UTILIZATION OF BOTTOM ASH IN ASPHALT MIXES

by
Khaled Ksaibati and Jason Stephen
Department of Civil and Architectural Engineering
University of Wyoming
P.O. Box 3295
Laramie, WY 82071-3295

May 1999

Acknowledgment

This report has been prepared with funds provided by PacifiCorp and the U.S. Department of Transportation to the Mountain-Plains Consortium (MPC). The MPC member universities include North Dakota State University, Colorado State University, University of Wyoming, and Utah State University.

Abstract

This research project used field and laboratory evaluations to study the possibility of incorporating bottom ash in asphalt mixes. For the field portion of this research project, a pavement test section was constructed. This test section included control and bottom ash asphalt mixes. The laboratory evaluations involved design and accelerated testing of control and bottom ash asphalt mixes. In addition, the control and bottom ash asphalt mixes used to construct the field test sections were evaluated. Laboratory testing was accomplished by using the Georgia Loaded Wheel Tester (GLWT) and Thermal Stress Restrained Specimen Tester (TSRST). The GLWT was used to evaluate the high-temperature rutting characteristics of asphalt mixes. The TSRST was used to evaluate the low-temperature cracking characteristics of the asphalt mixes.

Initial observations of the test road indicated no difference in performance between the control and bottom ash pavement sections. Laboratory evaluations indicated the various bottom ash asphalt mixes possess significantly different high-temperature rutting and low-temperature cracking characteristics.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

Background

Coal-fired electric power plants consume approximately 900 million tons of coal each year in the United States [DOE 1998]. This consumption results in the production of more than 78 million tons of coal ash [ACAA 1998]. Between 1993 and 1997, the rise in demand for electricity caused coal consumption to increase by an average of 2.9 percent per year [DOE 1998]. This trend is expected to continue and will result in the increase of coal ash production.

Coal ash consists of fly ash and bottom ash. Bottom ash is the heavier ash that falls through the bottom of the furnace where it is collected in a hopper. It is a relatively coarse material and is classified as wet or dry bottom ash depending on the type of boiler used. The lighter fly ash is carried through the furnace with the exhaust gases and is collected by ash precipitators [Huang 1990]. Fly ash accounts for 70 to 80 percent of the coal ash produced by most electric power plants.

Fly ash and bottom ash possess properties that give them several productive uses as construction materials, yet more than 70 percent of the ash remains unused [ACAA 1998]. The majority of unused coal ash is disposed of in landfills or mined out areas of coal mines prior to their reclamation.

Problem Statement

As the consumption of coal by power plants increases, so does the production of coal ash. Disposal of unused coal ash is costly and places a considerable burden on the power industry. In addition, the disposing of ash in landfills contributes to the ongoing problem of diminishing landfill space in the United States. Ash disposal also may pose an environmental hazard.

There are several benefits to finding an alternative use for coal ash. Because fly ash accounts for a larger portion of the total coal ash produced than bottom ash does, considerably more research has been performed exploring its properties and possible uses as a construction material. However, a review of recent research on bottom ash seems to indicate it has the capability to improve asphalt pavement performance when used to replace a portion of the aggregate in asphalt mixes. Given the potential benefits of using bottom ash in asphalt mixes, additional research on the subject is justified.

Wyoming power plants consumed nearly 24 million tons of coal each year [DOE 1998]. This produces a substantial amount of unused bottom ash. The majority of ash is disposed of in mines prior to their reclamation. The successful use of bottom ash in asphalt pavements in Wyoming would provide significant economic savings. Therefore, it is the intent of this research project to determine the feasibility of using bottom ash produced by several Wyoming power plants in asphalt mixes.

Research Objectives

This research project has the following objectives:

- To initiate a field performance study of bottom ash asphalt mixes using a test road containing control and bottom ash pavement sections. This will be accomplished by monitoring the traffic loading and performance of the test sections over several years. The Pavement Conditioning Index (PCI) will be used to determine pavement performance.
- To use laboratory testing to predict susceptibility of bottom ash asphalt mixes to high temperature rutting and low temperature cracking. To satisfy this objective, three different bottom ash mixes and one control mix were evaluated. The primary testing devices used were the Georgia Loaded Wheel Tester (GLWT) and Thermal Stress Restrained Specimen Tester (TSRST).
- To compare the mean laboratory test results of the different bottom ash and control asphalt mixes using a statistical analysis to determine if they displayed a significantly different resistance to rutting and cold temperature cracking.

- To correlate the field and laboratory test results of the asphalt mixes. Several years of field monitoring and periodic follow-up reports will be required to accomplish this objective.

Report Organization

Chapter 2 of this report is a literature review of asphalt pavement materials, bottom ash and its use in asphalt mixes, mix design procedures, and laboratory as well as field performance testing. Chapter 3 includes discussion of the experimental design of this research project and explains the tasks performed. Chapter 4 describes the testing and data collection procedures used during this research project. Collected data also is presented in this chapter. The raw data collected throughout this research project is shown in Appendix A through E. Chapter 5 describes the statistical analysis of laboratory test results. This analysis determined if significant performance differences between the different samples exist. The statistical analysis results are shown in Appendix F. Finally, Chapter 6 summarizes the research performed, presents conclusions, and offers recommendations for further research.

CHAPTER 2

LITERATURE REVIEW

Currently, there is approximately 6.4 million kilometers (4 million miles) of roads in the United States. 3.7 million kilometers (2.3 million miles) of these roads are surfaced with asphalt or concrete while the remaining are surfaced with gravel, stone, or soil or not surfaced at all. Of the hard surfaced roads, 3.5 million kilometers (2.2 million miles) are surfaced with asphalt while only 0.2 million kilometers (0.1 million miles) are surfaced with concrete [Roberts, Kandhal, Brown, Lee, and Kennedy. 1991].

In the United States, approximately 453 billion kilograms (500 million tons) of Hot Mix Asphalt (HMA) are produced and placed each year at a cost of roughly $10.5 billion dollars [Roberts et al. 1991]. Of this material, approximately 93 percent or 421 billion kilograms (465 million tons) is aggregate-related material [Asphalt Institute (AI) 1983]. The large quantity of aggregate used in HMA has put a strain on the supply of high-quality naturally occurring aggregate materials and has directed the attention of some research to searching for more innovative materials [Lovell, Ke, Huang, and Lovell 1991].

A possible alternative aggregate material that may be used in highway construction is coal ash produced by utility power plants. An increasingly large quantity of coal ash is produced each year. Disposing of coal ash as a waste product is costly, puts a strain on limited landfill space, and may pose environmental problems. If productive use of ash becomes more common, this disposal problem may be solved. As an alternative aggregate material, ash also may provide economical savings to utility companies and highway agencies [Lovell et. all 1991].

Asphalt Pavement Materials

Asphalt pavements are constructed using two primary ingredients: aggregate and asphalt cement. The aggregate usually is a combination of course and fine material with mineral filler added as needed. These aggregates generally are obtained from a local pit or quarry. The binder used in asphalt pavements ordinarily is asphalt cement. For an asphalt pavement to withstand current traffic demands, high quality materials must be used. A pavement is only as good as the materials and workmanship that go into its construction [AI 1983].

Pavement engineers are continually looking for ways to improve the performance of asphalt pavements. There are several asphalt additives and aggregates currently being studied to determine if they improve the properties of an asphalt mix.

Aggregate

There are several types of aggregates that can be used in producing HMA. These aggregates ordinarily are classified according to their source. The three main source classifications of aggregate include natural, processed, and synthetic. Natural aggregates are used in their natural form and need no processing. The two most common natural aggregates are sand and gravel. Common sources of natural aggregates are open pits and streambeds. Processed aggregates are natural aggregates or fragments of bedrock that have been crushed and screened prior to their use. Aggregates are processed to make them more suitable for construction by improving their gradation as well as changing their size and shape. Synthetic aggregates are artificial aggregates that do not exist in nature. They are produced as a result of chemical or physical processing. The most common synthetic aggregate is blast-furnace slag. Blast-furnace slag is a nonmetallic substance that rises to the top of molten iron [AI 1983]. Bottom ash from coal burning power plants also is considered a synthetic aggregate. Another source of aggregate that recently has seen an increase in use is reclaimed asphalt pavement (RAP). RAP is used by removing an existing asphalt pavement and reprocessing the material to produce new HMA [AI 1995b].

The performance of an asphalt mix relies heavily on the selection of the appropriate aggregate. When considering an aggregate for potential use in a mix, it is important to determine if it possesses the desired characteristics [American Association of State Highway and Transportation Officials (AASHTO) 1991]. Aggregates used in HMA normally are required to have a hard, strong, and durable structure that is cubical in shape with low porosity. A clean, rough, and hydrophobic surface also is desirable [Roberts et al. 1991].

To determine if a potential aggregate possesses the desired characteristics, several properties of the aggregates can be determined. These properties include: hardness or resistance to wear, durability or resistance to weathering, shape, surface texture, specific gravity and absorption, chemical stability, and freedom from deleterious materials [Wright and Paquette 1987]. Several different tests may be used to determine these properties: the Los Angeles abrasion, sulfate soundness, sand equivalent, deleterious substances, crushed face count, polishing, flat elongated particles, specific gravity, and stripping tests [Roberts et al. 1991].

Another important characteristic of the aggregate selected for use in an asphalt mix is its gradation. If a poor gradation is used, the asphalt mix may not posses the stability or shear strength needed to withstand construction or traffic loading [Peurifoy, Ledbetter, and Schexnayder 1996]. Aggregate normally is grouped into three sizes: course aggregate, fine aggregate, and mineral filler. Course aggregate consists of particles retained on a 4.75 mm (No. 4) sieve, fine aggregates pass through a 4.75 mm sieve, and mineral filler has at least 70 percent passing a 75 μm (No. 200) sieve. The desired gradation can be achieved through combining different proportions of the varying aggregate sizes. The acceptable gradation ranges used by the Wyoming Department of Transportation (WYDOT) are shown in Table 2.1.

Table 2.1 WYDOT Aggregate Gradation Specifications for 19 mm Maximum Aggregate Size [Wyoming Department of Transportation 1996].

Sieve Size	% Passing for 19 mm (3/4 in.) Maximum Aggregate Size	
	Grading A	Grading B
25.0 mm (1 in.)	100	100
19.0 mm (3/4 in.)	90 - 100	90 – 100
12.5 mm (1/2 in.)	60 - 85	-
9.50 mm (3/8 in.)	-	60 – 85
4.75 mm (# 4)	40 - 60	40 – 65
2.36 mm (# 8)	25 - 45	25 – 55
600 μm (# 30)	10 - 30	10 – 30
75 μm (# 200)	2 - 7	2 – 10

Asphalt Cement

Asphalt cement is the binding agent used in HMA. It holds the aggregate together forming a structural framework able to withstand traffic loading. Petroleum based asphalt cement used in HMA is obtained during distillation refining of crude oil.

Asphalt cement is a durable material that displays excellent adhesive and waterproofing properties. It also is highly resistant to reactions with most acids, alkalies, and salts [AI 1995a]. Asphalt cement is viscoelastic, meaning it displays viscose and elastic properties. At room temperature, asphalt cement is a semisolid material displaying elastic behavior. However, once heated it becomes more like a viscous fluid and can easily be mixed with aggregate to make HMA [AI 1995b].

Modified Asphalt Mixes

Asphalt mixes often are modified in an attempt to improve their performance. Modification of an asphalt mix can be accomplished using a variety of additives. Some additives are combined with the asphalt cement or the aggregate prior to mixing. Others are added during the mixing process [Little, Button, White, Ensley, Kim, and Ahmed 1987]. The additive used to modify an asphalt mix depends on the desired performance improvement. Different additives may improve rut resistance, resistance to cold temperature cracking, or reduce stripping. There are several different additives currently being studied to determine if they improve the performance of asphalt mixes. Some of these additives include, rubber, plastic, fiber, oil, lime, portland cement, fly ash, and bottom ash [Miller 1995].

Wyoming Bottom Ash

The bottom ash produced by the Wyodak power plant and the majority of other power plants in Wyoming is considered wet bottom ash. The Wyodak bottom ash exits through the boiler bottom into water-filled hoppers. It is then ground into one-inch minus material and sluiced to a settling pond. This process provides a water wash where the larger particles settle near the edge of the pond. It is this ash that may provide the best highway material.

According to information given to the Wyoming Department of Environmental Quality by Wyodak power plant officials, there are several potential highway-related uses for bottom ash:

- Road Traction Agent
- Road Surface Material
- Hot Mix Asphalt Additive
- Road Base
- Structural Fill

Wyodak officials stated there are 11 states that currently allow bottom ash to be used as a road traction agent. Sixteen additional states also specify the use of Coal Combustion Byproducts (CCBs) for a variety of highway uses. These states are listed in Table 2.2.

Wyoming currently does not use CCBs as a highway material. However, CCBs are exempt from the Wyoming hazardous waste regulations and there are no specific regulations addressing their use. The use of CCBs in concrete and other similar product is allowed and additional uses are handled on a case-by-case basis [Evans 1995].

Table 2.2 States Currently Utilizing Bottom Ash as a Highway Material.

States Allowing Bottom Ash Use as a Road Traction Agent	States That Specify the Use of CCBs in Various Highway Applications
Alaska	Colorado
Arkansas (case by case)	Georgia
Indiana	Illinois
Kentucky	Iowa
New York	Michigan
Ohio	Minnesota
Texas	Missouri
Virginia	Montana
West Virginia	Nebraska
	New Jersey
	New Mexico
	North Dakota
	Oklahoma
	Oregon
	Utah
	Washington

The main environmental concern involving the use of bottom ash as a highway material is air quality. According to the information given to the Wyoming Department of Environmental Quality, bottom ash generates one-half of the PM10 dust (dust smaller than 10 microns in size) as compared to scoria, a commonly used aggregate in Wyoming. This conclusion was based on results from Hardgrove

Grinability testing. The Hardgrove testing procedures are defined in ASTM Standard D409-93a. Hardgrove Grindability test results on Wyodak bottom ash and scoria aggregate are shown in Tables 2.3 and 2.4. The Hardgrove index is based on a "standard coal," which has an index of 100. The higher the index, the easier the material is to grind.

Table 2.3 Hardgrove Grindability Test Results.

Sample	Hardgrove Index		Material	Description
	Test #1	Test #2		
11086-44376	49	52	Bottom Ash	Wyodak Pond
11086-44377	40	--	Scoria	I 90 Exit 132 – Crushed fines by traffic prior to collection
11086-44378	72	--	Scoria	Stetson Drive – Little traffic on this sample
11086-44379	76	74	Scoria	Supplied by City Street Department from stock pile, no salt added

Table 2.4 Sieve Analysis of Material After Hardgrove Testing.

