA was performed using treatment, age, and PCI. The P-value for treatment resulting from this analysis was 0.398. The null hypothesis also is not rejected in this situation, implying the equivalence of the performance of CTB and PMBB. The P-value for age in this model was 0.000, implying that age does have an effect on PCI.

Then, the ANCOVA was performed using treatment, age, age squared, and PCI. The P-value for treatment in this analysis was 0.185. This also results in a failure to reject the null hypothesis, or in a determination that the performance of CTB and PMBB are equivalent. The P-value for age was 0.000, which means that age does have an effect on PCI. The P-value for age squared was 0.009. Since this is less than 0.05, it can be determined that the squared term also has an effect on PCI. These results are summarized in Table 5.7.

Table 5.7 ANCOVA Results for Treatment, Age, and Age^2.

Factor Considered	P-value
Treatment	0.185
Age	0.000
Age^2	0.009

Finally, ANCOVA was performed using treatment, age, age squared, roadway, and PCI. The P-value for treatment in this analysis was 0.191. This also means that the performance of CTB and PMBB are equivalent. The P-value for age was determined to be 0.000, causing the null hypothesis to be rejected. The age squared term had a P-value of 0.009, which also determines that this term has an effect on the PCI. The roadway term had a P-value of 0.002, which indicates that the interstate route number also has an effect on the PCI. A summary of these results can be seen in Table 5.8.

Table 5.8 ANCOVA Results for Treatment, Age, Age^2, and Road.

Factors Considered	P-Value
Treatment	0.191
Age	0.000
Age^2	0.009
Roadway	0.002

Overall Equation

The ANCOVA and regression analysis results both were considered and a final regression model was developed for predicting the PCI for a Wyoming interstate section with either cement-treated base or plant mix bituminous base.

Since there are three categories of Wyoming interstate, dummy variables were created to represent these cases. The regression model contains two categorical variables to represent the interstate system. The first variable is I25. In the case where the road of interest is on I-25, this variable is given a value of one. If the road of interest is not I-25, then this variable is given a value of zero. The same concept applies for the I80 variable: value of one if I-80 is roadway of interest and zero if not. The situation of I-90 being the roadway of interest is considered within the value of the intercept, so it does not need a separate variable. All that needs to be done to consider I-90 is to set the values of the I25 and I80 variables equal to zero.

The regression equation produced can be seen in Eqn. 5.10.

$$PCI = 99 - 1.48 * Age + 0.0442 * Age^2 - 0.62 * I25 + 2.60 * I80 \qquad \textbf{(Eqn. 5.10)}$$

Where: PCI: is the pavement condition index

Age: is the number of years in service

I25: has a value of 1 if on I-25 and 0 if otherwise

I80: has a value of 1 if on I-80 and 0 if otherwise

The R^2 value for this model is 0.623. This means that 62.3 percent of the variability in PCI can be explained through the use of this model. It should be noted that for pavements of the same age on different interstate systems, I-25 would have the lowest PCI and I-80 the highest. Interstate 90 falls between these values. These differences can be attributed to the variations in initial construction thicknesses of the surface and base layers, and also to the level of maintenance on each interstate.

Chapter Summary

This chapter described the statistical techniques followed in the analysis of data for Interstate, Primary, and Secondary roadways. Analysis of Covariance and regression analysis were used.

Analysis of covariance was conducted to determine which factors explained the variability in the data. Regression analysis was used to fit the data to a mathematical curve presented by an equation.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

Summary

This research project used field evaluations to study the performance of cement-treated bases. First, primary and secondary roadways were evaluated to determine if the addition of fly ash to cement-treated bases had a significant effect on pavement performance. Three percentages of fly ash replacement were considered: 0, 20, and 25 percent, where the 0 percent fly ash group acted as a control. Secondly, interstate sections were evaluated to determine if cement-treated bases performed as well as other base types. Only plant mix bituminous bases were used as a comparison due to a lack of suitable test sections for other base types on the interstate system in Wyoming.

Both of these evaluations used the Pavement Condition Index (PCI) as a way to rate the pavement performance. Also, statistical analysis was performed on both data sets to determine if the differences in performance among the bases was statistically significant. The following sections summarize the findings of the analysis performed in this study.

Conclusions from the Field Performance of Bases on the Primary and Secondary Roadway System

Based on the evaluation of the primary and secondary test sections, the following conclusions were drawn:

1. The partial replacement of cement with fly ash in cement-treated bases caused no significant change in the pavement condition index, transverse crack density, longitudinal crack density, or the distance between transverse cracks. Therefore, fly ash should be used more often to replace cement in CTB construction.
2. Pavement age does have an effect on the pavement condition index, transverse crack density, longitudinal crack density, and the distance between transverse cracks. This reinforces the concept that pavement performance declines later in the pavement life, as a result of repeated traffic loads and the environment.
3. Pavement performance on non-interstate roadways in Wyoming can be modeled using the equations presented in Table 5.5. These models may be used to predict when rehabilitation is required on a non-interstate roadway in Wyoming.

