EVALUATION OF LOW TEMPERATURE CRACKING IN ASPHALT PAVEMENT MIXES

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contents or use thereof.

Preface

This report examines feasibility of using the thermal stress restrained specimen test to evaluate low temperature cracking in asphalt pavement mixes. Data were collected from laboratory and field evaluations. Various mixing, aging, and compaction methods were used to prepare test samples with materials obtained from two WYDOT highway projects.

Field data were obtained from two recently built test sections and compared with laboratory test results. Pavement condition surveys quantified low temperature cracking of both test sections after one winter. Temperature data for the project sites also were collected. Pavement condition and temperature data were compared to results from the thermal stress restrained specimen test.

The thermal stress restrained specimen test was effective in testing asphalt pavement mixes. However, test results indicated that lab prepared samples did not closely simulate field samples. Also comparisons of lab results with field conditions were performed although it is recommended to perform a more comprehensive analysis after test sections have been in service for a few years.

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INTRODUCTION

BACKGROUND

Low-temperature thermal cracking in asphalt pavements is a problem where extremely cold weather occurs. When temperatures dip well below freezing, pavements tend to shrink. As this shrinking occurs, stresses build in the pavement since it cannot shrink along the length of the roadway. When tensile stresses reach the tensile strength of pavements, pavements pull apart and cracks form. Thermal cracks tend to be in the transverse direction across the road and can occur at fairly regular spacings. Daily temperature cycles also can propagate thermal cracking. Repeated heating and cooling will drive a crack across the road and down through the pavement structure. A major drop in temperature over a short period of time also can cause thermal cracking, even if the temperatures aren't extremely cold. Low-temperature cracking occur in pavements regardless of traffic volumes or loads because they are caused by environmental, not traffic, conditions. Cracks can result in a bumpy and noisy ride as edges of the cracks push up or sink down and potholes can form as pavement deteriorates from traffic. Cracks also allow water into pavement structures which can cause problems such as loss of fines or reduced subbase strength. Each of the problems can affect rideability and reduce pavement service life.

Using softer asphalts in pavement mixes can reduce thermal cracking, however this solution results in softer pavements that are more susceptible to rutting. Many laboratory tests have been developed to determine low-temperature properties of the asphalt itself. Other tests have been developed to evaluate low-temperature cracking of mixes, but most do not relate directly to field conditions [Jung and Vinson, SHRP-A-400, 1994]. It is essential that any lab test should be correlated to field conditions.

PROBLEM STATEMENT

Current asphalt cement mix design procedures including Marshall and Hveem do not evaluate low-temperature properties of asphalt mixes. However, the new mix design procedure developed by the Strategic Highway Research Program (SHRP) has incorporated tests that characterize mixes based on anticipated field performance. Accelerated tests that simulate field conditions are being developed to determine how an asphalt mix will perform before it is placed and will allow agencies to select optimum mix designs that will perform as expected. This procedure should help to eliminate poor performing pavements, and save time and money. An accelerated test to determine low-temperature properties of a mix would allow state agencies to see how a pavement will perform in cold locations before it is built. Because of the cold climate of Wyoming, virtually all roads in the state are subjected to low-temperature cracking. While it may not be possible to eliminate thermal cracking due to frigid winter temperatures, it is important to the Wyoming Department of Transportation (WYDOT) to build pavements that perform well in a low temperature environment. The main objective of this study was to determine feasibility of using the thermal stress restrained specimen test (TSRST) to predict low-temperature properties of asphalt mixes to reduce thermal cracking. WYDOT and other agencies in the cold region may use results from TSRST testing to produce asphalt mixes that are less susceptible to low-temperature cracking.

OBJECTIVES

The main objectives of this study were to:

1. Evaluate characteristics of typical asphalt mixes in Wyoming. This evaluation will help determine if currently used mixes are adequate to resist low temperature cracking. Currently available accelerated laboratory tests such as the thermal stress restrained specimen test and the Georgia loaded wheel test, were used in evaluating asphalt mixes at low and high temperatures.