Particle Size	Bottom Ash 11088-44367	Scoria 11086-44379
% > 200 mesh (75 micron)	90.61	81.94
% < 200 mesh, > 325 mesh	6.16	12.06
% < 32 mesh (45 micron)	3.23	6.00

Bottom Ash Use in Asphalt Mixes

Research into the use of coal ash as a construction material largely has been focused on fly ash rather than bottom ash. This is understandable because fly ash accounts for approximately 80 percent of the total coal ash produced. Nonetheless, recent studies have indicated that bottom ash possesses desirable engineering properties that make it a legitimate construction material. This justifies further research into possible uses of bottom ash [Huang 1990].

West Virginia was one of the first states to use bottom ash in asphalt mixes. According to an article published in 1976 by the Asphalt Institute (AI), The West Virginia Department of Highways regards coal ash as aggregate that can be used in asphalt mixes. From 1971 to 1976 West Virginia paved more than 200 miles of low-volume roads with a mixture of bottom ash and emulsified asphalt cement. This mixture was referred to as "ashphalt." West Virginia found that there were several advantages to using "ashphalt." It was found that well-graded bottom ash was easily stockpiled and did not require additional blending prior to its use. In addition, mixing the bottom ash with emulsified asphalt was simple and could be performed on site rather than depending on plant production. The simplicity and flexibility of its use combined with the large supply of ash resulted in a lower cost. Although its production was based on the same guidelines and controls as asphalt concrete, West Virginia found that "ashphalt" effectively helped provided for safer and stronger roads [Root 1976].

According to the Lafarge Corporation, The Texas Department of Transportation also has experimented with asphalt mixes containing bottom ash. The asphalt mixes used in Texas used hot asphalt cement. In 1986, the Texas Department of Transportation Paris District constructed a fourteen-mile pavement test section on Interstate 30. Nine years after its construction, there had been no apparent failures due to mixture characteristics. As result of the apparent success of bottom ash asphalt mixes, the Paris District began developing bottom ash mix design procedures and using them on new projects [Lafarge].

The Lafarge Corporation reports that bottom ash may be used a substitute for field sands in asphalt mixes. Lafarge also states that bottom ash has a large surface area and rough texture that give it excellent adhesion capabilities and improves the skid resistance of pavements. Bottom ash also is a porous material that requires large amounts of asphalt cement. This improves cohesion of the aggregate and provides a more durable pavement. Lafarge finally notes that bottom ash, containing large amounts of mineral iron pyrite, will react with water and may cause deterioration of the pavement surface [Lafarge].

Additional research in Texas, performed by the Texas Transportation Institute at Texas A&M University, involved the use of sulfur-modified bottom ash in asphalt mixes. According to the Texas Transportation Institute, previous research has indicated asphalt mixes containing bottom ash require large amounts of asphalt cement, have low crush resistance, and an increased amount of air voids. Liquid sulfur, when added to the asphalt mixes containing bottom ash, coats the bottom ash and fills the voids on the surface of the particles. This increases the crush resistance of the bottom ash and reduces the required asphalt cement content for the mix. To meet asphalt mix air void and gradation specifications, a minimum of 50 percent crushed stone aggregate was required in the sulfur modified asphalt mixtures [Saylak, Estakhri, and Chimakurthy 1997].

Four tests were used by the Texas Transportation Institute to evaluate the performance of sulfur-modified bottom ash asphalt mixes: resilient modulus, indirect tension, Lottman freeze-thaw durability, and static creep. The test results indicated that sulfur-modified bottom ash asphalt mixes should perform quite well. It also was noted that the low unit weight of sulfur-modified mixes compared to standard mixes would allow 20 to 30 percent more roadway to be constructed per ton of mix. In addition, these mixes maintain approximately the same demand for asphalt cement [Saylak et al. 1997].

Kentucky is another state that has experimented with the use of bottom ash asphalt mixes. In a research project performed by the Kentucky Transportation Center, a one-mile experimental asphalt pavement overlay was placed in 1987. Forty percent of the aggregate used in the asphalt mix for this experiment was bottom ash. The optimum asphalt content was 8.5 percent [Hunsucker 1992].

Hunsucker's research showed that bottom ash aggregate may effectively replace a portion of the aggregate used in asphalt mixes. The combination of bottom ash with other aggregates in asphalt mixes appeared to improve the skid resistance of asphalt pavements. Bottom ash is a porous material and increases the requirement of asphalt cement in mixes by as much as 50 percent. This increases the cost of the asphalt mix. However, the bottom ash is a potential source for high-friction, nonpolishing aggregates for use in asphalt mixes. Hunsucker states that with the success of this experiment, it is possible that the unit price of asphalt mixes containing bottom ash will decrease to the point that will make them an economically viable alternative [Hunsucker 1992].

Bottom ash research performed in Ohio by the American Electric Power Civil and Mining Engineering Division focused on using a combination of boiler slag and bottom ash in asphalt mixes. The research included the evaluation of boiler slag and bottom ash characteristics, asphalt mix designs containing the two coal combustion by-products, and a demonstration project implementing asphalt mixes containing bottom ash and boiler slag in a test road. The aggregate blends in this research contained 70 to 75 percent boiler slag and 25 to 30 percent bottom ash [Amaya 1997].

Results from the research indicated that a combination of boiler slag and bottom ash could be used as an aggregate replacement in asphalt mixes. The materials were found to meet all aggregate quality requirements for aggregate acceptance in Ohio, Kentucky, and West Virginia. It also was found that a well-designed asphalt mix containing boiler slag and bottom ash can be achieved using standard asphalt mix design methodology and performance indicators. After one year of service, the test road containing boiler slag and bottom ash pavement showed no signs of deterioration.

According to Ahmed and Lovell (1992), comprehensive laboratory research has been conducted at Purdue University into the use of bottom ash as a highway material. This research concluded bottom ash has a non-hazardous nature, minimal impact on ground water quality, low radioactivity, and a low potential for erosion. However, it also was found that bottom ash is a potentially corrosive material.

Research into the use of bottom ash in pavement construction also was performed by Ormsby and Fohs (1990). Their research produced several conclusions concerning the use of bottom ash in asphalt mixes. As the ash content in an asphalt mix is increased, the optimum asphalt content increases, the mix density decreases, and the air voids and voids in the mineral aggregate increases. The stability of asphalt mixes decreases as the ash content increases, up to approximately 30 percent. Asphalt mixes containing bottom ash have a lower resilient modulus and approximately the same poisson ratios as standard mixes. The fatigue life and fracture toughness of bottom ash asphalt mixes are higher than standard mixes. In addition, rutting and plastic deformation also are increased. Asphalt mixes containing bottom ash have a substantial resistance to environmental conditioning and damage caused by moisture. Finally, it was concluded that bottom ash could meet the performance specifications of natural aggregate, and successfully be used in asphalt mixes.

Pavement Performance

An asphalt pavement may experience several distresses. Distresses are normal, can be expected to occur toward the end of the design life of a pavement, and are usually the result of repeated traffic loads and the environment. However, when pavements begin to show signs of distress early in their life, a poor asphalt mix, an inadequate base, or improper placement normally is the cause. The main categories of distresses experienced by asphalt pavements include cracking, distortion, disintegration, and loss of skid resistance. Two of the most common specific distresses are rutting and low-temperature cracking, each of which are described in the following two sections [Roberts et al. 1991].

Pavement Rutting

Rutting is the permanent deformation in any of the pavement layers or subgrade caused by the lateral movement or consolidation of pavement materials due to traffic loads [Huang 1993]. Some insignificant rutting occurs in pavements due to continued densification caused by traffic loading after initial compaction. It is common under normal conditions for a 10.16 cm (4.0 in.) asphalt pavement layer compacted to 7 or 8 percent air voids to consolidate under traffic loads to between 4 and 5 percent air voids within in a short period of time. This usually results in rut depths of approximately 0.30 cm (0.12 in.). However, when pavements begin to display a rut depth significantly greater than 0.30 cm (0.12 in.), overstressing, improper densification, or shear failure may be occurring on the surface or in underlying layers [Roberts et al. 1991].

There are several possible causes of excessive asphalt pavement rutting. Insufficient compaction of the asphalt pavement during construction is one. Pavements should be compacted to an air void level between 3 and 8 percent. When compaction produces greater than 8 percent air voids, significant rutting may occur from repeated loading. Compaction of an asphalt mix should be preformed to its more natural air void percentage of approximately 3 to 5 percent [Miller 1995].

Overstressing the subgrade soils also can produce rutting. Overstressing normally results from using inadequate subgrade materials or poor water drainage in and along the pavement section. At high water contents, most soils or granular materials loose strength. Therefore, it is critical to keep the moisture content of subgrade material as low as possible [Roberts et al. 1991].

Using excessive amounts of mineral filler or uncrushed gravel and natural sands in asphalt mixes often will cause pavement rutting. Excessive filler tends to lower the voids in the asphalt mix resulting in lower optimum asphalt contents. The low asphalt content reduces stability of the pavement allowing ruts to develop. Using natural sand or gravel also has a tendency to lower pavement stability. This is because most naturally-occurring aggregates tend to be round or partially round, reducing the friction and interlocking between aggregates. Many states limit the amount of natural sand and require that coarse aggregate be sufficiently crushed prior to being used in an asphalt mix [Roberts et al. 1991].

The largest contributor to pavement rutting is an excessive asphalt content. Too much asphalt cement reduces the fric tion between the aggregate. This causes loads to be carried by the asphalt cement rather than the aggregate structure. Repeated loading under this condition will force the asphalt cement to move out of the aggregate voids resulting in permanent deformation [Miller 1995].

It is important to establish the cause of excessive rutting prior in attempting a repair. A full-depth analysis of the pavement structure should be used for this purpose. To eliminate excessive rutting in new asphalt pavements, it is important to closely monitor material usage and its placement. Insuring the proper pavement density and air voids content is also important [Miller 1995].

Low-Temperature Cracking

Low-temperature cracks are traverse cracks that run perpendicular to the road and often are equally spaced [Roberts et al. 1991]. These cracks form when an asphalt pavement shrinks in cold weather. As the pavement shrinks, tensile stress builds until the tensile strength of the pavement is reached producing traverse cracks. Low-temperature cracks can appear from a single low-temperature occurrence or as a result of repeated temperature cycling [AI 1995b].

Many factors affect low-temperature cracking of asphalt pavements. These factors include asphalt mix properties, pavement age, base layer quality, time and severity of temperatures below freezing, rate of temperature change, and pavement thickness. The most significant factor affecting cold-temperature cracking is the stiffness of the asphalt mixture [Miller 1995]. In general, pavements that are more flexible tend to crack less. However, pavements that are too flexible will tend to have other performance problems.

Aging also plays an important role in the low-temperature cracking of asphalt pavements. Asphalt cement reacts with the oxygen in the atmosphere; a process called oxidation. This process causes the asphalt cement to become more brittle and results in the pavement loosing tensile strength. A considerable amount of aging occurs when an asphalt mix is placed. This is because the asphalt cement exists as a thin film on the aggregate of the loose mix, which is kept at elevated temperatures for an extended amount of time [AI 1995a].

Therefore, the two types of hardening that take place in asphalt pavements are volatilization and physical hardening. Volatilization results from the evaporation of volatile materials from the asphalt cement during mixing and placement of the asphalt mix. Physical hardening occurs when the asphalt cement is exposed to cold temperatures, usually bellow 0^{o}C (32^{o}F), for long periods of time [AI 1995a].

To overcome low-temperature cracking the asphalt cement of a pavement mix must be sufficiently soft and resist aging. The air voids and density of the compacted mix also must be closely monitored so that the asphalt cement does not become excessively oxidized [AI 1995b].

Laboratory Accelerated Testing of HMA

Laboratory accelerated testing devices are used to predict the performance of asphalt mixes prior to their use. If an asphalt mix sample performs well using an accelerated test, it can be expected to display adequate performance in the field. Several accelerated testing devices are used to predict pavement performance. The two devices used in this study are the Georgia Loaded Wheel Tester (GLWT) and the Thermal Stress Restrained Specimen Tester (TSRST). Background information on the devices is given in the following two sections.

Georgia Loaded Wheel Tester

In 1985, the Georgia Department of Transportation (GDOT) began a study on loaded wheel testers. Georgia Tech was contracted by GDOT to develop a loaded wheel tester that would meet their needs. After the preliminary research was conducted, Georgia Tech determined that the original GLWT already used by GDOT could be modified to perform loaded wheel research on asphalt mixes. This original machine was developed by Benedict Slurry Seals, Inc. and was used to design and test slurry seals. The modified GLWT tests asphalt beams 7.62 x 7.62 x 38.1 cm (3 x 3 x 15 in.). These beams can be compacted using a variety of methods [Collins, Shami, and Lai 1995]. The GLWT used at the University of Wyoming is shown in Figure 2.1.

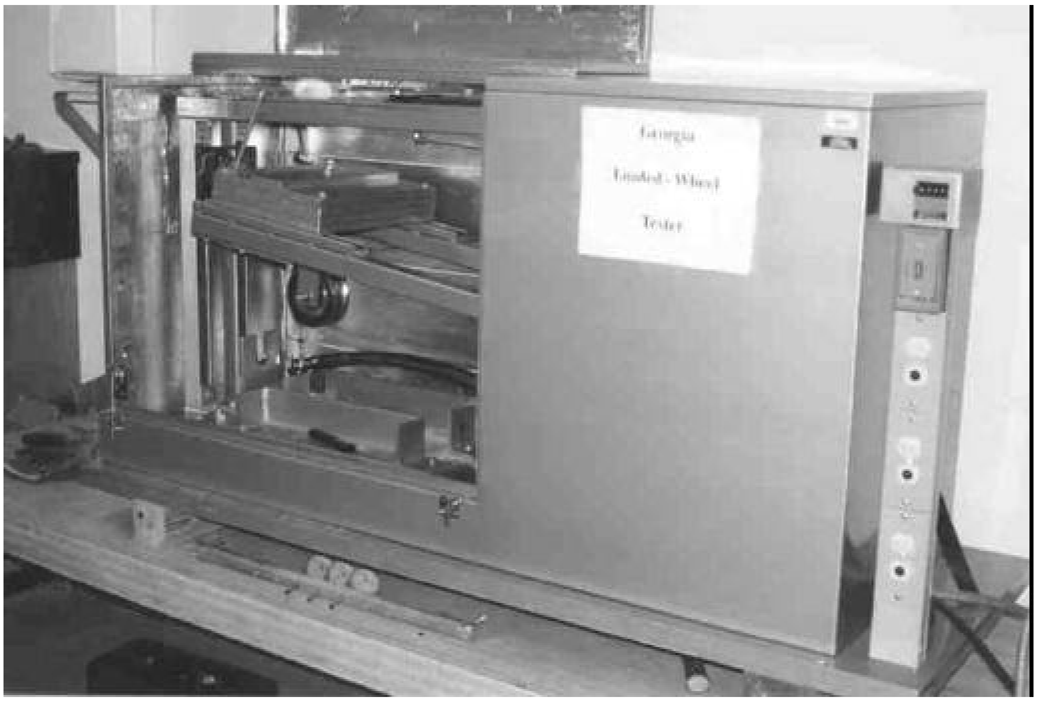

Figure 2.1 Georgia Loaded Wheel Tester.