Conclusions from the Field Performance of Bases on the Interstate

Based on the evaluation performed on the interstate test sections, the following conclusions can be drawn:

1. Cement-treated bases and plant mix bituminous bases have statistically equal Pavement Condition Indices. Although CTB may induce reflective cracks into the asphalt layers, their overall performance was as good as the performance of PMBB.
2. Age and the square of age (Age^2) do affect the Pavement Condition Index of cement-treated bases and plant mix bituminous bases. This reinforces the concept

that pavement performance declines later in the pavement life, as a result of repeated traffic loads and the environment.

3. The pavement condition index varies among the three interstate systems in Wyoming. These variations may be caused by differences in the initial construction thickness, level of maintenance, and traffic volumes.

4. The pavement performance of interstate roadways in Wyoming can be determined by using the model presented in Equation 5.10. This model may be used to predict when rehabilitation may be required on an interstate roadway in Wyoming.

Recommendations

Based on the conclusions listed above, the following recommendations can be made:

1. An additional field performance evaluation should be performed to determine the effects of high fly ash replacement of cement in CTBs on the Pavement Condition Index of roadways. Such a study may allow incorporating more fly ash than currently specified by WYDOT in CTBs.

2. A field performance should be performed to compare the performance of cement-treated bases to crushed bases on the primary and secondary roadway system.

3. A complete field performance evaluation should be performed to incorporate other factors such as maintenance, equivalent single axle load, pavement

thickness, and base thickness. Such a study would provide more detailed information concerning the effects of these factors on the pavement performance.

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APPENDIX A

PHYSICAL CHARACTERISTICS AND PAVEMENT CONDITIONS OF INTERSTATE SECTIONS

Note: The following abbreviations are used in this appendix.

- System: The roadway system (interstate, primary, secondary). For this data, only interstate sections were considered. "I" designates interstate roadways.

- Route: The route number assigned to the roadway. In Wyoming, the only interstates are numbered 25, 80, and 90. Interstate 25 runs south to north. Interstates 80 and 90 run west to east. Interstate 80 is located in the southern portion of the state. Interstate 90 is located in the northern portion of the state.

- BMP: The beginning milepost of the section.

- EMP: The ending milepost of the section.

- Length: The section length in miles.

- PvThk: The pavement layer thickness in inches.

- BsThk: The base layer thickness in inches.

- AADT: The Average Annual Daily Traffic on the roadway section.

- Daily ESAL: The daily equivalent single axle load on the roadway section.

- PCI: The pavement condition index of the roadway section as determined by the Wyoming Department of Transportation.

- Last Rehab: The date of the last rehabilitation of the roadway section.

- Age: The number of years the roadway was in service prior to PCI measurement. Note: A gray shaded line indicates data that was removed from the study due to a high age.

CTB with Asphalt Surface											
System	Route	BMP	EMP	Length	PvThk	BsThk	AADT	Daily ESAL	PCI	Last Rehab	Age
I	25	17.64	25.86	8.22	7	6	2600	560	96	1993	5
I	25	30.75	39.56	8.81	7	6	2525	553	100	1999	0
I	25	39.56	47.84	8.28	5	6	2505	553	94	1991	7
I	25	58.4	68.97	10.57	7	6	2572	560	87	1988	10
I	25	81.5	94.84	13.34	5	6	2470	586	100	1999	0
I	25	94.84	100.1	5.26	4	6	2545	605	100	1999	0
I	25	100.1	109.1	9	4	6	2535	605	94	1993	5
I	25	141.42	150	8.58	7	6	3145	723	79	1989	9
I	25	150	160.7	10.7	7	6	3130	715	84	1983	15
I	25	160.7	167	6.3	7	6	3040	715	100	1998	0
I	25	167	175.1	8.1	5	6	3405	710	88	1983	15
I	25	175.1	185.4	10.3	5	6	3935	670	94	1983	15
I	25	189	191.9	2.9	7	6	4730	589	88	1987	11
I	25	191.9	196	4.1	7	6	2110	470	92	1994	4
I	25	210.41	219	8.59	7	6	1290	340	83	1983	15
I	25	244	254	10	6	6	1275	335	87	1988	10
I	80	28.1	39	10.9	5	6	4800	2243	100	1997	2
I	80	49	53	4	8	6	4745	2118	91	1983	16
I	80	53	57	4	10	6	4750	2175	94	1993	6
I	80	76	83	7	9	6	5580	2598	98	1996	3
I	80	107.6	120.3	12.7	9	6	5500	2563	99	1993	6
I	80	120.3	130	9.7	8	6	5335	2525	99	1996	3
I	80	171.7	186.6	14.9	10	6	4613	2290	95	1997	2
I	80	186.6	199	12.4	9	6	4645	2323	100	1999	0
I	80	216.2	221.2	5	6	6	4855	2221	90	1974	25
I	80	221.2	227.5	6.3	8	6	4610	2225	100	1997	2
I	80	227.5	233.7	6.2	6	6	4600	2140	92	1983	16
I	80	289.9	295	5.1	5	6	4270	2173	98	1995	4
I	80	295	299.5	4.5	5	6	4270	2173	98	1995	4
I	80	299.5	310.5	11	11	6	4315	2165	97	1991	8
I	80	324	329.6	5.6	5	6	4835	1788	89	1987	12
I	80	329.6	336.6	7	5	6	4830	1788	91	1988	11
I	80	336.6	348.5	11.9	4	6	4885	1791	92	1986	13
I	80	348.5	357.7	9.2	7	6	4900	1795	97	1992	7
I	90	0	9.9	9.9	5	13	1775	470	100	1998	0
I	90	9.9	14.47	4.57	6	13	2745	528	100	1997	1
I	90	14.47	19.96	5.49	4	6	2825	533	86	1991	7
I	90	19.96	21.5	1.54	4	6	2820	533	78	1991	7
I	90	21.5	23.9	2.4	7	10	2705	495	96	1996	2
I	90	45.14	50.75	5.61	4	6	2730	525	88	1991	7
I	90	50.75	56.4	5.65	5	6	2100	523	85	1991	7
I	90	59.4	64	4.6	7	6	2100	298	100	1996	2
I	90	64	69.8	5.8	6	6	2100	310	97	1995	3
I	90	69.8	85.5	15.7	9	6	2100	310	91	1993	5
I	90	85.5	93.2	7.7	6	6	2100	310	96	1994	4
I	90	93.2	100.9	7.7	6	6	2100	310	98	1995	3