2. Determine best conditions for preparing samples for laboratory testing to fully simulate field conditions. Sample conditions considered in this study were field slabs, paver mix compacted in the laboratory, mixes prepared and compacted in the lab with various techniques, and mixes aged and then compacted in the lab.
3. Correlate field and laboratory results on the typical mixes included in the experiment. Although it is known that comparing field and lab results requires years of field measurements, this study will provide comparisons after test sections have been in service for one winter. A follow-up study should provide a comprehensive comparison after test sections have been in service for a few years.

ORGANIZATION OF STUDY

Chapter 2 includes a literature review on low-temperature cracking, current asphalt mixes and mix designs, as well as the SHRP mix design procedure. Chapter 3 discusses experiment design, test section selection, and experiments to be performed. Chapter 4 provides information on testing and data collection for both laboratory and field. Results also are presented in this chapter. Chapter 5 contains data analysis and statistical procedures used. Chapter 6 presents a summary of findings and recommendations.

CHAPTER 2

LITERATURE REVIEW

INTRODUCTION

Asphalt mixtures have been used by man for thousands of years. Natural asphalts were used in road surfaces by the ancient Babylonians, Egyptians, Greeks, and Romans. Widespread use of asphalt mixtures as paving materials did not occur until the early 1900s when modern petroleum refining techniques were developed [Asphalt Institute SP-1 (AI SP-1), 1995]. In 1988, there were approximately 6.4 million kilometers of roads in the United States, of which 3.7 million were surfaced with asphalt or concrete. Of that 3.7 million kilometers, about 3.5 million kilometers were surfaced with asphalt mixes [Roberts, Kandhal, Brown, Lee, and Kennedy, 1991]. It is clear from the above numbers that asphalt concrete mixes contribute significantly to the mobility of our society.

CURRENT ASPHALT MIX DESIGN PROCEDURES

Asphalt mixes were developed to provide a stable and inexpensive surface for vehicles. Asphalt concrete or hot mix asphalt (HMA) is made up of various types of asphalt cements and mineral aggregates. The type and quality of asphalt cement or aggregate may change properties of the asphalt mix [Asphalt Institute SP-2 (AI SP-2), 1995]. Objectives of asphalt pavement design and construction are to support traffic loads, protect the base and subbase from moisture, provide a smooth but skid resistant surface, and to resist weathering [Peurifoy, Ledbetter, and Schexnayder, 1996]. The following few sections describe currently used materials and asphalt mix design procedures.

Asphalt Cement

Asphalt cement is the glue that holds aggregate together in an asphalt mix. It also waterproofs the mixture. Aggregate provides a skeleton that gives the mixture strength. Overall properties of the system depend on asphalt cement and aggregate, and their combined reaction [AI SP-2, 1995]. Asphalts used today are either natural or petroleum asphalts. Natural asphalts are relatively soft and can be found at various locations around the world such as Trinidad, Venezuela, and the La Brea “Tar” Pits near Los Angeles, Calif. Petroleum asphalts are obtained by refining crude petroleum and removing lighter fractions such as gasoline, kerosene, diesel, and gas oil. Practically all asphalt used in the United States comes from refineries [Roberts et al., 1991].

Properties of asphalt cement are temperature susceptible, meaning behavior of the material can change with temperature. Asphalt cement is a viscoelastic material because it has viscous and elastic characteristics at a given temperature. At low temperatures, asphalt cement behaves most like an elastic solid, rebounding to its original shape after being loaded and unloaded. At high temperatures, asphalt cement acts more like a viscous liquid. Asphalt cement properties also can change with age of the material through oxidation. As asphalt oxidizes, it becomes more brittle. Oxidation occurs more rapidly at higher temperatures. A considerable amount of aging occurs during HMA production. The material will continue to age throughout the life of the pavement [AI SP-2, 1995].

Since asphalt cement comes from naturally occurring materials, there is great variation in its properties. Attempts have been made to distinguish among asphalts with different properties based on the consistency of the material at a given temperature. Asphalt cements have been classified by penetration, which is a measure of the depth of penetration by a standard needle into asphalt cement at 25˚C at five seconds [Peurifoy et al., 1996]. Viscosity also has been used for classification, which is a measure of the flow of asphalt cement through a viscometer tube at 60˚C and 135˚C. Other information on asphalt characteristics are determined from additional tests related to aging and safety [Roberts et al., 1991].