Researchers from GDOT and other agencies have conducted several studies involving the initial or other modified versions of the GLWT. Results from these studies have shown applicability of the test method for predicting performance of asphalt mixes. A recent study performed by GDOT and Georgia Tech involved the testing of samples prepared using the gyratory compactor. This study showed that the GLWT results for samples compacted using the gyratory compactor correlated well with results for beam samples. This allows for the possibility of using the GLWT in conjunction with Superpave design procedures and tests [Collins et. all 1996], [Miller 1995].

Thermal Stress Restrained Specimen Tester

SHRP initiated research project A-003A to better understand the low temperature cracking of asphalt pavements. It was under this research project that Oregon State University developed the TSRST [Jung and Vinson 1994a]. Since its development, SHRP researchers have performed several different validation studies involving the TSRST. Results from these studies show that the TSRST can be used to successfully predict the low-temperature susceptibility of asphalt mixes [Kanerva, Vinson, and Zeng 1994]. There are several different tests that have been developed over the years to evaluate cold temperature susceptibility of asphalt mixes. However, SHRP researchers have determined that the TSRST is best suited for this purpose because it successfully simulates field conditions, accommodates large stone mixes, and is easy to use [Erickson 1997]. The TSRST manufactured by OEM, Inc and used at the University of Wyoming is shown in Figure 2.2.

Figure 2.2 TSRST Used at the University of Wyoming.

The TSRST evaluates low temperature cracking characteristics of an asphalt mix by slowly cooling an asphalt specimen while restraining it from contracting. This process causes tensile stress to build within the sample. When the tensile stress reaches the tensile strength of the sample, it fractures. The testing equipment consists of a temperature control system, load/displacement system, and a control/data acquisition system. Individual components of the TSRST are shown in Figure 2.3 [OEM 1995].

The TSRST can test a variety of sample sizes. Studies have been conducted on samples with cross sections from 25 mm X 25 mm to 76 mm X 76 mm (1.0 in. X 1.0 in. to 3.0 in. X 3.0 in.) and length to width ratios from 4 to 20. Researchers recommend that the minimum sample cross section should be 51 mm x 51 mm (2 in. X 2 in.) and the length to width ratio should be 5 to 1. A wide range of sample cooling rates also can be accommodated with the TSRST. The most frequently observed field-cooling rate in North America ranges from 0.5^oC to 1.0^oC/hr (0.9^oF to 1.8^oF/hr). To simulate field conditions, researchers should use cooling rates slower than 2.0^oC/hr (3.6^oF/hr). However, this rate results in long test periods. Most researchers conduct tests using a 10.0^oC/hr (18.0^oF/hr) cooling rate and use their results to determine the relative temperature susceptibility of asphalt mixes. Researchers also have determined that cooling rates equal to or greater than 5.0^oC/hr (9.0^oF/hr) have little to no effect on the tensile stress [Jung and Vinson 1994b].

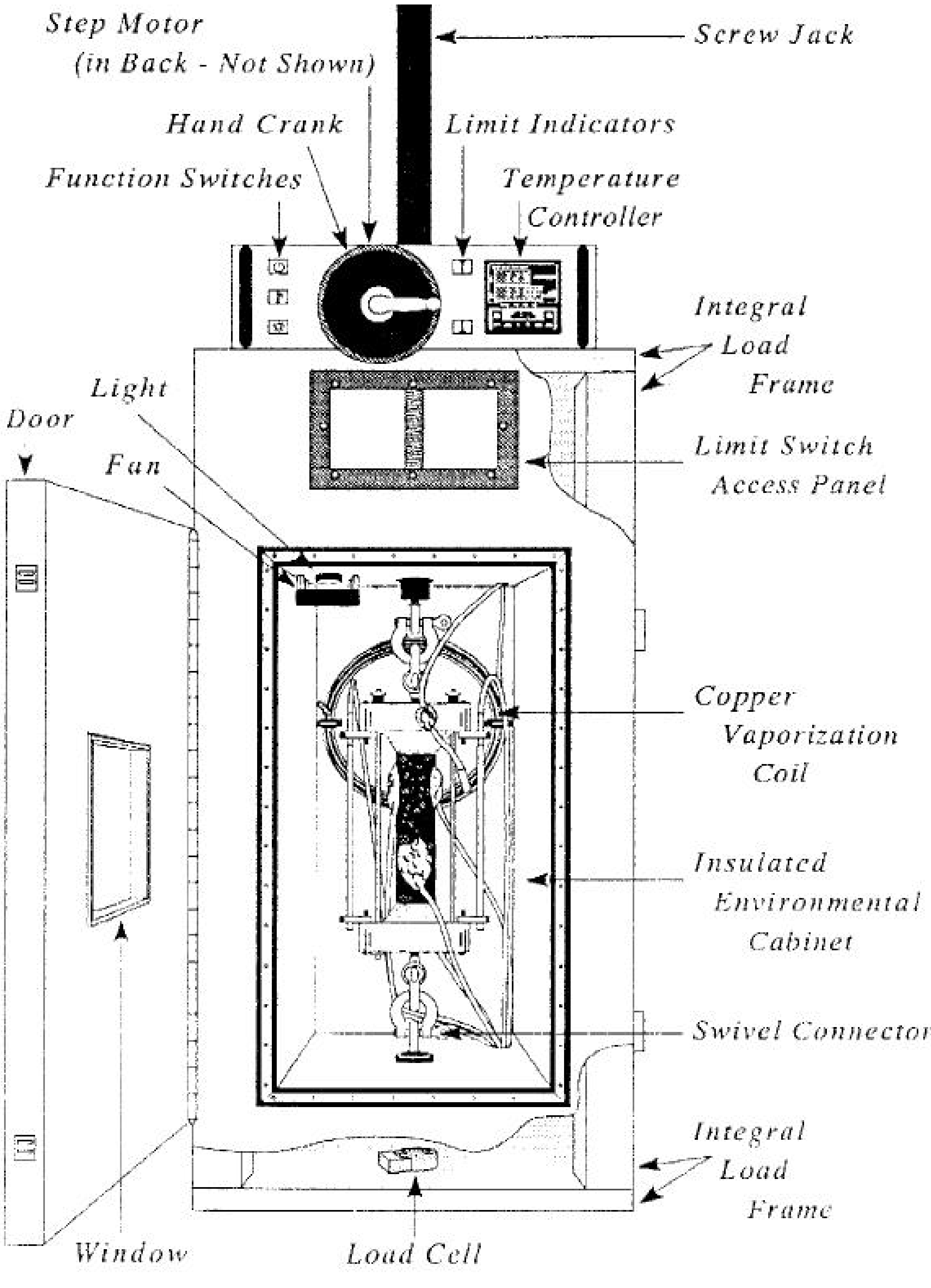

Figure 2.3 Thermal Stress Restrained Specimen Tester Equipment Components [OEM 1995].

Field Evaluation of Pavement Performance

Evaluating field performance of asphalt pavements is important for a number of reasons. When used as part of a pavement management system, pavement performance evaluations can aid in the selection of maintenance projects, maintenance requirements, and specific treatment strategies [Haas, Hudson, and Zaniewski. 1994]. Performance evaluations also can be used to evaluate the asphalt mix used in construction a pavement. When asphalt pavement experiences an unusual amount of distresses early in its life, a poorly performing asphalt mix may be at fault.

Asphalt pavements performance usually is monitored using a pavement distress condition rating system. The Pavement Condition Index (PCI), developed by the U.S. Amy Corps of Engineers, is one such system. Use of PCI has become widespread over the past few years. It has even been adopted as the standard procedure for evaluating the condition of airfield pavements, roads, and parking lots in several locations around the world. The PCI was used to evaluate the field performance of the test road in this research project.

The PCI is a number used to rate a pavement's condition and can range form 0 for failed pavements to 100 for pavements in perfect condition. PCI is determined using the results of a visual condition survey involving different pavement distresses. PCI was developed to display the structural integrity and operational condition of the pavement surface. It also provides an insight as to whether the distresses are related to climate or traffic loading.

Chapter Summary

Bottom ash accounts for a significant portion of the waste produced by power plants, yet, productive use of this material has remained relatively limited. Recent research has indicated that the use of bottom ash in asphalt mixes has the potential to improve performance of asphalt pavements while providing an environmentally and economically sound alternative to ash disposal. Given the large production of bottom ash in Wyoming, this justifies further research into the use of bottom ash as an aggregate replacement in asphalt mixes used by WYDOT and other agencies.

CHAPTER 3

DESIGN OF EXPERIMENT

The objective of this research project was to explore the possibility of using bottom ash in asphalt mixes. To accomplish this objective, a two-part study was performed. Part one of the study involved the field and laboratory evaluation of a control and bottom ash asphalt mix used to construct a test section at the Wyodak power plant. The field performance of the test section was later evaluated using the Pavement Conditioning Index (PCI). The laboratory testing was accomplished using the Georgia Loaded Wheel Tester (GLWT), Thermal Stress Restrained Specimen Tester (TSRST), and Marshal stability and flow tester.

The second part of this study involved laboratory evaluation of bottom ash asphalt mixes designed using the Marshall mix design at the University of Wyoming. The laboratory testing devices used in this part of the study included the GLWT and TSRST. Figure 3.1 summarizes tasks performed in both parts of this study. The remainder of this chapter describes these tasks in detail.

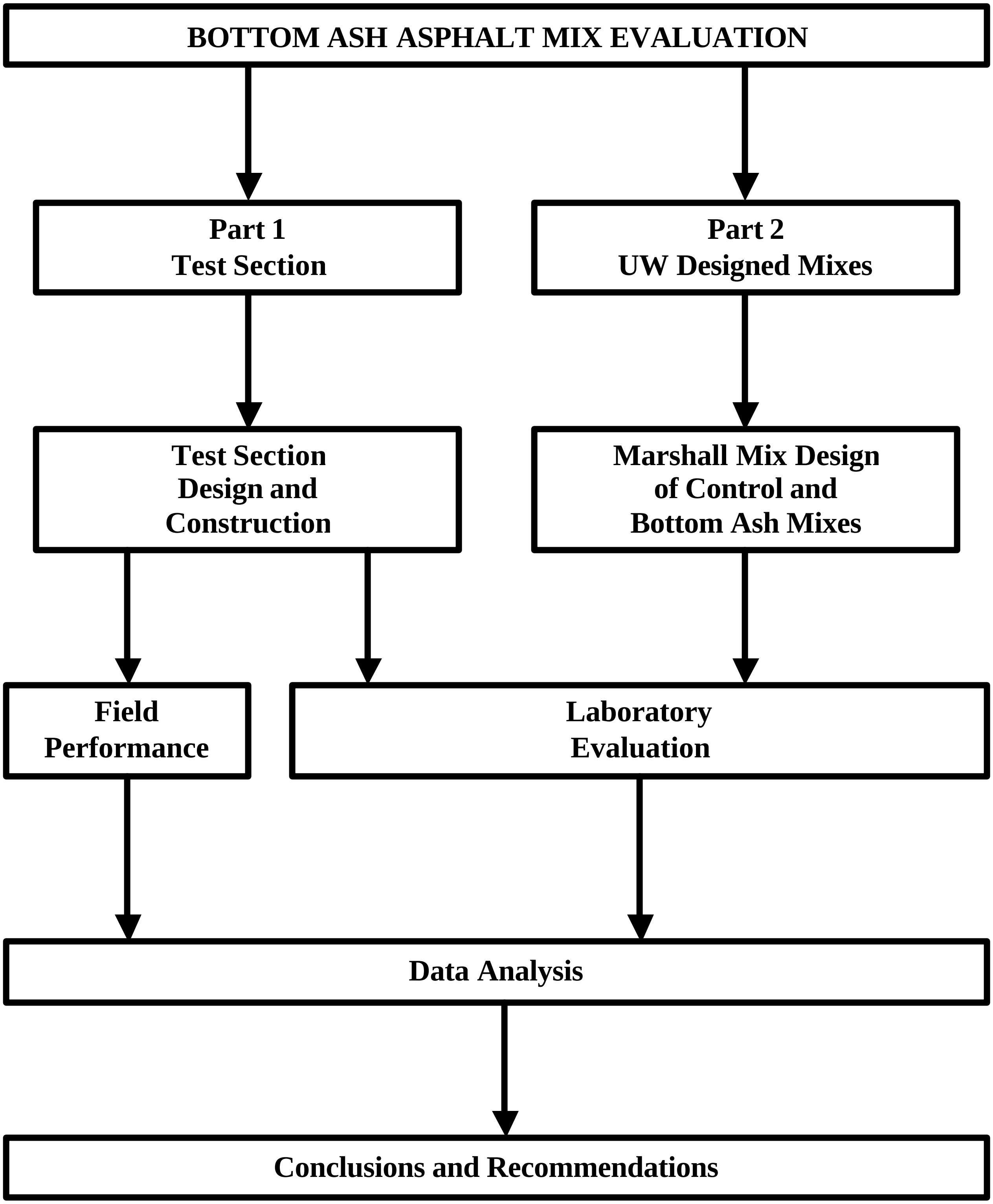

Figure 3.1 Research Project Tasks.

Asphalt Mix Materials

The bottom ash used in this research project was obtained from the Wyodak, Jim Bridger, and Naughton coal-fired power plants. The Wyodak power plant is located in the northeast corner of Wyoming near the town of Gillette. The Jim Bridger power plant is located in the southwest corner of Wyoming near Point of Rocks. Naughton also is located in the Southwest corner of Wyoming, but near Kemmerer. Each power plant burns coal from different mines located near their operation. All three plants are considered "wet bottom" units and thus produce wet bottom ash [PacifiCorp 1998].

Three different collections of mineral aggregate were used in preparing asphalt mixes for this project. They included coarse aggregate, crushed fines, and sand filler. The coarse aggregate and crushed fines were obtained from Pete Lien & Sons in South Dakota. Cundy Asphalt Paving supplied the sand filler.