I	90	100.9	106.7	5.8	7	6	2100	310	99	1996	2
I	90	106.7	112.5	5.8	6	6	2105	315	100	1997	0
I	90	112.5	118.3	5.8	9	6	2195	335	100	1999	0
I	90	118.3	124.3	6	6	6	2255	345	84	1977	21
I	90	155.1	160.3	5.2	5	6	1790	370	94	1994	4
I	90	160.3	168.5	8.2	6	6	1790	370	99	1996	2
I	90	168.5	177	8.5	6	6	1965	368	100	1998	0
I	90	177	185.7	8.7	5	6	1790	368	97	1995	3

PMBB with Asphalt Surface											
System	Route	BMP	EMP	Length	PvThk	BsThk	AADT	Daily ESAL	PCI	Last Rehab	Age
I	25	17.64	25.86	8.22	7	6	2625	560	100	1980	18
I	25	25.86	30.75	4.89	7	6	2565	553	88	1992	6
I	25	47.84	51.6	3.76	7	6	2525	553	91	1989	9
I	25	51.6	58.4	6.8	7	6	2445	545	92	1986	12
I	25	68.97	75.31	6.34	5	6	2590	545	98	1991	7
I	25	120.82	126.7	5.88	7	8	2585	610	90	1991	7
I	25	126.7	134.9	8.2	7	6	3095	733	82	1984	14
I	25	196	200	4	7	7	1890	470	100	1997	1
I	25	200	205.9	5.92	5	6	1890	470	96	1992	6
I	25	205.92	210.4	4.49	4	12	1890	470	95	1995	3
I	25	219	228	9	9	6	1288	340	100	1998	0
I	25	228	234.9	6.85	7	6	1295	353	82	1986	12
I	25	234.85	244	9.15	5	6	1275	340	89	1987	11
I	25	254	263.7	9.7	5	6	1270	330	88	1991	7
I	25	263.7	271.1	7.44	4	6	1275	330	91	1989	9
I	25	271.14	279.4	8.26	5	6	1275	330	88	1991	7
I	25	279.4	285	5.6	4	6	1270	330	97	1989	9
I	25	285	293.8	8.81	4	6	1270	330	90	1989	9
I	80	39	44	5	6	6	4640	2315	100	1998	1
I	80	44	49	5	6	6	4555	2163	100	1997	2
I	80	57	65.4	8.4	16	6	4740	2163	91	1993	6
I	80	65.4	76	10.6	4	6	5385	2535	99	1995	4
I	80	138	143	5	19	6	4645	2329	100	1994	5
I	80	143	148.5	5.5	6	6	4635	2290	100	1998	1
I	80	148.5	153.8	5.3	6	6	4610	2290	100	1998	1
I	80	153.8	161	7.2	9	6	4595	2290	95	1997	2
I	80	199	210.9	11.9	10	6	4746	2364	100	1998	1
I	80	245.9	240	-5.9	7	6	4230	2130	99	1996	3
I	80	245.9	252	6.1	8	6	4230	2130	100	1999	0
I	80	252	258.6	6.6	10	6	4240	2165	100	1999	0
I	80	275.6	280.7	5.1	6	6	4260	2173	97	1995	4
I	80	280.7	285	4.3	6	6	4260	2173	99	1995	4
I	80	285	289.9	4.9	4	5	4260	2173	98	1989	10
I	90	23.9	28.21	4.31	6	6	2850	505	100	1998	0
I	90	28.21	35.1	6.89	8	6	2655	543	99	1994	4
I	90	35.1	39.8	4.7	4	12	2585	528	93	1988	10
I	90	39.8	45.14	5.34	4	12	2555	503	96	1988	10
I	90	132.2	155.1	22.9	10	6	2135	390	96	1991	7