Since asphalts from different sources have different characteristics, specifications have been developed to identify asphalt characteristics. Asphalt consistency originally was determined by chewing. According to Roberts et al. (1991), this method was used into the late 1800s, when H.C. Bowen invented the Bowen Penetration Machine, however chewing still was used by many to check results of the penetration machine. The Bureau of Public Roads (now the Federal Highway Administration) and the American Society for Testing and Materials (ASTM) modified and standardized the penetration test, which became the main method of measuring asphalt consistency at 25˚C by 1910. A penetration grading system was introduced by the Bureau of Public Roads in 1918 to specify asphalts for different climates of the country. Standard specifications for penetration grading were published by the American Association of State Highway Officials (AASHO) in 1931.

By the early 1960s, a system to specify asphalt by viscosity at 60˚C was introduced by the FHWA, ASTM, AASHTO, and other highway agencies [Roberts et al. (1991)]. This system would be more scientific than empirical and would measure properties at a realistic high pavement temperature. Viscosity grades were developed to specify asphalts for different climates and conditions. Also in the 1960s, the California Department of Highways was developing an asphalt grading system based on viscosity of aged residue (AR) from the rolling thin film oven (RTFO). They believed this would reduce mix setting problems they had experienced in the past due to differences in viscosity after plant mixing.

Aggregates

Aggregate types used in HMA production vary widely. Natural aggregate can be taken from rivers or glacial deposits and used directly in asphalt mix. Processed aggregates that have been quarried, crushed, and separated into distinct sizes also are used in HMA. Synthetic aggregate, such as blast furnace slag, can make use of an industrial by-product that may otherwise be wasted. Another source of aggregate is reclaimed asphalt pavement (RAP) which can be reprocessed into new HMA [AI SP-2, 1995]. Aggregate accounts for 90-95 percent of asphalt mix weight. A proper gradation can be obtained by blending different aggregate sizes and types. Improper gradations may cause problems such as segregation, lack of stability, and lack of tensile strength [Peurifoy et al. 1996]. The acceptable range of gradations for WYDOT is shown below in Table 2.1.

TABLE 2.1 WYDOT Aggregate Gradation Specifications [Wyoming Department of Transportation, 1996]

Sieve Size	Percent Passing for 19 mm (3/4") Max Size	
	Grading A	Grading B
25 mm (1")	100	100
19 mm (3/4")	90 - 100	90 - 100
12.5 mm (1/2")	60 - 85	--
9.50 mm (3/8")	--	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

Aggregate must provide enough shear strength in the mix to resist repeated load applications without showing permanent deformation. Aggregate shape can affect shear strength. Rough textured aggregates can interlock and provide more internal friction than rounded aggregates even though the strength properties of individual pieces may be the same [AI SP-2, 1995]. Aggregate must be tough to resist crushing and disintegration from the time it is produced throughout the pavement life. Tests such as the Los Angeles abrasion test are used to determine toughness and abrasion characteristics of aggregate. Durability and soundness of aggregates indicate how they will resist breakdown due to wetting and drying along with freezing and thawing. Good aggregates also will be free of materials that can weaken HMA, such as vegetation, shale, clay lumps, and excess dust [Roberts et al., 1991].

Asphalt Concrete Mix Design

Roberts et al. (1991) presents an overview of the history of asphalt mix designs. In the late 1800s, asphalt mixes used tar to glue aggregate together and involved no mix design procedure. By the early 1900s, Clifford Richardson had developed procedures to determine if a mix contained the correct amount of asphalt. Richardson's "Pat Test" was used for nearly 20 years on fine-grained mixes. Frederick Warren developed a mix procedure that would incorporate aggregate up to three inches in size called Bitulithic pavement. But with the decline of steel-rimmed tires, large stone mixes were no longer necessary to prevent rutting. Roy Green, an associate professor at the Agricultural and Mechanical College of Texas, developed procedures to obtain a dense graded mix by using ideas from the Bitulithic process. In the mid-1920s Hubbard and Field developed empirical tests to determine optimum asphalt content of fine-graded mixes. This method was modified to work with large stone mixes in the 1950s, but was not widely used due to the popularity of the Marshall method. Francis Hveem developed a mix design method in the 1930s that took aggregate properties into account. He also developed tests to determine rutting characteristics of a mix. Procedures in his mix design continued to change until 1959, and have essentially stayed the same since. The Hveem method has been used by about 25 percent of state highway departments. Bruce Marshall of the Mississippi Highway Department developed a mix design procedure that was studied and further developed by the Corps of Engineers Waterways Experiment Station (WES). WES used characteristics such as asphalt content and density to evaluate mixes that had been compacted with the same compactive effort. These procedures initially were used by WES for airfield pavements, but now are used extensively by highway agencies across the country [Roberts et al., 1991]. Performance-based mix design procedures recently have been developed by the Strategic Highway Research Program (SHRP) that are now being used by some highway agencies. Superpave mix design evaluates how HMA will perform in the field instead of using empirical tests to determine mix characteristics.