AC-20 asphalt cement was the binder used in this project to prepare field and laboratory asphalt mixes. It was obtained from the Exxon refinery located in Billings, Mont. This grade of asphalt commonly is used for highway and interstate construction around the region [CE&MT].

Part 1: Test Section Evaluation

In June 1998, a pavement test section was constructed at the Wyodak power plant by Cundy Asphalt Paving of Gillette, Wyo. The purpose of the test section was to evaluate the field performance of a bottom ash road base and bottom ash asphalt mix. However, this research project is primarily concerned with evaluating the performance of the bottom ash asphalt mix.

The test section was incorporated into a route used solely by trucks carrying bottom ash from the Wyodak plant. This allowed the total load endured by the pavement to be precisely calculated. Figure 3.2 shows a typical bottom ash truck carrying bottom ash over the Wyodak test section.

Figure 3.2 Truck Carrying Bottom Ash across the Wyodak Test Section.

According to PacifiCorp, the empty weight of a bottom ash truck is 16,300 kg (36,000 lb.) while the average weight of bottom ash carried on each truck is 23,600 kg (52,000 lb.). Therefore, the average total weight of a loaded bottom ash truck is 39,900 kg (88,000 lb.).

The bottom ash trucks haul approximately 600 loads of bottom ash per month and 7,200 loads per year. Each load requires two trips, one loaded and one unloaded, across the test section. The total weight crossing the test section in both directions is 404,967,000 kg (892,800,000 lb.) per year. This equals approximately 118,000 18 kip Equivalent Axle Loads (EALs).

Test Section Design and Construction

The test section is 91.4-m (300-ft.) long and 10.4 m (34-ft.) wide, 7.3 m (24 ft.) of travelway and 3.1 m (10 ft.) of shoulder, with a supperelevation of 2 percent throughout. The test area consists of four

22.9-m (75-ft.) sections. Each contains a different combination of control and bottom ash base couse and asphalt pavement [CE&MT].

Construction of the test section began by preparing the subgrade, which involved excavation of existing roadway materials and placement of Tensar 1100 SX geosynthetic fabric. During excavation of the subgrade material, live utilities and several other obstacles were encountered. Consequently, the subgrade could not be prepared to the specified depth of 30.5 cm (12 in.) nor could the fabric be used in all areas. A sand subbase was placed on top of the subgrade and compacted. Next, the control base course and bottom ash base course followed by the control asphalt and bottom ash asphalt were placed and compacted to produce four test sections [CE&MT]. Table 3.1 lists the four sections of the test area and materials used in each one. A diagram showing placement of the different materials over the entire length of the test section is presented in Figure 3.3.

Table 3.1 Test Section Construction Materials.

Section # (Stationing)	Pavement Type Thickness	Base Course Type Thickness	Subbase Type Thickness
#1 (0+10 to 0+85)	Control 12.7 cm (5 in.)	Control 19.05 cm (7.5 in)	Sand 62.23 cm (24.5 in.)
#2 (0+85 to 1+60)	Control 12.7 cm (5 in.)	Bottom Ash 17.78 cm (7.0 in.)	Sand 60.96 cm (24.0 in.)
#3 (1+60 to 2+35)	Bottom Ash 15.24 cm (6 in.)	Bottom Ash 17.78 cm (7.0 in.)	Sand 60.96 cm (24.0 in.)
#4 (2+35 to 3+10)	Bottom Ash 15.24 cm (6 in.)	Control 19.05 cm (7.5 in)	Sand 62.23 cm (24.5 in.)

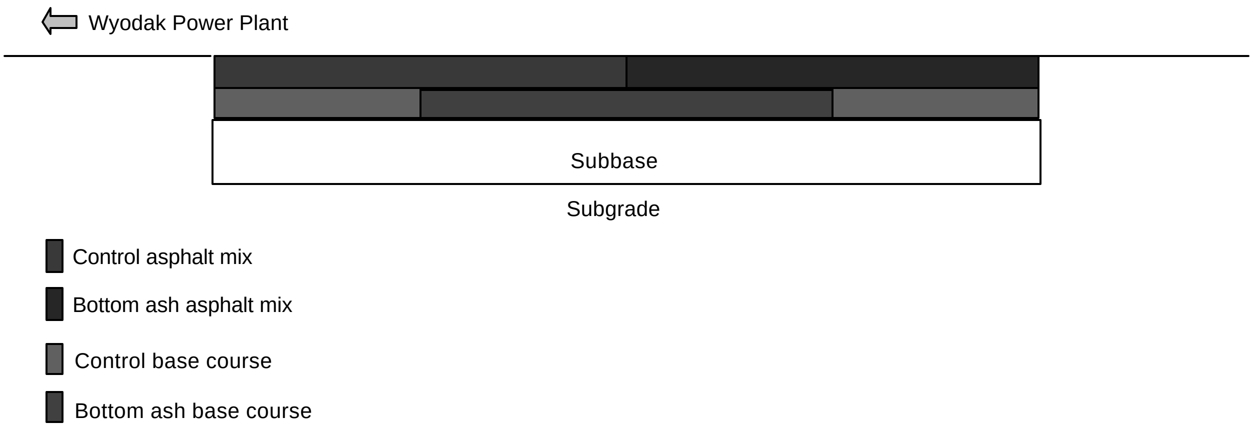

Figure 3.3 Test Section Material Locations.

The control base course used in the construction of the test section was within specifications for Wyoming Department of Transportation (WYDOT) Grading H base course. The bottom ash base course was 100 percent Wyodak bottom ash [CE&MT].

CE&MT designed the control and bottom ash asphalt mixes using the Marshall mix design method. The aggregate blend chosen for the control asphalt mix contained a combination of coarse aggregate, crushed fines, and sand filler. The bottom ash and aggregate blend chosen for the bottom ash asphalt mix contained coarse aggregate, crushed fines, and Wyodak bottom ash. The control and bottom ash asphalt mixes were applied to the test section in two lifts. The first lift of control mix was 8 cm (3 in.), and the first lift of bottom ash mix was 10 cm (4 in.). In both cases, the final lift of asphalt was 5 cm (2 in.) [CE&MT].

Laboratory Evaluation of Test section Asphalt Mixes

Laboratory tests on the asphalt mixes used to construct the test section were performed at the University of Wyoming. The laboratory testing predicted the future field performance of test sections and would later correlate the laboratory results with actual field performance. Hot Mix Asphalt (HMA) and field cores were obtained from the test sections during and after construction. These samples were used to perform the Marshal stability and flow test, GLWT, and TSRST.

During construction of the pavement test section, samples of the control and bottom ash loose Hot Mix Asphalt (HMA) were obtained from in front of the paver. These samples were brought to the University of Wyoming so that their resistance to rutting and cold temperature cracking could be evaluated using the GLWT and TSRST. For each test, the control HMA and the bottom ash HMA were used to produce two samples. The GLWT samples were compacted to a height of 7.6 cm (3.0 in.) and a diameter of 15.2 cm (6.0 in.) using the gyratory compactor. The TSRST samples were compacted using a static load applied from a hydraulic press. This produced 7.6 x 7.6 x 38.1 cm (3.0 x 3.0 x 15.0 in.) square samples. These samples were then cored to achieve a test specimen 5.1 cm (2.0 in.) in diameter and approximately 24.9 cm (9.8 in.) in length.

After construction of the test section was completed, eight cores were extracted by CE&MT and sent to the University of Wyoming for testing. Four of these samples were tested using the Marshall testing apparatus to determine their stability and flow. The remaining four samples, which included two control and two bottom ash samples, were cored to a 15.2 cm (6.0 in.) diameter and tested in the GLWT. After all Marshall and GLWT were completed, the Wyoming Department of Transportation subjected the core samples to an extraction test, which verified their asphalt cement content and aggregate gradation.

Test Section Evaluation

In March 1998, a field performance evaluation was conducted. The evaluation was accomplished by determining the pavement's present condition using the PCI. The present condition was then compared to its original condition in the previous year. Although the study described here was completed after only one field evaluation, the test area will continue to be evaluated on an annual basis.

Part 2: University of Wyoming Designed Mixes Evaluation

For this portion of the study, the University of Wyoming received bottom ash samples from the Wyodak, Jim Bridger, and Naughton power plants. Samples of the aggregate and the AC-20 asphalt cement used to construct the test section also were received. After acquiring all needed materials, a sieve analysis was performed on the aggregate and bottom ash samples to determine their gradation.

Control and bottom ash asphalt mixes were designed using the Marshall mix design. Samples were prepared based on these designs and evaluated using GLWT and TSRST. The main objective of the tests was to determine how well asphalt mixes would perform at high and low temperatures.

Marshall Mix Design of University of Wyoming Mixes

Four asphalt mixes were designed using the Marshall mix design. They included one control mix and three bottom ash mixes. The control mix contained the same aggregate and AC-20 asphalt cement as the test section control mix. Each bottom ash mix contained AC-20 asphalt cement — the same aggregate used for the test section, and one of the three types of bottom ash. The aggregate and aggregate/bottom ash combinations were blended to have gradations similar to the control and bottom ash segments of the Wyodak test section.

Laboratory Testing of University of Wyoming Mixes

The GLWT and TSRST were used to evaluate the laboratory asphalt mixes. For the Georgia Loaded Wheel Test, samples were short-term aged and then compacted using the gyratory compactor. The purpose of short-term aging was to replicate the aging process that occurs in the field during the asphalt mixing and construction process.

For the Thermal Stress Restrained Specimen Test, asphalt mixes were aged and then compacted using a static load applied by a hydraulic press. After compaction, samples were cored to produce the desired sample size.

Data Analysis

After completing the field and laboratory evaluation, the resulting data was entered into a database and analyzed. The purpose of analyzing the data was to determine if the asphalt mixes containing bottom ash performed significantly different than the control asphalt mix. It was decided that the best approach to determine this was through the use of a statistically-based analysis. This analysis was completed using a full Analysis of Variance (ANOVA) and the statistical deviation of the difference between the means.

Chapter Summary

This chapter described strategies used in this two-part study to evaluate the possible use of bottom ash in asphalt mixes. Part one involved evaluation of the field and laboratory performance of the test section containing a control and bottom ash asphalt mixes. Part two involved the design and laboratory evaluation of one control and three bottom ash asphalt mixes. The data acquired throughout the field and laboratory testing was inserted into a database. This information was then analyzed using standard statistical analysis procedures so conclusions and recommendations could be obtained.

CHAPTER 4

RESULTS FROM FIELD AND LABORATORY EVALUATIONS

Field and laboratory evaluations used in this research project evaluated performance of asphalt mixes containing bottom ash. The field evaluation was accomplished by observing and collecting data on the test sections constructed at the Wyodak power plant. Consolidated Engineers and Materials Testing, Inc. (CE&MT) provided the Marshall mix designs, field compaction test results, construction observations, and Marshall test results on asphalt mixes used during construction. The load applied to the pavements and the Pavement Conditioning Index (PCI) were determined by the University of Wyoming to assess the performance of the test road over time. This portion of the evaluation will continue on an annual basis.

The laboratory evaluation was conducted at the University of Wyoming. It involved testing field and laboratory samples using the Georgia Loaded Wheel Tester (GLWT) and Thermal Stress Restrained Specimen Tester (TSRST). The purpose of these tests was to predict how well asphalt mixes would perform under high and low temperature conditions. The laboratory mixes were designed using the Marshall mix design procedure.

Test Sections Construction and Evaluation

As described in the design of experiment, the test road contained a control and bottom ash pavement section. CE&MT designed the asphalt mixes for each section using the Marshall mix design method.

Test Sections Asphalt Mix Designs

To determine the asphalt mix design for the control section of the test road, CE&MT completed two Marshall mix designs. The designs used two different aggregate blends and AC-10 asphalt cement.

The aggregate blends were a 19.0-mm (¾-in.) blend and a 12.5-mm (½-in.) blend. The 19.0-mm (¾-in.) aggregate blend contained a combination of 19.0-mm (0.75-in.) course aggregate, crushed fines, and sand filler. The 12.5-mm (½-in.) aggregate blend contained 12.5-mm (0.5-in.) course aggregate, crushed fines, and sand filler. The gradation of each individual aggregate set used to create the two aggregate blends was supplied to CE&MT by Cundy Asphalt Paving and is shown in Table 4.1.

Table 4.1 Gradation of the Individual Aggregates Used for Control Mixes.

Sieve Size	Percent Passing			
	¾ in. Aggregate	½ in. Aggregate	Crushed Fines	Sand Filler
25.0 mm (1.0 in.)	100	--	--	--
19.0 mm (3/4 in.)	95	100	--	--
12.5 mm (1/2 in.)	45	94	--	--
9.5 mm (3/8 in.)	26	66	100	--
4.75 mm (No. 4)	5	8	93	--
2.36 mm (No. 8)	2	2	54	100
0.600 mm (No. 30)	1	2	20	95
0.075 mm (No. 200)	1.1	1.4	8.0	17.8

Based on gradation of each aggregate set, CE&MT determined that a 45/40/15 split of course aggregate, crushed fines, and sand filler should be used for three-quarter and one-half inch aggregate blends. The resulting gradations for each blend and the specifications used by CE&MT are shown in Table 4.2.

The Marshall mix designs determined that the optimum asphalt content for the 19.0-mm (¾-in.) blend was 4.75 percent. For the 12.5-mm (½-in.) blend, the optimum asphalt content was 5.0 percent.

Table 4.2 Aggregate Blend Gradations and Tolerances Used for the Control Marshall Mix Designs.

Sieve Size	Percent Passing			
	¾ in. Blend (45/40/15)	**¾ in. Blend Tolerances**	**½ in. Blend (45/40/15)**	**½ in. Blend Tolerances**
25.0 mm (1.0 in.)	100	100	--	--
19.0 mm (3/4 in.)	97	97 – 100	100	100
12.5 mm (1/2 in.)	--	--	97	97 – 100
9.5 mm (3/8 in.)	67	60 – 74	78	--
4.75 mm (No. 4)	53	46 – 60	53	46 – 60
2.36 mm (No. 8)	37	32 – 42	38	33 – 43
1.18 mm (No. 16)	--	--	--	--
0.600 mm (No. 30)	23	18 – 28	23	18 – 28
0.295 mm (No. 50)	--	--	--	--
0.150 mm (No. 100)	--	--	--	--
0.075 mm (No. 200)	5.0	3.0 - 8.0	5.0	3.0 – 8.0

For the bottom ash section of the test road, CE&MT completed four Marshall mix designs. These designs used AC-10 and AC-20 asphalt cements and three different aggregate/bottom ash blends. The course aggregate and crushed fines used for the bottom ash blends were obtained from the same source as the control blends. However, the bottom ash blends used Wyodak bottom ash in place of the sand filler used in the control blends. Table 4.3 shows the gradations determined by Cundy Asphalt Paving of the course and crushed fines used to create the different blends. These gradations differed slightly from the gradations of the similar control aggregate. The gradation of the Wyodak bottom ash was determined by CE&MT and also is shown in Table 4.3.