APPENDIX B

PHYSICAL CHARACTERISTICS OF NON-INTERSTATE ROADWAY SECTIONS

Note: The following abbreviations are used in this appendix.

- Site: The lettered site designation used throughout this study for a given roadway section.

- System: The roadway system (interstate, primary, secondary). For this data, only primary and secondary sections were considered. A primary roadway is indicated by a P, and a secondary roadway is indicated by an S.

- Route: The route number assigned to the roadway. These numbers do not correspond with the numbers used by the public.

- BMP: The beginning milepost of the section.

- EMP: The ending milepost of the section.

- Length: The section length in miles.

- PvThk: The pavement layer thickness in inches.

- BsThk: The base layer thickness in inches.

- FA%: The percentage of fly ash used in the cement-treated base.

- AADT: The Average Annual Daily Traffic on the roadway section.

- Daily ESAL: The daily equivalent single axle load on the roadway section.

- Last Rehab: The date of the last rehabilitation of the roadway section.

Primary and Secondary Site Locations					
Site	System	Route	BMP	EMP	Length
B	P	34	27.577	30.058	2.481
C	P	42	131.793	144.011	12.218
D	P	43	144.011	150.304	6.293
E	P	43	137.837	144.011	6.174
F	P	43	34.000	35.560	1.560
G	P	43	35.560	40.091	4.531
H	P	43	40.640	46.000	5.360
I	S	300	31.898	38.881	6.983
J	P	18	2.468	12.981	10.513
K	P	18	12.981	18.643	5.662
L	P	24	56.039	63.676	7.637
M	P	33	8.178	13.784	5.606
N	P	33	13.784	20.847	7.063
O	P	33	20.847	27.579	6.732
P	P	43	46.000	56.507	10.507
Q	S	302	20.328	27.640	7.312
R	S	607	167.179	174.631	7.452
S	S	1004	2.622	10.015	7.393
V	P	34	21.237	27.577	6.340
W	P	34	30.058	39.225	9.167
X	S	2300	32.001	43.711	11.710

Primary and Secondary Physical Characteristics						
Site	PvThk	BsThk	FA%	AADT	Yearly ESAL	Last Rehab
B	4	12	0	1235	73000	1994
C	3	11	0	343	35770	1992
D	2	12	0	320	36500	1992
E	2	12	0	320	36500	1992
F	2	12	0	785	52925	1995
G	4	6	0	700	50370	1995
H	4	6	0	700	50370	1994
I	4	9	0	235	14600	1993
J	3	12	20	345	27375	1990
K	3	12	20	345	27375	1990
L	4	8	20	300	8395	1990
M	3	10	20	495	21900	1990
N	3	10	20	485	22995	1990
O	4	8	20	490	23725	1990
P	4	6	20	700	50370	1995
Q	2	7	20	205	5475	1992
R	3	8	20	400	15695	1990
S	3	6	20	143	3650	1992
V	4	12	25	1235	73000	1994
W	4	6	25	1118	73000	1994
X	3	10	25	910	44895	1991

APPENDIX C

PAVEMENT CONDITIONS FOR NON-INTERSTATE ROADWAY SECTIONS

Note: The following abbreviations are used in this appendix.

- Site: The lettered site designation used throughout this study for a given roadway section.
- PCI: The pavement condition index of the roadway section as determined by the Wyoming Department of Transportation.
- T Dens: The average transverse cracking density over a given section.
- L Dens: The average longitudinal cracking density over a given section.
- Dist Btwn: The average distance between cracks for a given section.
- Last Rehab: The date of the last rehabilitation of the roadway section.
- The years given are the years the videotapes were made, therefore the year of the PCI calculation.

Note: In this section, first summary tables presenting the averages for each site are presented for all years. Then, tables presenting the data for each 200 foot long section are presented.