SHRP MIX DESIGN

Since the 1940s, most asphalt mixes have been designed using either Marshall or Hveem mix design procedures. This provides the designer with an asphalt content that may be suitable for a given situation. However, these design procedures do not directly deal with properties related to pavement performance. The procedures are based on empirical relationships that may or may not provide adequate information on pavement performance [AI SP-2, 1995].

In 1987, the Strategic Highway Research Program (SHRP) was established by Congress to begin a five-year $150 million program to improve roadways in the United States. The objective was to make roadways safer for motorists and highway workers by improving durability and performance of pavements. Part of this program was to develop pavement specifications based on field performance. This new system was called Superpave, which stands for Superior Performing Asphalt Pavements. The Superpave system incorporates asphalt binder and mineral aggregate specifications, mix design, and prediction of pavement performance. Tests have been designed to determine how asphalt concrete will perform in the field by looking at physical properties that have direct relationships to field performance and by testing at temperatures that pavements will be subjected to in the field [AI SP-2, 1995].

Superpave mix design has three levels, each providing more information on anticipated pavement performance. Level 1 is an improved material selection and mix design process applicable to lower traffic levels. Level 2 expands on Level 1 by providing additional tests to produce performance predictions. Higher traffic levels are appropriate for Level 2 since it has a more reliable level of performance prediction. Level 3 consists of additional tests on a Level 2 design, which will further increase reliability of predicted performance. This added reliability would be necessary to design a mix adequate for high volume roadways [AI SP-2, 1995].

SHRP Binder Specification

Before SHRP, physical properties such as penetration, viscosity, and ductility were used to specify grades of asphalt cement. The properties do not directly relate to the field performance of asphalt. Experience is needed to relate test results to field performance, and relationships used with these methods may not be adequate to predict pavement performance. Asphalts in the same grading may react quite differently to temperature and field conditions [AI SP-1, 1995].

SHRP has developed new binder specifications that will relate asphalt cement grade to field performance. Criteria for specification are constant, but asphalt is graded depending on the temperature at which criteria is met. Tests used to specify asphalt may be related to field performance through engineering principles [AI SP-1, 1995].

Aging of Asphalt Cement

Since asphalt cement performance changes depending on binder age, procedures have been developed to simulate the aging of asphalt throughout its service life. Three critical stages of asphalt aging have been identified. Original binder may be tested to determine ease of handling and transporting. The binder is tested after mixing and construction. Aging that takes place over this period is simulated in the laboratory using a rolling thin film oven (RTFO). Final testing is conducted after service life of the pavement. Aging that occurs over life of the asphalt is simulated in a pressure aging vessel (PAV) [AI SP-1, 1995].

SHRP Binder Tests

Superpave binder specifications select binders according to the location where they will be used. Specific physical properties must be met by all binders. They are graded depending on the temperature at which requirements are met. Both high and low temperature requirements are included in the grading of a binder. For example, an asphalt with a grade of PG 52-28 indicates that high temperature requirements were met at 52˚C and low temperature requirements were met at -28˚C. Information used to select asphalt binders are geographical area where the binder will be used, pavement temperatures that will be experienced, and air temperatures at the location which are converted to pavement temperatures [AI SP-2, 1995].

Superpave binder tests are performed on the asphalt at varying degrees of aging. The Dynamic Shear Rheometer (DSR) can be used to test original binder or binder that has been RTFO and/or PAV aged. The DSR measures rheological properties that characterize viscous and elastic behavior of a binder. The complex shear modulus (G*) and phase angle (δ) of an asphalt binder are measured during this test. G* measures resistance to deformation while subjected to pulses of shear stress. This deformation has elastic (recoverable) and viscous (non-recoverable) components. δ is an indicator of how much deformation is elastic and how much is viscous. The tests are performed at intermediate to high temperatures that would be encountered by an asphalt binder [AI SP-1, 1995].