Table 4.3 Gradation of the Individual Aggregates Used in Bottom Ash Mixes.

Sieve Size	Percent Passing			
	¾ in. Aggregate	**½ in. Aggregate**	**Crushed Fines**	**Bottom Ash**
25.0 mm (1.0 in.)	100	--	--	--
19.0 mm (3/4 in.)	100	100	--	--
12.5 mm (1/2 in.)	50	99	--	--
9.5 mm (3/8 in.)	27	83	100	100
4.75 mm (No. 4)	5	10	90	94
2.36 mm (No. 8)	3	5	57	85
1.18 mm (No. 16)	--	--	--	72
0.600 mm (No. 30)	1.5	3	21	59
0.295 mm (No. 50)	--	--	--	46
0.150 mm (No. 100)	--	--	--	27
0.075 mm (No. 200)	1.5	1.5	8.0	13

The three aggregate blends used for the bottom ash mix designs consisted of one 19.0-mm (¾-in.) blend and two 12.5-mm (½-in.) blends. The course aggregate, crushed fines, and bottom ash proportions used for the 19.0-mm (¾-in.) blend were 40/40/20. The proportions of each material used for the two 12.5-mm (½-in.) blends were 45/40/15 and 40/40/20. Based on the individual aggregate and bottom ash gradations, the final gradation for each blend determined by CE&MT is shown in Table 4.4 with their respective tolerances.

Table 4.4 Aggregate Blend Gradations and Tolerances Used for the Bottom Ash Marshall Mix Designs.

Sieve Size	Percent Passing				
	¾ in. Blend (40/40/20)	¾ in. Blend Tolerances	½ in. Blend (45/40/15)	½ in. Blend (40/40/20)	½ in. Blend Tolerances
25.0 mm (1.0 in.)	100	100	--	--	--
19.0 mm (3/4 in.)	100	97 – 100	100	100	100
12.5 mm (1/2 in.)	80	--	100	100	97 – 100
9.5 mm (3/8 in.)	71	60 – 74	92	93	--
4.75 mm (No. 4)	57	46 – 60	55	59	46 – 60
2.36 mm (No. 8)	41	32 – 42	38	40	33 – 43
1.18 mm (No. 16)	--	--	--	--	--
0.600 mm (No. 30)	21	18 – 28	19	21	18 – 28
0.295 mm (No. 50)	--	--	--	--	--
0.150 mm (No. 100)	--	--	--	--	--
0.075 mm (No. 200)	6.4	3.0 - 8.0	5.8	6.4	3.0 – 8.0

For the 19.0-mm (¾-in.) blend, two mix designs were completed; one used the AC-10 and the other used AC-20 asphalt cement. For the 12.5-mm (½-in.) aggregate, the 45/40/15 blend used AC-10 and the 40/40/20 blend used AC-20. The Marshall mix design results for the bottom ash mixes are summarized in Table 4.5.

Table 4.5 Marshall Mix Design Results for Bottom Ash Mixes.

Aggregate Blends	AC-10 Mixes	AC-20 Mixes
¾ in. (40/40/20)	7.5	8.0
½ in. (45/40/15)	7.0	--
½ in. (40/40/20)	--	7.5

Test Road Materials and Construction Evaluation

During construction, test sections were evaluated by CE&MT through visual observations and field density tests. Observations noted during construction include the following:

- The asphalt mixes appeared “oily” and flush during compaction.
- The bottom ash mix appeared “tender” and shoved with little effort.
- The compaction effort used for the top lift of the bottom ash mix was less than specified.

Compaction tests were performed on each compacted lift of the control and bottom ash asphalt pavements. The results of the tests indicated that the first and second lifts of the control section and the first lift of the bottom ash section were over-compacted. Consequently, the compaction effort applied to the top lift of the bottom ash pavement was reduced. Figure 4.1 shows the bleeding that occurred after the first lift of the bottom ash pavement was compacted. A summary of the CE&MT density analysis is shown in Table 4.6.

Figure 4.1 Bleeding of Bottom Ash Pavement.

Table 4.6 Asphalt Pavement Compaction Test Results.

Pavement Type	Lift	Average Percent Compaction Achieved	Specified Percent Compaction
Control	Top	97.0	95
Control	Bottom	99.0	95
Bottom Ash	Top	95.0	95
Bottom Ash	Bottom	98.7	95

CE&MT collected and evaluated samples of the aggregate and loose Hot Mix Asphalt (HMA) during construction. The aggregate was tested to determine its gradation and asphalt samples were evaluated to determine their Marshall properties. Gradation of the aggregate was determined to be within the desired specifications. Marshall testing revealed that flow values for both the control and bottom ash asphalt mixes were higher than the desired specifications. Testing also showed that the percent air voids of bottom ash samples were lower than the desired specifications.

Test Road Performance Evaluation

On March 20, 1998, the first performance survey was conducted to determine PCI of the test sections. This was approximately nine months after the test section had been constructed. During this period, approximately 5,400 loads of bottom ash had been hauled over the section. During the performance survey, it was observed that trucks returning to the power plant generally travel in the same wheel path on the test section as the trucks leaving the power plant. Therefore, the total load generated in the wheel path since construction was calculated to be 303,480,000 kg (669,600,000 lb.). This equals approximately 88,500 Equivalent Axle Loads (EALs).

Procedure for Determining the Pavement Condition Index

Three basic steps determine PCI for a particular pavement section. First, the pavement is divided into inspection units. An inspection unit usually is defined as an area approximately 232 m^2 (2500 ft^2). Next, a condition survey is performed on each inspection unit by determining different distress types present and their severity. Possible survey distress types include alligator cracking, bleeding, block cracking, bumps, sags, corrugation, depressions, edge cracking, reflection cracking, lane or shoulder drop-off, longitudinal cracking, transverse cracking, patching, aggregate polishing, potholes, railroad crossing, rutting, shoving, slippage cracking, swell, weathering, and raveling. The procedures for determining the different distress types and their severity are described in Shahin, 1994. If the number of inspection units is large, a limited number of units may be surveyed to reduce time and resources needed. The degree to which the section must be surveyed depends on the pavement use and the level at which the survey is being conducted. When the condition survey has been completed, the PCI is calculated.

The PCI calculation is based on deduct values or weighing factors ranging from 0 to 100. A deduct value of 0 means the distress has no effect on the pavement's performance. A deduct value of 100 means the distress has a significantly large effect on the pavements performance. PCI deduct values are determined using a deduct curve for each distress type [Shahin 1994]. The PCI deduct curves can be found in Shahin 1994.

Test Road Pavement Condition Results

The PCI was determined for each of the four 22.9-m (75-ft.) sections of the test road. Each of these sections was 10.4 m (34 ft.) wide, making the cross-sectional area of each section 238.2 m^2 (2550 $ft.^2$). This was smaller than the recommended area for an inspection unit, but could not be avoided due to the short length of each test section. Of the distress types used to determine the PCI, none of them were found to be present to any degree. This results in a current PCI of 100. Figure 4.2 shows the rut depth being measured in the left wheel path during the pavement condition survey. This figure also shows the single path used by both directions of travel.

The condition of the test road will continue to be monitored throughout the life of the test pavement. Over time, the results of the additional surveys will be compared to the results of accelerated performance testing performed in the laboratory.

Figure 4.2 Rut Depth Measurement for the Pavement Condition Survey.

Laboratory Evaluation

Field and laboratory designed samples were evaluated using a variety of tests including the GLWT and TSRST. The field samples consisted of control and bottom ash HMA and core samples taken from the test road. Laboratory samples included control and bottom ash mixes designed in the laboratory. The remainder of this chapter contains a description of procedures used to test the samples followed by the test results.

Georgia Loaded Wheel Tester

Originally, the GLWT was designed to test asphalt beams 7.62 x 7.62 x 38.1 cm (3 x 3 x 15 in.). However, a set of procedures was developed at the University of Wyoming to test cylindrical samples 15 cm (6 in.) in diameter and 7.6 cm (3.0 in.) tall. These samples require less material, are easier to handle, can be compacted in the lab with less effort, and are more readily acquired from the field [Miller 1995].

The laboratory-compacted samples tested in the GLWT were prepared using the gyratory compactor. The compaction procedure was based on SHRP designation M-002, "Standard Test Method for Preparation of Compacted Specimens of Modified and Unmodified Hot Mix Asphalt by Means of the SHRP Gyratory Compactor" found in Harrigan et al. 1994. The Troxler gyratory compactor used at the University of Wyoming is shown in Figure 4.3.

Figure 4.3 Gyratory Compactor Used at the University of Wyoming.

First, the amount of HMA required to produce the desired sample size at a specific density was calculated. If needed, the aggregate and asphalt cement were then heated to the appropriate temperature and mixed. Next, the sample was allowed to reach the compaction temperature, placed into the preheated gyratory compaction mold, and loaded into the compactor. The sample was then compacted to a specified height or number of gyrations. After the sample was compacted, it was allowed to cool and removed form the mold.

Initially, the amount of material needed to achieve optimum density was based on the compacted sample size. Samples were then compacted until they reached a height of 7.6-cm (3.0-in.). It was later determined that this procedure produced samples that were too dense. Imperfections on the surface of samples do not allow for an accurate calculation of the needed materials based on the desired density and sample dimensions. Later, trial specimens were used to determine the amount of material and number of gyrations needed to achieve the desired density for each sample type. Samples were then compacted based on these findings.

Prior to being tested in the GLWT, compacted asphalt samples were heated to a testing temperature of 46.1^{o}C (115^{o}F). They were then confined in the GLWT using sample-holding molds. An initial measurement was taken using three dial indicators attached to an aluminum dowel. Next, a hose inflated to 689 kPa (100 psi.) was secured over the sample. The wheel, loaded using 45.4 kg (100 lb.) of weight, was lowered on top of the sample. The machine was turned on and a motor caused the wheel to travel back and forth over the rubber hose producing a 689-kPa (100-psi.) contact pressure between the hose and sample. A sample being tested in the GLWT is shown in Figure 4.4.

At a predetermined number of cycles, normally 1,000, 4,000, and 8,000, the GLWT was stopped and rut depth measurements were recorded. One cycle consists of a single load in each direction. An asphalt sample is considered to pass if after 8,000 cycles the rut depth is less than 0.762 cm (0.30 in.) [Miller, 1995].

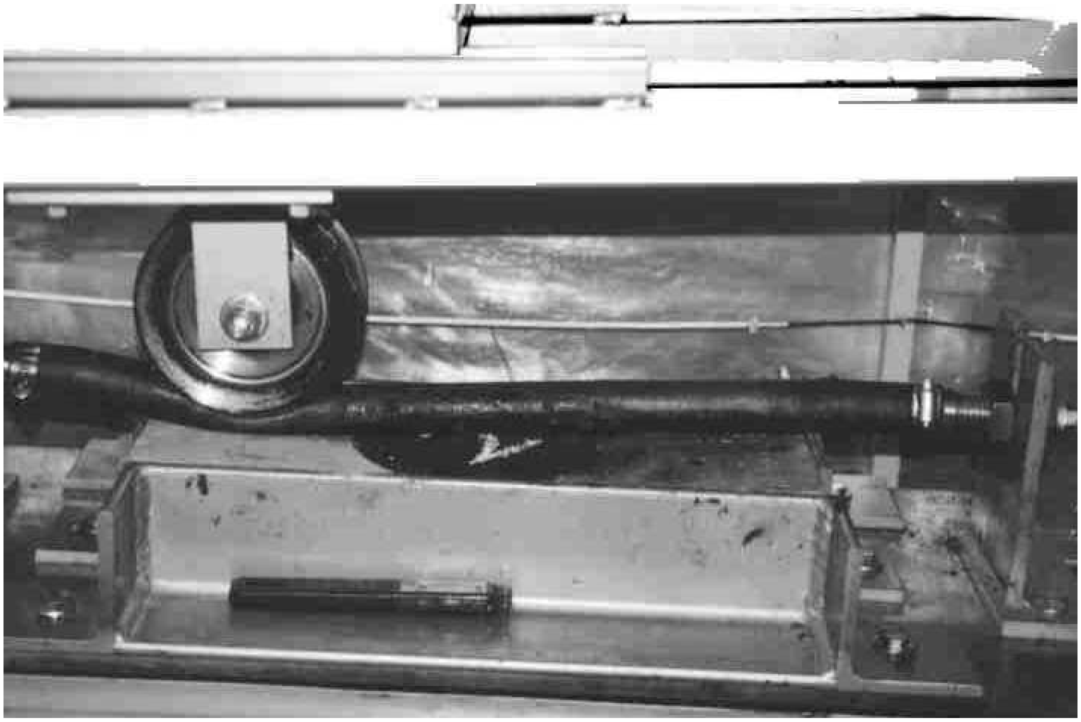

Figure 4.4 Sample Being Tested Using Georgia Loaded Wheel Tester.

Thermal Stress Restrained Specimen Tester

The TSRST is capable of testing a variety of sample sizes. However, samples tested in this study were 5.08 cm (2.0 in.) in diameter and approximately 25 cm (9.8 in.) in length. At the University of Wyoming, these samples were manufactured by coring 7.62 x 7.62 x 38.1 cm (3 x 3 x 15 in.) asphalt beams compacted using a static load from a hydraulic press.

Asphalt beams were constructed by placing the appropriate amount of Hot Mix Asphalt (HMA) in a heated steel mold in three even lifts with each lift being tamped 20 times. Compaction was accomplished by applying a static load three times. The first two loads were applied and then immediately released. The third load was applied and sustained for five minutes. The amount of load needed to achieve the desired densities was determined by compacting trial specimens. After compaction, the samples were cooled and removed from the mold. Coring of the samples was accomplished using a 5.08-cm (2.0-in.)

coring bit and drill press. They were then trimmed to an approximate length of 25-cm (9.8-in.) using a table saw.

Prior to testing a sample in the TSRST, it is epoxied between two end platens. The epoxy is allowed to cure while the platens are attached to a stand to insure proper alignment as shown in Figure 4.5. Next, spring-loaded alignment rods are placed between the top and bottom platens to compensate for the weight of the sample while it hangs in the testing chamber. Invar rods are attached to the bottom platens and linear variable differential transformer (LVDT) holders are attached to the top platens. LVDTs rest against the end of the invar rods and measure the displacement of the sample during testing. The sample is then secured with swivel connectors in the environmental cabinet between the load cell and step motor. Four resistance temperature devices (RTDs) are placed on the specimen to read its temperature during testing. The temperature control RTD is attached to the top platen and the LVDTs are placed in their holders. The sample is precooled to between $2^{o}C$ ($35.6^{o}F$) and $4^{o}C$ ($39.2^{o}F$) before it is tested. This may be accomplished in the TSRST or prior to securing the sample in the cabinet [Harrigan, Leahy, and Youtcheff 1994]. A sample prepared for testing is shown in Figure 4.6.