Primary and Secondary Pavement Condition												
Site	1997				1995				1993			
	PCI	T Dens	L Dens	Dist Btwn	PCI	T Dens	L Dens	Dist Btwn	PCI	T Dens	L Dens	Dist Btwn
B	87.33	4.58	0.42	24.21	95	2.25	0	40				
C	88.13	4.84	1.18	17.94	98.29	0.96	0	119.05	100	0	0	200
D	86.29	6.79	1.05	14.57	96.43	1.75	0	78.1	100	0	0	200
E	87.5	3.44	2.5	30.78	96.25	1.75	0	60.42	100	0	0	200
F	89.88	3.3	0.13	34.11	98.67	0.89	0	112.96				
G	94.79	2.04	0	63.88								
H	93.44	2.76	0.54	46.48	93.89	2.64	0.67	89.76				
I	94.4	2.95	2.3	35.93	91	3.04	1.26	34				
J	89.55	3.89	1.21	27.05	91	3.84	0.22	27.1	96.18	1.59	0.19	86.21
K	89.6	4.22	1.3	24.45	100	0	0	200				
L	91.4	2.63	1.31	39.38	94.5	2.4	0.07	43.67				
M	86	3.68	3.87	25.86	90.4	2.95	1.46	34.13	93.5	2.6	0.43	40.52
N	87.5	3.97	2.18	26.56	86.75	4.31	2.12	23.89	90.63	3	1.45	35.28
O	87.25	4.44	2.06	21.5	88	2.38	4.02	37.32	91.63	1.91	2.15	53.75
P	93.36	2.11	0.72	56.97	98.64	0.8	0	130.3				
Q	87	4.3	2.38	22.68	86.91	3.66	3.91	28.32	91.8	3.3	0.31	31.41
R	88.11	3.28	2.18	31.38	94.27	2	0.77	46.58				
S	85.5	3.93	2.32	29.12	88.5	3.44	1.73	33.08	94.42	2.24	0.38	55.25
V	88	2.92	1.92	34.84	96.5	1.63	0.04	63.89				
W	88.67	4.67	0.41	28.78	94.17	2.67	0	42.26				
X	86.56	3.89	2.98	15.51	90.44	3.31	1.5	30.24				

1997 SECTION CONDITION DATA

SITE B			
MP	PCI	T Dens	L Dens
27.977	90	3	0
28.586	90	4	0
29.836	82	6.75	1.25

SITE C			
MP	PCI	T Dens	L Dens
131.951	90	5.25	0
132.613	89	3.75	1.79
133.276	89	4.5	0.83
133.939	86	7.75	0
134.602	88	4.25	1.42
135.265	90	4.5	0
135.928	88	3.75	1.58
136.622	85	5	3.83

SITE D			
MP	PCI	T Dens	L Dens
144.09	88	6.25	0
144.703	87	6	0.71
145316	84	5	0
145.929	90	5	4.92
146.542	89	5.5	0
147.155	86	7.25	0
147.768	80	12.5	1.71

SITE E			
MP	PCI	T Dens	L Dens
137.846	89	2.75	2.42
138.618	83	2	1
139.39	88	2.5	3.79
140.162	88	3	3.42
140.934	86	4.25	2.79
141.706	95	2.5	0
142.478	88	3.5	2.75
143.25	83	7	3.83

SITE F			
MP	PCI	T Dens	L Dens
34.17	88	2.5	0
34.322	89.5	3.25	0
34.474	88.25	3	0
34.626	94	2	0
34.777	89.5	2.25	0
34.928	91.5	3	0
35.082	94	2	0
35.233	91	3.75	0
35.385	90	2.5	1.3
35.538	83	8.75	0

SITE G			
MP	PCI	T Dens	L Dens
35.712	92	3.5	0
36.147	93.25	2.5	0
36.583	95.75	2	0
37.017	91.5	3	0
37.454	94	2	0
37.89	99.5	0.5	0
37.973	97.5	0.75	0

SITE H			
MP	PCI	T Dens	L Dens
40.716	91	9	0
41.246	90	2.375	1.417
41.777	95	2	0.58
42.836	98	1	0
43.367	89	2	2.375
43.898	96	2	0
44.428	93	2.5	0.5
44.957	95	1.5	0
45.489	94	2.5	0

SITE I			
MP	PCI	T Dens	L Dens
32.105	79	3.5	1.75
32.8	78	5	5.7
33.499	84	3	0
34.149	90	3.5	0
34.854	84	5	1.25
35.517	83	1	7.75
36.199	87	1	0
36.881	86	4	1
37.567	85	3.5	1.75
38.243	88	0	3.75

SITE J			
MP	PCI	T Dens	L Dens
2.583	92	3.5	0
3.604	90	3.5	1.54
4.627	84	4.25	2.83
5.65	91	3.25	1.29
6.672	89	5.5	0
7.695	86	2.5	4.67
8.718	89	5.75	0
9.74	84	5.25	2.96
10.763	91	4.25	0
11.788	96	2	0
12.809	93	3	0

SITE K			
MP	PCI	T Dens	L Dens
13.17	89	5.5	0
13.701	95	2.75	0
14.231	86	5	2.83
14.761	86	4	3.46
15.292	85	5.75	2.67
15.881	91	3.5	1.17
16.353	90	3.75	1.54
16.883	92	3.75	0
17.413	92	4	0