A Rotational Viscometer tests flow characteristics of asphalt cement. This will indicate the ease at which binder can be pumped and handled. A cylindrical spindle is submerged in an asphalt binder sample in a thermo-container, which keeps the sample at a constant desired temperature. Torque required to maintain a constant rotational spindle speed is measured by the viscometer, which automatically calculates sample viscosity. Since this test is performed to ensure pumpability of asphalt, original or “tank” binder is used in this test [AI SP-1, 1995].

The Bending Beam Rheometer (BBR) is used to measure properties of asphalt cement at low temperatures. Test temperatures simulate the lowest service temperatures of asphalt, which provides information on asphalt stiffness. Samples at low temperatures are too stiff to be tested by the DSR. By using the BBR and DSR, stiffness behavior of an asphalt cement can be determined over a wide range of temperatures. Materials tested in the BBR have been aged in the RTFO and PAV to simulate asphalt that has been subjected to plant mixing and some in-service aging. In this test, a small asphalt beam is placed on simple supports in a constant temperature bath. A blunt-nosed shaft applies a load to the middle of the beam, while load applied and beam deflection are recorded by a computer over a four-minute period. Computer software calculates the creep stiffness and creep rate of the sample, which then are compared to specifications set forth by Superpave [AI SP-1, 1995].

Strain and strength properties of binder at low temperatures can be found using the Direct Tension Tester (DTT). Some asphalts at low temperatures will stretch considerably before breaking and are called “ductile,” while others will break after minimal stretching and are called “brittle.” Some stiff but ductile binders cannot be tested adequately by the BBR and must be subjected to additional testing in the DTT. The DTT test is performed after RTFO and PAV aging, and at temperatures where binder has brittle behavior, typically between 0˚C and -36˚C. Results of the DTT will determine whether an asphalt will behave in a brittle or ductile manner at low temperatures [AI SP-1, 1995].

SHRP Aggregate Selection

There is wide agreement that aggregate characteristics are crucial for HMA to perform adequately. These characteristics are referred to as “consensus properties” due to wide acceptance of their use. Values used for the properties depend on traffic levels that a pavement will be exposed to and position of a pavement level in the pavement structure [AI SP-2, 1995]. Consensus properties consist of coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and clay content. Coarse aggregate angularity ensures a high degree of internal friction in the coarse aggregates to resist rutting. Fine aggregate angularity ensures a high degree of internal friction in the finer aggregates. The flat and elongated particles test determines percentage of aggregate that has a maximum to minimum dimension greater than five.This indicates an aggregate that may break during construction or during life of the pavement. Clay content is the percent of clay present in fine aggregate smaller than 4.75 mm. Excessive amounts of clay in the fines can result in reduced mix performance [AI SP-2, 1995].

Other aggregate properties also can impact HMA quality, but critical values could not be determined since they change depending on material source. These characteristics are referred to as “source properties” and include toughness, soundness, and deleterious materials. Toughness is the percent loss of aggregate during the Los Angeles Abrasion test, which indicates if an aggregate will degrade during handling and construction or during service life. Soundness looks at aggregate loss after repeated immersions in a sodium or magnesium sulfate solution followed by oven drying. Rehydration of salts that find their way into void spaces act to simulate forces caused by freezing water. The soundness test determines aggregate resistance to in-service weathering. Deleterious materials, such as clay lumps, shale, wood, mica, and coal, can reduce HMA quality. Presence of the materials in aggregate is determined by wet sieving. Acceptable values vary depending on the type of contaminant present [AI SP-2, 1995].

Gradations used for Superpave mix designs must fall in specifications. A 0.45 power gradation chart is used to specify gradations. Actual gradations must fall between control points on the chart, and also must avoid a restricted zone in the fine area as shown in Figure 2.1. By keeping the gradation out of this restricted zone, over-sanded mixtures are avoided are gradations following the maximum density curve [AI SP-2, 1995].