Figure 4.5 Mounting of TSRST Samples.

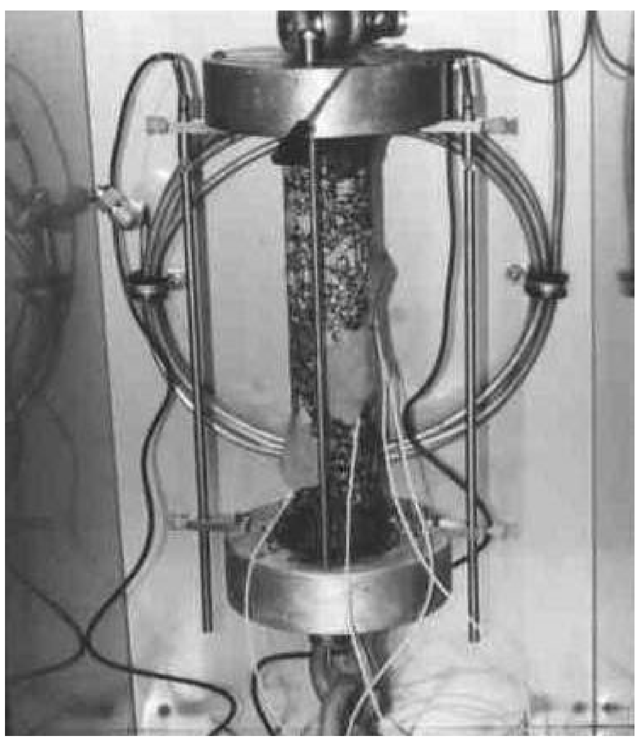

Figure 4.6 A Sample Prepared for TSRST Testing.

During the testing process, the environmental cabinet is cooled at a rate of 10.0°C/hr (18.0°F/hr) using liquid nitrogen. As the specimen contracts due to cooling, the step motor pulls it back to its original size creating tensile stress. The data acquisition system uses the load cell, LVDTs, and RTDs to record the sample load, temperature, and displacement at set time intervals. The test is complete when the sample fractures. The data recorded at the completion of the test includes type of failure, time to failure, specimen temperature at failure, ultimate load, ultimate strength, and slope of the thermally induced stress curve [Harrigan et. all 1994].

Laboratory Test Results on Field HMA

Loose HMA samples of the control asphalt mix and bottom ash asphalt mix were acquired from test sections during construction. These samples were taken to the University of Wyoming for testing in the GLWT and TSRST.

For the GLWT, two control and two bottom ash samples were prepared using the gyratory compactor. Samples were compacted to achieve optimum design densities based on their respective Marshall mix designs. For control samples, this density was 2.390 g/cm^3 (149.2 lb/ft^3). For bottom ash samples, the optimum density was 2.276 g/cm^3 (142.1 lb/ft^3). To achieve a density close to optimum, the control samples were compacted using 25 gyrations and bottom ash samples were compacted using five gyrations. After compaction, the dimensions and bulk specific gravity of each sample were determined. The specific gravity was calculated using AASHTO Designation T 166-93, "Standard Method of Test for Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens."

During GLWT testing, each sample's rut depth was measured at the conclusion of 1,000, 4,000, and 8,000 cycles. The complete test results for each sample tested is shown in Appendix A. A summary of these results including sample densities, and rut depth after 8,000 cycles is shown in Table 4.7. The samples GLWT-C-12 and GLWT-W-12 are shown after being tested in the GLWT in Figures 4.7 and 4.8 respectively.

Table 4.7 GLWT Results for Field HMA.

Sample	Sample Type	Density (g/cm^3)	Average Rut Depth After 8000 Cycles (cm)
GLWT-C-12	Control	2.410	0.25
GLWT-C-13	Control	2.409	0.28
	Average	**2.410**	**0.27**
GLWT-W-12	Bottom Ash	2.292	0.64
GLWT-W-13	Bottom ash	2.286	0.54
	Average	**2.289**	**0.59**

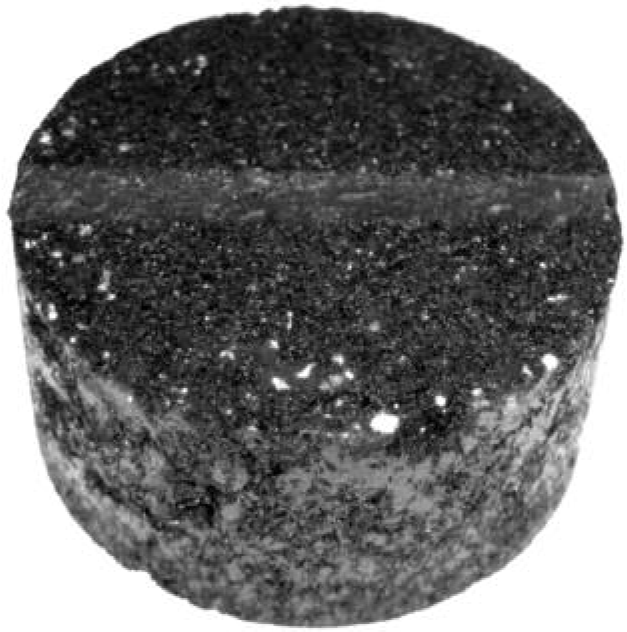

Figure 4.7 Rutting of GLWT Control Sample GLWT-C-12 After 8,000 Cycles.

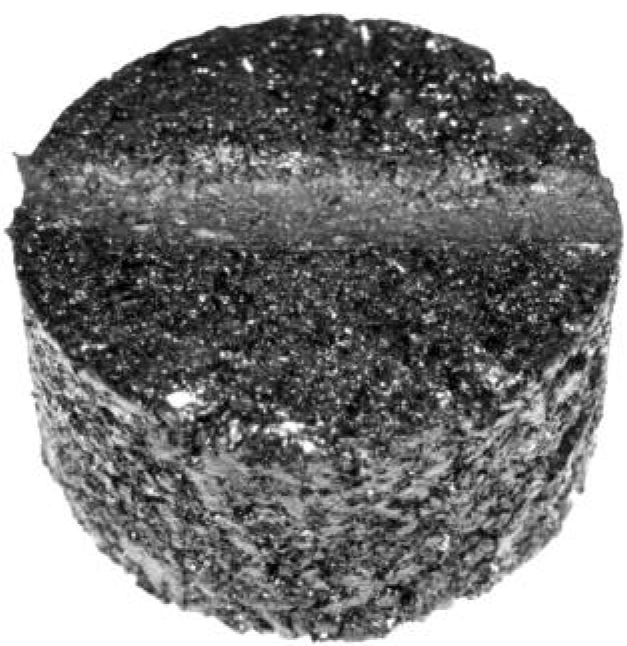

Figure 4.8 Rutting of GLWT Wyodak Sample GLWT-W-12 After 8,000 Cycles.

Two control and two bottom ash core samples were prepared for testing in the TSRST. Again, the optimum densities for control and bottom ash samples were 2.390 g/cm^3 (149.2 lb/ft^3) and 2.276 g/cm^3 (142.1 lb/ft^3) respectively. To attain these densities, control sample beams were compacted using a 222.4 kN (50,000 lb.) load and bottom ash beams were compacted using a 177.9 kN (40,000 lb.) load. Densities of the compacted control samples were slightly lower than the optimum density. However, if a larger compaction load was used the sample's aggregate fractured.

After samples were compacted and cored, their dimensions and bulk specific gravity were determined using AASHTO procedure T 166-93. Samples were then tested in the TSRST. During testing, the sample temperature and load were recorded in two-minute intervals. Following the fracture of each sample, the type, time, temperature, pressure, and load at failure were determined. The slope of the thermal stress curve also was determined from a stress versus temperature plot. Figure 4.7 shows a typical sample after being tested in the TSRST. The TSRST results for each sample and the stress versus temperature plots are found in Appendix A. A summary of these results is shown in Table 4.8. A typical sample after being fractured in the TSRST is shown in Figure 4.9.

Table 4.8 TSRST Results for Field HMA.

Sample	**Sample Type**	**Density (g/cm^3)**	**Failure**			
			Time (min.)	**Temp. (^{o}C)**	**Load (kN)**	**Pressure (kPa)**
TSRST-C-10	Control	2.357	202	-28.1	4.48	2212
TSRST-C-11	Control	2.361	208	-28.2	4.93	2432
	Average	**2.359**	**205**	**-28.2**	**4.71**	**2322**
TSRST-W-10	Bottom Ash	2.279	182	-24.4	6.21	3064
TSRST-W-11	Bottom Ash	2.291	180	-23.7	5.65	2787
	Average	**2.285**	**181**	**-24.1**	**5.93**	**2926**

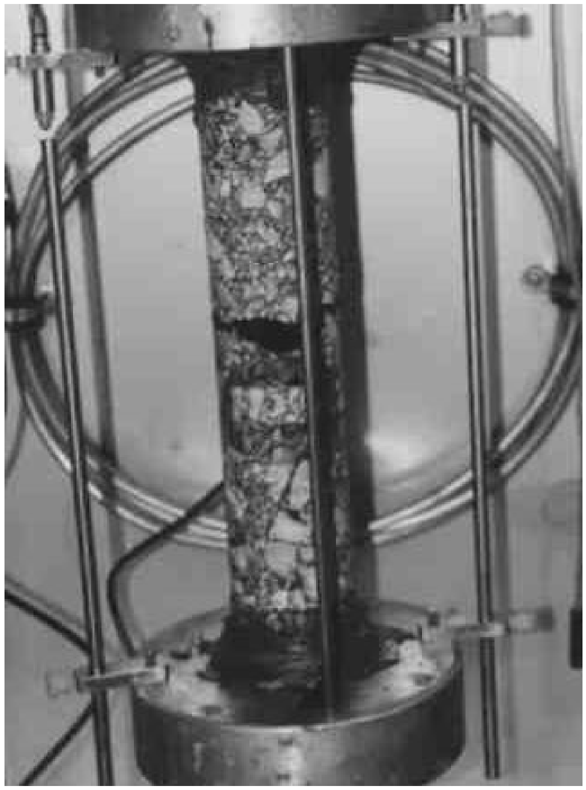

Figure 4.9 A Fractured TSRST Sample.

Laboratory Results for Field Core Samples

Eight core samples of the test road pavement were collected by CE&MT and shipped to the University of Wyoming for testing. Four of the samples were tested using the GLWT and four were tested using the Marshall apparatus. CE&MT noted that during coring bottom ash samples were tender and difficult to extract from the coring bit. The core samples also appeared to contain excessive asphalt cement. Because of these observations, the Wyoming Department of Transportation (WYDOT) was asked to perform an extraction test on several of the samples to verify their aggregate gradation and asphalt content.

Two control and two bottom ash core samples were tested using the GLWT. Prior to performing the tests, each sample's BSG was determined. During the early stages of testing, the control and the bottom ash samples began to display severe rutting. In all four cases the samples failed prior to being subjected to 8,000 cycles. Close examination of the failed samples revealed small cracks developing on their sides just below the ruts. An example of a failed sample containing these cracks is shown in Figure 4.10. The GLWT test results for each sample are shown in Appendix B. A summary of these results is found in Table 4.9.

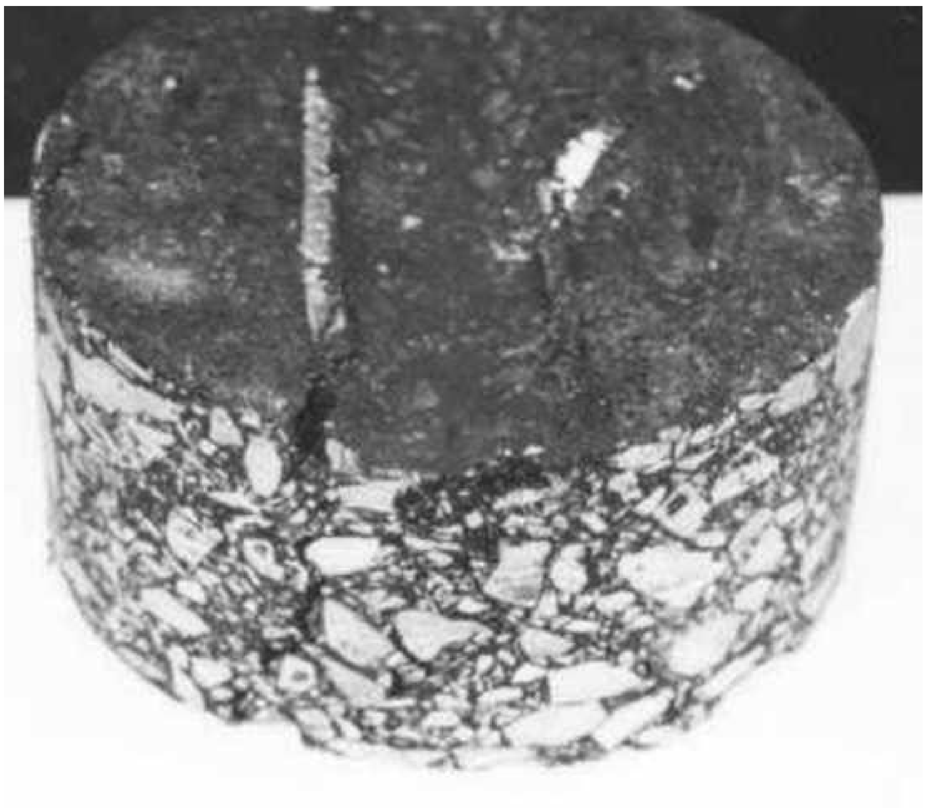

Figure 4.10 Failed GLWT Core Sample.

Table 4.9 GLWT Results for Field Cores.

Sample	Sample Type	Density (g/cm^3)	Number of Cycles	Average Rut Depth After 8000 Cycles (cm)
GLWT-C-1	Control	2.369	7000	0.77
GLWT-C-2	Control	2.346	4000	0.89
	Average	**2.358**	**5500**	**0.83**
GLWT-W-1	Bottom Ash	2.271	475	1.02
GLWT-W-2	Bottom ash	2.303	675	0.84
	Average	**2.287**	**575**	**0.93**

The Marshall samples received from CE&MT for stability and flow testing had a 9.5-cm (3.75-in.) diameter, rather than the required 10.2-cm (4.0-in.) diameter. These samples could not be tested in the Marshall Equipment.