SITE L			
MP	PCI	T Dens	L Dens
56.116	89	2.5	2.92
56.872	94	2.75	0
57.63	89	2.5	2.54
58.387	94.5	2.5	0
59.144	90.5	3	1.33
59.902	90	3	0.88
60.661	90	2	2.67
61.418	88	3.5	2
62.175	97	1.5	0
62.933	92	3	0.79

SITE M			
MP	PCI	T Dens	L Dens
8.33	88	3.75	0.79
8.859	86	4.25	0.75
9.39	88	4.75	7.17
9.92	81	5	4.17
10.451	84	3.25	6.17
10.98	85	3.25	5.08
11.511	88	3.75	0.46
12.042	86	3.5	5.17
12.572	86	3	5.25
13.102	88	2.25	3.67

SITE N			
MP	PCI	T Dens	L Dens
13.86	94	2.75	0
14.541	89	3.5	0.63
15.224	80	5	6.46
15.904	88	4.5	1.46
16.587	85	5.25	2.29
17.27	88	5	0.75
17.951	86	4	3.29
18.632	90	1.75	2.54

SITE O			
MP	PCI	T Dens	L Dens
21.188	85	6.5	3.17
21.832	92	3	0.42
22.476	86	4.75	2.29
23.12	90	3	2.08
23.765	86	4.25	2.27
25.701	91	3.5	1.21
26.338	82	6.25	2.29
26.983	86	4.25	2.71

SITE P			
MP	PCI	T Dens	L Dens
46.076	97	1.5	0
47.022	90	3	0.375
47.97	96	1	0
48.917	94	2.75	0
49.683	83	4.25	3.96
50.812	97	1.5	0
51.757	97	1.5	0
54.131	93	2.75	0.25
54.599	97	1.5	0
54.955	95	1	1.5
55.546	88	2.5	1.875

SITE Q			
MP	PCI	T Dens	L Dens
20.346	90	4.875	0.375
21.012	87	6.75	0
21.673	85	4.25	2.5
22.336	89	5	0
22.998	90	3.375	1.17
23.66	92	3.5	0
24.324	84	4.75	5.33
24.988	82	4.625	2.58
25.65	84	3.375	6.96
26.312	86	3.25	4.75
26.977	88	3.5	2.46

SITE R			
MP	PCI	T Dens	L Dens
168.713	89	1.5	3.625
1669.375	86	2.875	7.75
170.39	86	4.25	0.96
170.702	91	4	0
171	85.7	3	3.625
171.365	86.5	3.5	3.375
172.028	86.5	5.25	1.375
172.69	88	2.5	2.79
173.353	93	2.875	0.21
173.525	87.5	3.625	2.5
174.019	90	2.75	0.75

SITE S			
MP	PCI	T Dens	L Dens
2.771	81.75	4	0
3.436	84	3	0
4.099	85	5	0.7
4.763	85	2.5	5
5.432	86	2	0
6.083	80	9.5	0.33
6.751	79.5	8.5	0
7.416	80	5	6.5
8.077	92	2.5	0
8.739	86	2	4.6
9.407	86	3	4
10.147	94	3	0
10.848	80	5.5	6.33
11.545	87	4	2.5
12.252	93	3	0
12.954	74.5	2.5	3.8
13.646	92.5	4.5	0
14.354	85	2	6.8
15.054	88	1	5
15.755	84.5	4	4.7
16.453	89	4.5	0.8
17.154	88.25	5.5	0

SITE V			
MP	PCI	T Dens	L Dens
21.616	90	4.25	0
23.357	86	4.25	2.875
24.228	86	3.25	1.67
25.101	85	2.25	3.375
25.974	88	2	2.125
26.843	93	1.5	1.5

SITE W			
MP	PCI	T Dens	L Dens
30.329	87	6.5	0
31.156	86.5	6.75	0
32.066	85	5.5	0
32.975	88.5	5.25	0
33.884	94	2.5	0
34.161	91	1.5	2.46

SITE X			
MP	PCI	T Dens	L Dens
18.761	85	3	5.5
18.818	81	3.5	8.875
18.873	82	4.25	3.21
18.93	86	4.5	1.33
18.987	87	5.25	1.125
19.044	88	3.25	2.96
19.102	93	3	0
19.156	90	4.25	1.29
19.214	87	4	2.5

1995 SECTION CONDITION DATA

SITE B			
MP	PCI	T Dens	L Dens
27.977	94	2.5	0
28.586	96	2	0
29.836	95	2.5	0

SITE C			
MP	PCI	T Dens	L Dens
131.951	98	1	0
132.613	100	0.5	0
133.276	97	1.5	0
134.602	97	1.25	0
135.265	98	1	0
135.928	100	0.5	0
136.622	98	1	0