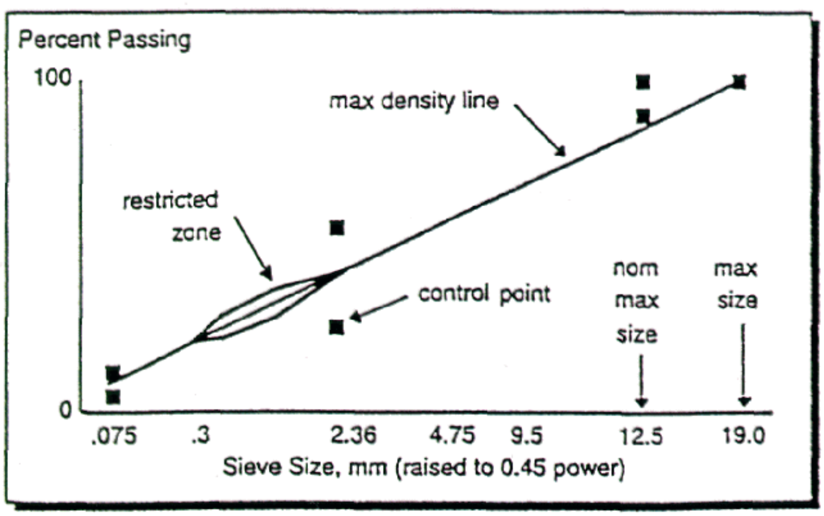


Figure 2.1 Superpave Gradation Limits [AI SP-2, 1995]

Asphalt Mixture Volumetrics

Volumetric proportions of asphalt cement and aggregate in an HMA determine how a pavement will perform during its service life. Volumetric properties of interest in a compacted mixture are air voids, voids in the mineral aggregate, voids filled with asphalt, and effective asphalt content. These properties are important to designing quality HMA, and were incorporated into Superpave [AI SP-2, 1995].

Samples are compacted using the Superpave Gyratory Compactor (SGC). This compactor simulates compaction achieved in the field. A 600 kPa load is applied to asphalt mix in a mold, and the mold is tilted 1.25 degrees and gyrated at 30 revolutions per minute. Superpave has determined the number of gyrations needed to compact a sample for a given temperature range and traffic level. Samples six inches or 150 millimeters in diameter generally are used. Samples are produced at several asphalt contents to determine optimum asphalt content to be used in a mix design [AI SP-2, 1995].

ENVIRONMENTAL CONDITIONS

The environment in which an asphalt pavement is placed is one of the most important factors affecting its performance. Water in the pavement system is a major cause of failure, whether it is in the subgrade, base, or asphalt concrete layer. Water may cause problems such as frost heaves, loss of stability during spring thaw, and a weak subgrade. These problems also depend on temperatures, soil types, pavement types, and traffic conditions. Water may enter a pavement system through various ways such as cracks in the pavement surface, permeable surfaces, pavement edges, lateral movement from shoulders, percolating water, high water table, and liquid and vapor movement from the water table [Yoder and Witczak, 1975].

Air temperature also may cause distress in asphalt pavements. Extremely low temperatures can cause low temperature cracking. In some locations, low temperature cracking is the primary pavement distress [Aschenbrener, 1995]. Cyclical loading caused by daily temperature variations can cause and enlarge cracks. In some cases where extremely low temperatures are not experienced, a high rate of temperature change may cause cracking [Scherocman, 1991]. High temperatures also can cause pavement distress as HMA is more likely to rut due to loading at high temperatures. Distresses mainly are due to temperature dependant characteristics of asphalt cement, which has a lower viscosity and strength at higher temperatures. If heavy loads are applied when pavement temperatures are high, rutting may occur.

A combination of low-temperature cracks and water may lead to more problems. Water entering a pavement system through cracks may freeze and form ice lenses, which can push the crack edge upward. During winter months de-icing material can infiltrate through pavements and thaw base materials, causing depressions to form. Fine materials mixed with water can pump through cracks, creating voids below the pavement, which also causes depressions to form. These problems may reduce rideability and service life of a pavement [Jung and Vinson, 1994b].