The core samples tested in the GLWT were taken to WYDOT for extraction testing. The results from these tests supplied by WYDOT are shown in Appendix C and summarized in Table 4.10. The tests verified that gradation of the aggregate blends used for the control and bottom ash sections of the test road were within specified tolerances for the project. They also showed that asphalt contents for both sections were close to the design optimum asphalt contents determined using the Marshal mix design.

Table 4.10 Extraction Test Results for Field Cores.

Sieve Size	Percent Passing					
	Control			Bottom Ash		
	Sample #1	Sample #2	Average	Sample #1	Sample #2	Average
25.0 mm (1.0 in.)	100	100	100	100	100	100
19.0 mm (3/4 in.)	99	97	98	98	96	97
12.5 mm (1/2 in.)	88	78	83	81	73	77
9.5 mm (3/8 in.)	77	68	73	74	61	68
4.75 mm (No. 4)	60	53	57	57	45	51
2.36 mm (No. 8)	41	37	39	36	31	34
1.18 mm (No. 16)	30	27	29	25	21	23
0.600 mm (No. 30)	24	22	23	18	16	17
0.295 mm (No. 50)	17	16	17	14	12	13
0.150 mm (No. 100)	10	9	10	11	9	10
0.075 mm (No. 200)	6.9	6.7	6.8	8.1	6.6	7.4
AC Content	5.29	4.89	5.09	8.47	7.44	7.96

Marshal Mix Design of Laboratory Mixes

For the laboratory portion of this study, the Marshall mix design procedure was used to design four asphalt mixes. These mixes included one control mix and three bottom ash mixes. The same aggregate and asphalt cement used to construct the test sections also were used in this part of the experiment. The bottom ashes used were from Wyodak, Jim Bridger, and Naughton power plants.

Upon receiving the materials at the University of Wyoming, a sieve analysis was performed on the aggregate and bottom ashes following procedures described in WYDOT 406.0 and WYDOT 407.0. The sieve analysis procedure used for the bottom ash also was found in the WYDOT testing manual. The sieve analysis for the bottom ash was performed on materials passing the 9.5-mm (3/8-in.) sieve. Any bottom ash material greater than 9.5 mm (3/8 in.) was discarded. This was done to produce a more consistent bottom ash gradation between lab samples and to bring the gradation of bottom ashes from other locations closer to the gradation of Wyodak bottom ash. The aggregate and bottom ash sieve analysis results are shown in Table 4.11.

Table 4.11 Aggregate and Bottom Ash Sieve Analysis Results for Lab Samples.

Sieve Size	Percent Passing					
	¾ in. Aggregate	Crushed Fines	Sand Filler	Wyodak	Jim Bridger	Naughton
25.0 mm (1.0 in.)	100	100	100	100	100	100
19.0 mm (3/4 in.)	94.6	100	100	100	100	100
12.5 mm (1/2 in.)	51.3	100	100	100	100	100
9.5 mm (3/8 in.)	23.0	100	100	100	100	100
4.75 mm (No. 4)	5	79	100	93	87	82
2.36 mm (No. 8)	4	45	100	83	77	69
1.18 mm (No. 16)	--	--	--	70	67	58
0.600 mm (No. 30)	3	16	94	57	58	50
0.295 mm (No. 50)	--	--	--	43	45	39
0.150 mm (No. 100)	--	--	--	24	25	25
0.075 mm (No. 200)	2.2	7.8	20.0	11.0	9.4	11.8

The blending of aggregates and bottom ash for the laboratory mix designs was performed to achieve gradations similar to those used in test sections. The samples also were blended to be within WYDOT gradation specifications. For the control samples, proportions of three-quarter inch aggregate, crushed fines, and sand filler were 45/40/15. For the bottom ash samples, three different Wyodak bottom ash and aggregate blends were first analyzed. This was done to determine the maximum proportion of bottom ash that could be used and still produce a mix that would pass the Marshall mix design criteria. It was found that a 40/40/5/15 combination of three-quarter inch aggregate, crushed fines, sand filler, and bottom ash produced the best results. This blend contained 5 percent less bottom ash than the test section bottom ash mix. Because of this, sand filler was added to achieve the desired gradation. The remaining two blends containing Jim Bridger and Naughton bottom ash used 15 percent bottom ash. The final gradations of all blends used in the Marshall mix designs are shown in Table 4.12.

Table 4.12 Gradations of Blended Samples Used in Laboratory Mix Designs.

Sieve Size	Percent Passing			
	Control (40/45/15)	Wyodak (40/40/5/15)	Jim Bridger (40/38/7/15)	Naughton (40/36/9/15)
25.0 mm (1.0 in.)	100	100	100	100
19.0 mm (3/4 in.)	98	98	98	98
12.5 mm (1/2 in.)	81	81	81	81
9.5 mm (3/8 in.)	69	69	69	69
4.75 mm (No. 4)	53	53	52	52
2.36 mm (No. 8)	37	37	37	37
0.600 mm (No. 30)	22	21	22	23
0.075 mm (No. 200)	7.4	6.7	6.7	7.3

The Marshall mix design was performed on the four aggregate and bottom ash blends. The Marshall data sheets and plots for each mix design is located in Appendix D. The results of these mix designs also are summarized in Table 4.13.

Table 4.13 Marshal Mix Design Results for Lab Samples.

Design Result	Control (40/45/15)	Wyodak (40/40/5/15)	Jim Bridger (40/38/7/15)	Naughton (40/36/9/15)
Optimum AC Content (%)	5.5	7.5	7.0	7.0
Stability, N (lb.)	2669	3156	2966	3062
Flow, 0.25 mm (0.01 in.)	16	13	14	16
% VTM	3	4	4	4
% VMA	14	16	16	14
% VFA	77	78	76	76

Test Results for Laboratory Mixes

The Marshall mix design results were used to prepare laboratory samples for testing in the GLWT and TSRST. These samples were aged using the SHRP short-term aging procedure M-007. This was done to simulate aging that took place during mixing and placing of the asphalt mix used to construct the test road.

For the GLWT, two samples of each laboratory mix were compacted using the gyratory compactor. These samples had densities close to their Marshall optimum densities. The Marshall mix design optimum densities for all mixes are shown in Table 4.14.

Table 4.14 Optimum Densities for Laboratory Asphalt Mixes.

Mix	Control	Wyodak	Jim Bridger	Naughton
Density (g/cm^3)	2.356	2.276	2.297	2.278

The number of gyrations needed for each mix to reach its appropriate sample density was determined by preparing a number of test samples. The control samples required 70 gyrations to reach their optimum density. The bottom ash samples required significantly fewer gyrations as shown in Table 4.15.

Once compacted, the bulk specific gravity was determined for each mix using AASHTO procedure T 166-93. In the GLWT, samples were subjected to 8,000 cycles of loading with rut depth measurements recorded at 1,000, 4,000, and 8,000 cycles. Figures 4.11 and 4.12 show samples GLWT-C-24 and GLWT-J-22 respectively after being tested in the GLWT. The test results for all samples can be found in Appendix E. These results also are summarized in Table 4.15.

Figure 4.11 Rutting of GLWT Control Sample GLWT-C-24 After 8,000 Cycles.

Figure 4.12 Rutting of GLWT Jim Bridger Sample GLWT-J-22 After 8,000 Cycles.

Table 4.15 GLWT Results for Laboratory Designed Samples.

Sample	Sample Type	Number of Gyrations	% Voids	Density (g/cm^3)	Average Rut 8000 Cycles (cm)
GLWT-C-24	Control	70	3	2.355	0.220
GLWT-C-25	Control	70	3	2.360	0.226
	Average	**70**	**3**	**2.358**	**0.223**
GLWT-W-22	Wyodak	45	3	2.284	0.240
GLWT-W-23	Wyodak	45	3	2.277	0.279
	Average	**45**	**3**	**2.281**	**0.260**
GLWT-J-22	Jim Bridger	40	3	2.287	0.519
GLWT-J-23	Jim Bridger	40	4	2.274	0.613
	Average	**40**	**3**	**2.281**	**0.556**
GLWT-N-22	Naughton	25	3	2.282	0.385
GLWT-N-23	Naughton	25	4	2.272	0.475
	Average	**25**	**4**	**2.277**	**0.430**

TSRST samples for each mix also were prepared based on their Marshall mix design results. These samples were compacted using the same procedures and compactive effort as the test section control and bottom ash samples. After being compacted and cored, each sample's dimensions and bulk specific gravity were determined. The samples were then mounted and tested in the TSRST. The TSRST results and stress versus temperature plots are located in Appendix E. A summary of the results is shown in Table 4.16.

Table 4.16 TSRST Results for Laboratory Designed Mixes.

Sample	Sample Type	Density (g/cm^3)	Failure			
			Time (min.)	Temp. (^{o}C)	Load (kN)	Pressure (kPa)
TSRST-C-20	Control	2.348	218	-29.6	4.62	2278
TSRST-C-21	Control	2.343	224	30.0	4.23	2087
	Average	**2.346**	**221**	**-29.8**	**4.42**	**2183**
TSRST-W-20	Wyodak	2.264	218	-29.0	3.34	1648
TSRST-W-21	Wyodak	2.281	232	-31.6	3.86	1903
	Average	**2.273**	**225**	**-30.3**	**3.60**	**1776**
TSRST-J-20	Jim Bridger	2.283	180	-23.7	3.92	1933
TSRST-J-21	Jim Bridger	2.296	184	-26.4	3.64	1795
	Average	**2.290**	**182**	**-25.1**	**3.78**	**1864**
TSRST-N-20	Naughton	2.292	146	-20.8	3.47	1710
TSRST-N-21	Naughton	2.273	150	-20.4	3.18	1567
	Average	**2.283**	**148**	**-20.6**	**3.32**	**1639**

Chapter Summary

In this chapter, the laboratory testing and data collection procedures and results were presented. For the test sections, the PCI was used to evaluate the field performance. In the laboratory, the Georgia Loaded Wheel Test and Thermal Stress Restrained Specimen Test were used to predict high and low temperature field performance. In the following chapter, a statistics based analysis of the test results is presented.

CHAPTER 5

STATISTICAL ANALYSIS OF DATA

Following the laboratory evaluation described in the previous chapter, a statistical analysis was performed on the Georgia Loaded Wheel Tester (GLWT) and Thermal Stress Restrained Specimen Tester (TSRST) mean test results. The analysis was performed using an Analysis of Variance (ANOVA) and the statistical deviation of the difference between the means.

This chapter describes the statistical analysis used to evaluate laboratory test data. A complete set of the analysis results can be found in Appendix F. The ANOVA was performed using MINITAB for Windows, release 11.21, by Minitab, Inc.

Statistical Analysis

An ANOVA was performed on four separate groups of mean laboratory test results. They included the TSRST and GLWT results for two asphalt mixes collected in the field and four asphalt mixes designed in the laboratory. The purpose of the ANOVA was to determine if the mean laboratory test results for a group of asphalt mixes were significantly different. An ANOVA is based on separation of the sums of squares and degrees of freedom associated with a response variable. ANOVA simplifies calculation of the F-test statistic and P-value so that significance of a difference in mean responses can be determined [Neter, Kutner, Nachtsheim, and Wasserman 1996]. The P-value calculated by MINITAB is the probability of observing the F-value or larger when the mean test results are equal. Therefore, if a P-value was less than the desired level of significance α, the hypothesis that two or more mean test results were equal was rejected. The level of significance used to determine if laboratory test results were equal was 0.05. This level of significance produced a 95 percent level of confidence in the statistical analysis results.

The difference between the observed means was used to determine if the mean test results of two different asphalt mixes within a test group were significantly different at a specified level of significance. The difference between pairs of means was investigated when the ANOVA determined that mean test sample results for a group of asphalt mixes were significantly different. The level of significance used was 0.05.

Statistical Analysis of GLWT Data

Asphalt mixes were tested using the GLWT to determine their average resistance to rutting. The mixes included the four designed in the laboratory and the two obtained from the field during construction of the test sections at the Wyodak power plant. The laboratory designed asphalt mixes included a control mix and mixes containing bottom ash from the Wyodak, Jim Bridger, and Naughton power plants. The field mixes included a control mix and a mix containing Wyodak bottom ash.

Statistical Analysis of Laboratory Designed Mixes

Two compacted samples of the control, Wyodak, Jim Bridger, and Naughton laboratory designed mixes were tested using the GLWT. At the end of each test, the rut depth was measured in three equally spaced locations along the samples. These depths were averaged to obtain a mean rut depth for the sample. Next, the mean rut depths of the two samples of each mix were averaged to determine the mean rut depth for that mix.

An ANOVA was performed on the GLWT results to determine if the mean rut depth for the mixes studied were statistically different. The individual rut depth observations were nested within each sample and the samples were nested within each source. The ANOVA for the laboratory designed samples are shown in Appendix F. A summary of these results is shown in Table 5.1. Because the P-value is near zero, the GLWT mean results for the mixes were significantly different.

Table 5.1 ANOVA Results for GLWT Tests on Laboratory Designed Mixes.

Samples	P-value	Significant Difference
All Means	+0.000	Yes

To determine which pair of laboratory-designed asphalt mixes were significantly different; the observed difference between the means was compared with a critical value based on the "t" distribution. The critical value is reported for varying levels of significance from $\alpha = 0.10$ to $\alpha = 0.01$. The results of this analysis are displayed in Appendix F and are summarized in Table 5.2. They indicate that all but two of the mixes produced significantly different results from each other throughout the range of confidence levels used. These samples were the control mix and Wyodak mix.

Table 5.2 The Difference Between the Means Analysis Results for GLWT Tests on Laboratory Designed Mixes.

Results Compared (mix - mix)		Mean Difference (cm)	Significant Difference at a Level of Significance		
			a = 0.1 (0.071)*	a = 0.05 (0.082)*	a = 0.01 (0.104)*
Control	Wyodak	- 0.037	No	No	No
Control	Jim Bridger	- 0.343	Yes	Yes	Yes
Control	Naughton	- 0.207	Yes	Yes	Yes
Wyodak	Jim Bridger	- 0.306	Yes	Yes	Yes
Wyodak	Naughton	- 0.170	Yes	Yes	Yes
Jim Bridger	Naughton	0.136	Yes	Yes	Yes

* Required mean difference needed to achieve the α level of significance shown.

Statistical Analysis of Field Mixes

Two compacted samples of the control and Wyodak mixes collected from the field were tested using the GLWT. The mean rut depth of both mixes were determined using the procedure previously described. The results of these tests were analyzed using an ANOVA to determine if the mixes achieved significantly different average rut depths. Nesting of the observations also was used in this case. The ANOVA results are displayed in Appendix F and are summarized in Table 5.3. These results show that there was a significant difference in the mean rut depths between the two mixes.