SITE D			
MP	PCI	T Dens	L Dens
144.09	95	2.25	0
144.703	96	2	0
145.316	93	3	0
145.929	97	1.5	0
146.542	97	1.5	0
147.155	100	0.5	0
147.768	97	1.5	0

SITE E			
MP	PCI	T Dens	L Dens
137.846	97	1.5	0
138.618	96	2	0
139.39	96	2	0
140.162	96	1.75	0
140.934	96	2	0
141.706	96	1.75	0
142.476	97	1.25	0
143.25	96	1.75	0

SITE F			
MP	PCI	T Dens	L Dens
34.17	98	1	0
34.322	100	0.5	0
34.474	100	0.5	0
34.626	97	1.5	0
34.777	98	1	0
34.928	97	1.5	0
35.082	100	0.5	0
35.233	98	1	0
35.385	100	0.5	0

SITE H			
MP	PCI	T Dens	L Dens
40.716	86	6	2.083
41.246	84	4	2.708
41.777	84	8.25	1.25
42.836	100	0.5	0
43.367	98	1	0
43.898	97	1.5	0
44.428	98	1	0
44.957	98	1	0
45.489	100	0.5	0

SITE I			
MP	PCI	T Dens	L Dens
32.105	93	3	0
33.308	89	3	2.21
34.463	92	2	1.55
35.619	93	2.5	0.79
36.774	86	5	2.99
37.928	93	3	0

SITE J			
MP	PCI	T Dens	L Dens
2.583	93	3	0
3.604	93	3.25	0
4.627	88	4	0
5.65	93	3	0
6.672	92	3.5	0
7.695	94	2.75	0
8.718	88	5.75	0.417
9.74	87	4.75	2.042
10.763	90	4.5	0
11.788	91	4.25	0
12.809	92	3.5	0

SITE K			
MP	PCI	T Dens	L Dens
13.17	86	5.75	2.042
13.701	90	4.5	0
14.231	91	3	1.167
14.761	93	3.25	0
15.292	88	2.75	3.542
15.881	88	3.5	2.042
16.353	91	4	0
16.883	87	5.25	1.792
17.413	88	5.5	0.33

SITE L			
MP	PCI	T Dens	L Dens
56.116	96	2	0
56.872	94	2.5	0
57.63	92	3	0
58.387	94	2.5	0
59.144	93	3	0
59.902	96	2	0
60.661	94	2.5	0
61.418	94	2.75	0
62.175	95	2.25	0
62.933	97	1.5	0

SITE M			
MP	PCI	T Dens	L Dens
8.33	89	3	2.042
8.859	92	3	0.375
9.39	89	4	0.917
9.92	91	3	0.875
10.451	87	2.75	3.125
10.98	92	3	0.875
11.511	95	2	0.354
12.042	88	1.75	4.417
12.572	89	3.5	1.583
13.102	92	3.5	0
SITE N			
MP	PCI	T Dens	L Dens
13.86	91	3	1.3
14.541	93	2.5	0.417
15.224	85	6.25	0.833
15.904	88	3.5	1.958
16.587	87	4.5	0.708
17.27	82	5.5	5.167
17.951	80	5.5	4.083
18.632	88	3.75	2.5
SITE O			
MP	PCI	T Dens	L Dens
21.188	94	2.5	0
21.832	89	2.5	2.625
22.476	86	2.25	5.042
23.12	84	1.75	7.54
23.765	88	2	3.833
25.701	82	3.5	7.792
26.338	86	2.25	5.292
26.983	95	2.25	0

SITE P			
MP	PCI	T Dens	L Dens
46.076	97	1.5	0
47.022	98	1	0
47.97	98	1	0
48.917	97	1.5	0
49.683	100	0.5	0
50.812	99	0.75	0
51.757	100	0	0
54.131	98	1	0
54.599	98	1	0
54.955	100	0	0
55.546	100	0.5	0

SITE Q			
MP	PCI	T Dens	L Dens
20.346	90	4.75	0
21.012	80	5.5	9.625
21.673	82	3	8.083
22.336	83	2.25	8.33
22.998	96	1.75	0
23.66	88	3.75	2.208
24.324	89	2.25	2.458
24.998	91	4.25	0
25.65	82	5.5	6.5
26.312	91	4	0
26.977	84	3.25	5.833

SITE R			
MP	PCI	T Dens	L Dens
168.713	94	2.5	0.25
169.375	96	1.5	1.083
170.39	88	3	3.333
170.702	98	1	0
171	93	2	1.25
171.365	96	2	0
172.028	94	2.5	0
172.69	97	1.25	0
173.353	93	2	1.083
173.525	93	2.5	0.792
174.019	95	1.75	0.625