LOW TEMPERATURE CRACKING IN WYOMING

Pavements in Wyoming are subject to extremely cold temperatures every winter. Factors contributing to low temperatures in Wyoming are high elevations, distance from moderating oceans, and a northern latitude. The average elevation of the state is about 2,040 meters above sea level. Virtually all temperature recording stations have seen temperatures of -35˚C or colder. All locations of the state can be subjected to temperatures well below 0˚C on numerous occasions throughout the year, and temperatures as low as -53˚C have been recorded [Martner, 1986]. Due to the extremely frigid temperatures, low-temperature cracking of asphalt pavements is a severe problem throughout Wyoming. Cracks can form during extreme cold or during repeated cycles of heating and cooling. The cracking problem in Wyoming is severe enough that the Wyoming Department of Transportation (WYDOT) Pavement Management System has a pavement condition index that takes only cracking into account.

LITERATURE RESEARCH ON LOW-TEMPERATURE CRACKING

Low temperature cracking has always been a problem in asphalt pavements, and significant research in this area has been conducted since the 1960s. Discussions on the early studies are found in Scherocman (1991). Studies such as Monismith, Secor, and Secor (1965) realized that low temperature cracking characteristics of pavements were not a result of temperature alone, but also were influenced by variations in mixes and climate. Anderson, Shields, and Dacyszyn (1966) described thermal cracking mechanisms such as shrinkage in asphalt pavements and the subgrades due to different temperatures at the surface than in the subgrade. It also was noted that cracking behavior could be correlated with penetration values of asphalt, but there were several exceptions. Hills and Brien (1966) reported that aging that occurs during construction and service life of a pavement will change characteristics of asphalt binder and mix. They also found that binder content had little effect on fracture temperature since the addition of binder increased the coefficient of thermal expansion, but decreased mix stiffness. Hindermann (1966) stated that subgrade and subbase materials can have a major effect on thermal cracking. A northern Minnesota road was observed in this study had cracks that appeared to reflect cracks in the soil, as they could be seen to extend beyond the road surface. Results from Littlefield (1967) and Jones, Darter, and Littlefield (1968) indicate that coefficients of thermal expansion and contraction are different and change with temperature. Three causes of low temperature cracking are presented by Haas and Anderson (1969). First, thermally-induced stresses exceed tensile strength of the pavement. This does not consider stresses caused by traffic. Next, subgrades can crack from freezing and shrinking, and these cracks propagate through the pavement. Finally, freezing and shrinking of the subbase or base can cause cracks to propagate through the pavement. It also was noted that pavements with a high stiffness modulus at low temperatures generally had more cracking.

Much of the research regarding low-temperature cracking in asphalt mixes has been performed in Canada, such as the Ste. Anne Test Road project. Results from this project are presented in Burgess, Kopvillem, and Young (1971). The Ste Anne Test Road was constructed in Manitoba in 1967 so researchers could observe low temperature cracking in the field. Three asphalt binders with different penetration grades were used in the road and the stiffness modulus of each was calculated. Also, thermal contraction coefficients and breaking stresses and strains were determined. Using this information, researchers found the temperature at which low temperature cracking would occur, then compared this prediction with actual results from the test road. It was found that predicted temperatures were consistently lower than actual fracture temperatures in the field. However, researchers concluded that the grade and type of asphalt binder used in a pavement is the most important factor in low temperature cracking. They also noted that initial cracking occurred at the pavement surface when the surface temperature was near the minimum for the day. Other discussions on the Ste. Anne test road are presented in Scherocman (1991) which suggest that there is a range of temperatures at which a pavement will crack, and predicting one temperature may not be correct. It was noted that pavements constructed on sandy subgrade material had significantly more cracking than those placed on clay subgrade soil. However, this difference was only noticeable when the binder used was susceptible to thermal cracking.

Haas (1973) and Finn, Hair, and Hilliard (1976) suggested that specifications be used for asphalt binders using penetration and viscosity that would eliminate asphalts that had poor low temperature performance in the past. A limiting stiffness value compared to some criteria also could be part of the specifications. A model for predicting low temperature cracking was presented by Shahin and McCullough (1974) that included air temperatures and solar radiation, which was used to calculate pavement temperatures. Mix stiffness also was used in the model and predictions for low temperature cracking were developed. Predictions from the model compared favorably to actual cracking that had occurred on test roads in Ontario and Manitoba.