Table 5.3 ANOVA Results for GLWT Tests on Field Mixes.

Samples	Mean Difference (cm)	P-Level	Significance Difference
Control and Wyodak	0.32	+0.000	Yes

Statistical Analysis of TSRST Data

The TSRST was used to test different asphalt mixes with respect to their cold temperature susceptibility. The asphalt mixes included the four designed in the laboratory and the two obtained from the field during the construction of the test section at the Wyodak power plant. The laboratory-designed mixes included a control mix and mixes containing bottom ash from the Wyodak, Jim Bridger, and Naughton power plants. The asphalt mix collected in the field included a control mix and a Wyodak bottom ash mix.

Statistical Analysis of Laboratory Designed Mixes

Two compacted samples of the control, Wyodak, Jim Bridger, and Naughton laboratory designed mixes were tested using the TSRST. The fracture temperature of the two samples representing each mix were averaged to obtain the mean results for that mix.

An ANOVA was performed on the TSRST data. The ANOVA determined if a significant difference existed in the mean fracture temperature between the different mixes. The ANOVA results are shown in Appendix F. A summary of these results is shown in Table 5.4. Again, it was concluded that the means are not equal if the P-value was less than the desired level of significance $\alpha = 0.05$. The results indicate that the mean fracture temperature among laboratory designed mixes were significantly different.

Table 5.4 ANOVA Results for TSRST Tests on Laboratory Designed Mixes.

Result	Samples	P-Level	Significance Difference
Fracture Temperature	All Means	0.008	Yes

Once it was determined that the laboratory-designed mixes displayed significant differences in their mean fracture temperature results, the difference between the means was used to determine which pairs of means were significantly different. This analysis was based on the α levels of significance 0.10, 0.05, and 0.01. The results of the analysis are shown in Appendix F. A summary of these results is displayed in Tables 5.5. The results in Table 5.5 indicate that the average TSRST fracture temperature for the asphalt mix containing Naughton bottom ash was significantly lower than the average fracture temperature for the control and Wyodak asphalt mixes. This table also indicates that none of the remaining samples displayed a significant difference in their average fracture temperature results.

Table 5.5 The Difference Between the Means Results for TSRST Fracture Temperature Data on Laboratory Designed Mixes.

Results Compared (mix - mix)		Mean Difference in Fracture Temperature (^oC)	Significant Difference at a Level of Significance		
			a = 0.1 (5.8)*	a = 0.05 (7.5)*	a = 0.01 (13.5)*
Control	Wyodak	- 0.5	No	No	No
Control	Jim Bridger	4.8	No	No	No
Control	Naughton	9.2	Yes	Yes	No
Wyodak	Jim Bridger	5.3	No	No	No
Wyodak	Naughton	9.7	Yes	Yes	No
Jim Bridger	Naughton	4.5	No	No	No

* Required mean difference needed to achieve the α level of significance shown.

Statistical Analysis of Field Mixes

Finally, an ANOVA was performed on the mean TSRST results for the control and Wyodak field samples. These samples were tested using the same procedure as the laboratory designed samples. The ANOVA results are shown in Appendix F. A summary of the results is displayed in Table 5.6. This table shows that the mean TSRST results for the control and Wyodak field mixes did not display significant differences.

Table 5.6 ANOVA Results for TSRST Tests on Field Mixes.

Result	Samples	Mean Difference (^oC)	P-Level	Significant Difference
Fracture Temp.	Control and Wyodak	4.1	0.062	No

Chapter Summary

A statistical analysis was performed on the mean GLWT and TSRST test results. This analysis determined if the asphalt mixes displayed significantly different average resistance to rutting and to cold temperatures.

The statistical analysis performed on the GLWT data found that all but two of the four laboratory-designed mixes displayed a significantly different average resistance to rutting. The two mixes that displayed a similar average resistance to rutting were the control and Wyodak mixes.

The statistical analysis performed on the TSRST results found that the average fracture temperature for the laboratory asphalt mix containing Naughton bottom ash was significantly lower than the average fracture temperature for the control and Wyodak asphalt mixes. The average laboratory fracture temperature for the Jim Bridger asphalt mix was not significantly different from the average fracture temperatures of the other three mixes. The remaining asphalt mixes, including the field mixes, did not display significantly different fracture temperatures when compared to each other.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Summary

This research project used field and laboratory evaluations to study the possibility of using bottom ash in asphalt mixes. The laboratory evaluation involved designing and testing a control and three bottom ash asphalt mixes. In addition, a control and Wyodak bottom ash asphalt mixes were used to construct test sections at the Wyodak power plant. The materials also were evaluated in the laboratory. Laboratory testing was accomplished using the Georgia Loaded Wheel Tester (GLWT) and the Thermal Stress Restrained Specimen Tester (TSRST). The GLWT was used to evaluate the high-temperature rutting characteristics of the asphalt mixes. The TSRST was used to evaluate the low-temperature cracking characteristics of the asphalt mixes. For the field portion of this study, the test sections were evaluated by performing a pavement condition survey to determine the Pavement Condition Index (PCI). Finally, a statistical analysis was performed to determine if the differences in average performance among the asphalt mixes were statistically significant.

Conclusions

Based on observations and testing performed in this study, the following conclusions were drawn:

1. The Marshall mix design results indicate that asphalt mixes containing bottom ash have higher optimum asphalt contents than standard asphalt mixes.
2. Compaction of asphalt mixes during field construction and in the laboratory indicate that asphalt mixes containing bottom ash require less compactive effort to achieve their desired optimum densities than the control asphalt mixes.
3. Initial observations of the test sections indicate no difference in performance between the control and bottom ash sections after one season of being in service.

4. The statistical analysis of the GLWT results indicate that the laboratory asphalt mixes possessed significantly different high-temperature rutting characteristics when compared to each other. Only the control and Wyodak mixes had statistically equal rut measurements.
5. Both the Jim Bridger and Naughton bottom ash mixes rutted significantly more than the control and Wyodak mixes. The Naughton asphalt mix resisted rutting significantly better than the Jim Bridger mix.
6. TSRST tests show that the control and Wyodak bottom ash asphalt mixes used to construct the test road do not possess significantly different low-temperature cracking characteristics.
7. The analysis of TSRST results indicate that the laboratory mixes possessed significantly different low-temperature cracking characteristics when compared to each other. Further analysis comparing the individual mixes showed that the Naughton bottom ash asphalt mix had a significantly higher fracture temperature than the control and Wyodak mixes. The Jim Bridger mix did not display any significant fracture temperature differences when compared to the other three mixes.
8. The asphalt mixes containing bottom ash from different power plants had significantly different low temperature cracking and high temperature rutting characteristics. Of the bottom ash asphalt mixes, the Wyodak mix performed better than the Naughton and Jim Bridger mix, and the Naughton mix performed better than the Jim Bridger Mix.

Recommendations

1. Additional laboratory testing should be performed to determine the optimum mixture of bottom ash and natural aggregates. Testing also should be performed in an attempt to improve performance of bottom ash mixes using other aggregate sources.
2. Initial evaluations of the field test sections have shown no distresses. These test sections should be monitored on a regular basis to determine if future performance differences will develop between the control and Wyodak bottom ash asphalt mixes.
3. Further research should be performed to study the differences in bottom ash properties obtained from different power plants and how these properties effect the performance of bottom ash asphalt mixes. In addition, the consistency of the bottom ash produced by a single plant should also be evaluated.
4. This research project has determined that asphalt mixes containing bottom ash perform well enough to be considered for additional use. Given the benefits that may be realized through the use of bottom ash in asphalt mixes, further investigation into this possibility is justified.

REFERENCES

Ahmed, Imtiaz, and C.W. Lovell. (1992). Use of Waste Materials in Highway Construction: State of the Practice and Evaluation of the Selected Waste Products. Transportation Research Record No. 1345. Washington DC: National Academy Press.

Amaya, P.J. (1997). Boiler Slag and Bottom Ash in Asphalt Concrete. Proceedings: 12th international Symposium on Coal Combustion By-Products (CCB) Management and use. Volume 1. Alexandria, Virginia. American Coal Ash Association.

American Association of State Highway and Transportation Officials. (1991). Hot-Mix Asphalt Paving Handbook. Washington, D.C.: AASHTO.

American Coal Ash Associaiation (ACAA). (1998). 1997 Coal Combustion Product (CCP) – Production and Use. Alexandria, Virginia. ACAA Web Site: Http://www.acaa-usa.org.

Asphalt Institute. (1995a). Performance Graded Asphalt Binder Specifications and Testing. Superpave Series No. 1 (SP-1). Lexington, Kentucky.

Asphalt Institute. (1995b). Superpave Level 1 Mix Design. Superpave Series No. 2 (SP-2). Lexington, Kentucky.

Asphalt Institute. (1983). Principles of Construction of Hot-Mix Asphalt Pavements. (MS-22). Lexington, Kentucky.

Cominsky, Ronald J. (1994). The Superpave Mix Design Manual for New Construction and Overlays. Report No. SHRP-A-407. Washington D.C.: National Academy of Sciences.

Collins, Ronald, Donald Watson, and Bruce Campbell. (1995). Development and Use of Georgia Loaded Wheel Tester. Transportation Research Record No. 1492. Washington D.C.: National Academy Press.

Collins, Ronald, Haroon Shami, and James S. Lai. (1996). Use of Georgia Loaded Wheel Tester to Evaluate Rutting of Asphalt Samples prepared by Superpave Gyratory Compactor. Transportation Research Record No. 1545. Washington DC: National Academy Press.

Department of Energy (DOE). (1998). Table 69. Coal Consumption at Electric Utility Plants by Census Division and State, 1988, 1993-1997. DOE Web Site: http://www.eia.doe.gov.

Erickson, Ryan Joy. (1997). Evaluation of Low Temperature Cracking in Asphalt Pavement Mixes. M.S. Laramie, Wyoming: University of Wyoming Department of Civil Engineering.

Evans, Larry D. (1995). State Soilid Waist Regulations Governing the Use of Coal Combustion Byproducts (CCBs). Prepared for: American Coal Ash Association. Alexandria, VA.

Harrigan, E.T., R.B. Leahy, and J.S. Youtcheff (1994). The SUPERPAVE Mix Design System Manual of Specifications, Test Methods, and Practices. Report No. SHRP-A-379. Washington D.C.: National Academy of Sciences.

Haas, Ralph, Ronald W. Hudson, and John Zaniewski. (1994). Modern Pavement Management. Malabar, Florida: Krieger Publishing Company.

Huang, Wei-Hsing. (1990). The Use of Bottom Ash In Highway Embankments, Subgrades, and Subbases. West Lafayette, Indiana. Purdue University School of Civil Engineering.

Huang, Y.H. (1993). Pavement Analysis and Design. Englewood Cliffs, New Jersey: Prentice Hall, Inc.

Hunsucker, David Q. (1992) Design and Performance of a Bituminous Surface Mixture Containing Bottom Ash Aggregate. Research Report KTC 92-14. Lexington, Kentucky. Kentucky Transportation Center.

Jung, D.H., and T.S. Vinson. (1994a). Low-Temperature Cracking: Binder Validation. Report No. SHRP-A-399. Washington D.C.: National Academy of Sciences.

Jung, D.H., and T.S. Vinson. (1994b). Low-Temperature Cracking: Test Selection. Report No. SHRP-A-400. Washington D.C.: National Academy of Sciences.

Kanerva, Hannele K., Ted S. Vinson, and Huayang Zeng. (1994). Low-Temperature Cracking: Field Validation of the Thermal Stress Restrained Specimen Test. Washington D.C.: National Academy of Sciences.

Lafarge Coperation. Ten Years. Hundreds of Storms. Thousands of Trucks. Zero Failures. Report received from PacifiCorp's Wyodak Plant, January 1997.

Little, D.N., J.W. Button, R.M. White, E.K. Ensley, Y. Kim, and S.J. Ahmed. (1987). Investigation of Asphalt Additives. Office of Engineering and Highway Operations. Report No. FHWA/RD-87/001.

Lovell, C.W., Te_Chih Ke, Wei-Hsing Huang, and J.E. Lovell. (1991). Bottom Ash as a Highway Material. Transportation Research Record 1310. Washington D.C.: National Academy Press.

Miller, Tyler R. (1995). Laboratory Evaluation of Rutting in asphalt Pavements. M.S. Laramie, Wyoming: University of Wyoming Department of Civil Engineering.

Neter, John, Michael H. Kutner, Christopher J. Nachtsheim, and William Wasserman. (1996). Applied Linear Regression Models. 3rd ed. Irwin.

OEM, Inc. (1995). Thermal Stress Restrained Specimen Test User's Manual. Corvallis, Oregon.

Ormsby, W.C., and D.G. Fohs. (1990). Use of Waist and By-Products in Highway Construction. Transportation Research Record No. 1288. Washington DC: National Academy Press.

PacifiCorp. (1998). PacifiCorp Thermal Generation Operations. PacifiCorp Web Site: http://www.pacificorp.com/about/gen/therm.html

Peurifoy, Robert L., William B. Ledbetter, and Clifford J. Schexnayder. (1996). Construction Planning, Equipment, and Methods. 5th ed. McGraw-Hill.

Roberts, Freddy l., Prithvi S. Kandhal, E. Ray Brown, Dah-Yinn Lee, and Thomas W. Kennedy. (1991). Hot Mix Asphalt Materials, Mixture, Design, and Construction. 1st ed. Lanham, Maryland: NAPA Education Foundation.

Root, Richard E., and Ellis G Williams. (1976). Ashphalt – West Virginia Turns Waist Material into Useful Aggregate. Asphalt Institute. Reprint by National Ash Association.

Saylak, Don, Cindy k. Estakhri, and Harshavardhan Chimakurthy. (1997). The Use of Sulfur-Modified Bottom Ash as an Aggregate in Asphaltic Concrete Mixtures. Proceedings: 12th international Symposium on Coal Combustion By-Products (CCB) Management and use. Volume 3. Orlando, Florida. American Coal Ash Association.

Scherocman, James A. (1991). International State-of-the-Art Colloquium on Low-Temperature Asphalt Pavement Cracking. Special Report 91-5. United States Army Cold Regions Research and Engineering Laboratory.

Shahin, M.Y. (1994) Pavement Management for Airports, Roads, and Parking Lots. New York, New York: Chapman & Hall.

Wright, Paul H., and Paquette J. Radnor. (1987). Highway Engineering. 5th ed. New York, New York: John Wiley & Sons.

Wyoming Department of Transportation. (1996). Standard Specifications for Roads & Bridge Construction. 1996 ed. Cheyenne, Wyoming.