SITE S			
MP	PCI	T Dens	L Dens
2.771	90	4.5	0
3.436	90	3	0.75
4.099	85	5.5	0
4.763	88	2.5	1.458
5.432	92	2.5	0
6.083	85	8	0.875
6.751	87	5	2.083
7.416	92	1.5	0
8.077	90	3.25	0.917
8.739	88	2	0.917
9.407	91	1.75	0
10.147	92	2.5	0
10.848	85	3	5.5
11.545	88	3	2.54
12.252	90	2	2.42
12.954	85	3	5.875
13.646	93	2.5	0.575
14.354	86	5.5	2.458
15.054	90	2.25	2.458
15.755	86	3.25	4.25
16.453	86	5.5	2.458
17.154	88	3.75	2.458

SITE V			
MP	PCI	T Dens	L Dens
21.616	96	1.75	0
23.357	94	2.5	0.25
24.228	97	1.5	0
25.101	98	1	0
25.974	97	1.5	0
26.843	97	1.5	0

SITE W			
MP	PCI	T Dens	L Dens
30.329	91	4	0
31.156	93	3	0
32.066	92	3.5	0
32.975	96	2	0
33.884	96	2	0
34.161	97	1.5	0

SITE X			
MP	PCI	T Dens	L Dens
18.761	88	3.75	1.67
18.818	88	3	2.75
18.873	92	3.5	0
18.93	93	3	0.375
18.987	88	4	1.417
19.044	92	3	0.417
19.102	92	3	0.542
19.156	90	2.5	2.042
19.214	91	4	4.25

1993 SECTION CONDITION DATA

SITE C			
MP	PCI	T Dens	L Dens
131.951	100	0	0
132.613	100	0	0
133.276	100	0	0
133.939	100	0	0
134.602	100	0	0
135.265	100	0	0
135.928	100	0	0
136.622	100	0	0

SITE D			
MP	PCI	T Dens	L Dens
144.09	100	0	0
144.703	100	0	0
145.316	100	0	0
146.542	100	0	0
147.155	100	0	0
147.768	100	0	0

SITE E			
MP	PCI	T Dens	L Dens
137.846	100	0	0
138.618	100	0	0
139.39	100	0	0
140.162	100	0	0
140.934	100	0	0
141.706	100	0	0
142.478	100	0	0
143.25	100	0	0

SITE J			
MP	PCI	T Dens	L Dens
2.583	96	2	0
3.604	98	1	0
4.627	94	2.5	0
5.65	98	1	0
6.672	96	2	0
7.695	96	2	0
8.718	91	4	0
9.74	92	1.5	2.138
10.763	100	0	0
11.788	100	0	0
12.809	97	1.5	0

SITE M			
MP	PCI	T Dens	L Dens
8.33	93	3	0
8.859	92	2.5	1.208
9.39	88	2.75	3.08
9.92	96	1.75	0
10.451	94	2.5	0
10.98	94	2.5	0
11.511	96	2	0
12.042	96	2	0
12.572	94	2.5	0
13.102	92	3.5	0

SITE N			
MP	PCI	T Dens	L Dens
13.86	94	2.5	0
14.541	93	2.5	0.583
15.224	90	4.5	0
15.904	93	2	0
16.587	91	3.75	0.167
17.27	84	4	5.167
17.951	86	2.25	5.65
18.632	94	2.5	0

SITE O			
MP	PCI	T Dens	L Dens
21.188	84	2.25	6.958
21.832	92	2	1.74
22.476	97	1.5	0
23.12	90	1.5	3.167
23.765	96	2	0
25.701	97	1.5	0
26.338	83	2	5.34
26.983	94	2.5	0

SITE Q			
MP	PCI	T Dens	L Dens
20.346	93	3	0
21.012	91	4	0
21.673	96	2	0
22.336	91	4	0
23.66	94	2.5	0
24.324	94	2.5	0
24.988	92	3.5	0
25.65	88	5.75	0
26.312	88	4	1.208
26.977	91	2.5	1.875

SITE S			
MP	PCI	T Dens	L Dens
2.771	97	1.5	0
3.436	97	1.25	0
4.099	96	2	0
4.763	97	1.5	0
5.432	96	2	0
6.083	95	1.75	0
6.751	97	2.25	0
7.416	97	1.25	0
9.407	97	1.5	0
10.147	100	1.5	0
10.848	98	0.5	0
11.545	90	1	0
12.252	92	4.5	1.838
12.954	88	2.5	2.917
14.354	88	4.75	0.792
15.054	92	3.5	0
15.755	96	2	0
16.453	88	4.5	1.167
17.154	93	2.75	0.417

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13. ABSTRACT (Maximum 200 words)

The main objectives of this research included determining if fly ash has an effect on the field performance of cement-treated bases; determining if cement-treated bases perform similarily to other base types; and developing pavement performance models for interstate and non-interstate roadways in the state of Wyoming.

This report documents the design of experiment, data collection, data analysis, and conclusions of evaluating the field performance of cement-treated base sections.

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