Gaw (1981) states that low temperature cracking is affected by climate, subgrade type, asphalt properties, mix design and properties, pavement design, age of pavement, and traffic. Ruth, Bloy, and Avital (1982) used a computer program to predict low temperature cracking using viscosity, coefficient of thermal contraction, and temperature susceptibility data. Results from this model indicated that predicted cracking temperatures depended mainly on viscosity and temperature susceptibility of the binder. Kallas (1982) states that aggregate type has an effect on fracture strength and that 10-15 percent of the fracture surface area was broken aggregate. The COLD computer program was used to predict fracture temperature with daily air and pavement temperatures, initial temperature gradients, stiffness modulus, tensile strength values, and thermal properties of the asphalt concrete layers as inputs. From the COLD model, it was determined that effects due to aggregate type were small compared to effects due to asphalt viscosity. Anderson, Leung, Poon, and Hadipour (1986) indicate that each asphalt source has its own stress-strain curve and that asphalts that have greater failure strains are more resistant to low temperature cracking.

A statistical analysis is presented in Haas, Meyer, Assaf, and Lee (1987) that includes variables such as minimum temperature, Pen Vis Number (PVN), asphalt layer thickness, coefficient of thermal contraction, base thickness, subbase thickness, road width, overlay age and construction year, asphalt content, consistencies of binder, and stiffnesses and stresses of binder at various temperatures. Using multiple regression models, the best single variable found to explain cracking was minimum temperature. Using a two-variable model, minimum temperature and PVN were the two best variables. The best three-variable model used minimum temperature, PVN, and coefficient of thermal contraction. The model with the highest correlation coefficient of $R^2 = 0.70$ was a four-variable model involving minimum temperature, PVN, coefficient of thermal contraction, and pavement layer thickness.

Ideas presented at a colloquium on low temperature cracking are given in Scherocman (1991). According to this report, many factors have been tied to low-temperature cracking, such as pavement age, granular base layers, degree-days of temperature below freezing, rate of change of temperature, and pavement layer thickness. However the most significant factor regarding low-temperature cracking has been found to be stiffness of an asphalt mixture. Methods of how to evaluate stiffness have been subject to disagreement. Whether or not to test asphalt binder alone or to only test mixes has been debated, along with what tests to perform on the materials.

The use of polymer modified asphalt has been found by some to significantly improve thermal cracking performance. Other factors, such as use of lime and aging of the HMA also have been found to have slight effects on the low temperature properties of the HMA [Aschenbrener, 1995]. Low temperature cracking occurs after the binder has aged. This is because the stiffness of a mix will have an effect on thermal cracking. While joints placed in portland concrete control cracking, this in not necessarily the case with asphalt concrete. When a new asphalt road in Manitoba was sawed at 6-meter intervals to provide joints, additional cracks formed between the joints [Scherocman, 1991]. It also was noted that cracks in existing pavement layers would most likely reflect through new overlays, and rehabilitation prior to constructing the overlay is necessary for reflective crack prevention [Aschenbrener, 1995].

THERMAL STRESS RESTRAINED SPECIMEN TEST

Low temperature cracking is a serious problem in portions of the northern United States, Alaska, Canada, and other locations that experience severely cold weather. To better understand the problem of low temperature cracking and how to best address it, a research program was instigated under SHRP contract A-003A. Part of this contract was to conduct an experimental program with the thermal stress restrained specimen test (TSRST) to evaluate low temperature cracking of asphalt mixes [Jung and Vinson, 1994b]. Many tests have been developed to observe thermal cracking in asphalt mixes, but the TSRST has shown the greatest potential to evaluate temperature cracking susceptibility because it simulates field conditions, is easy to use, and can accommodate large stone mixes [Vinson, Janoo, and Haas, 1990].

The thermal stress restrained specimen test device is comprised of systems controlling load, data acquisition, and temperature. Different components of the TSRST are shown in Figure 2.2. The load system consists of a load frame, a step motor, and a swivel connection system. A step motor is mounted on top of the load frame and a load cell is connected to the bottom. Swivels connect the specimen assembly to the step motor and load cell through plastic composite blocks that provide a thermal barrier [OEM, 1995]. The step motor keeps the specimen at a constant length throughout the test by using linear variable differential transformers (LVDTs). LVDTs are attached to the specimen assembly to detect

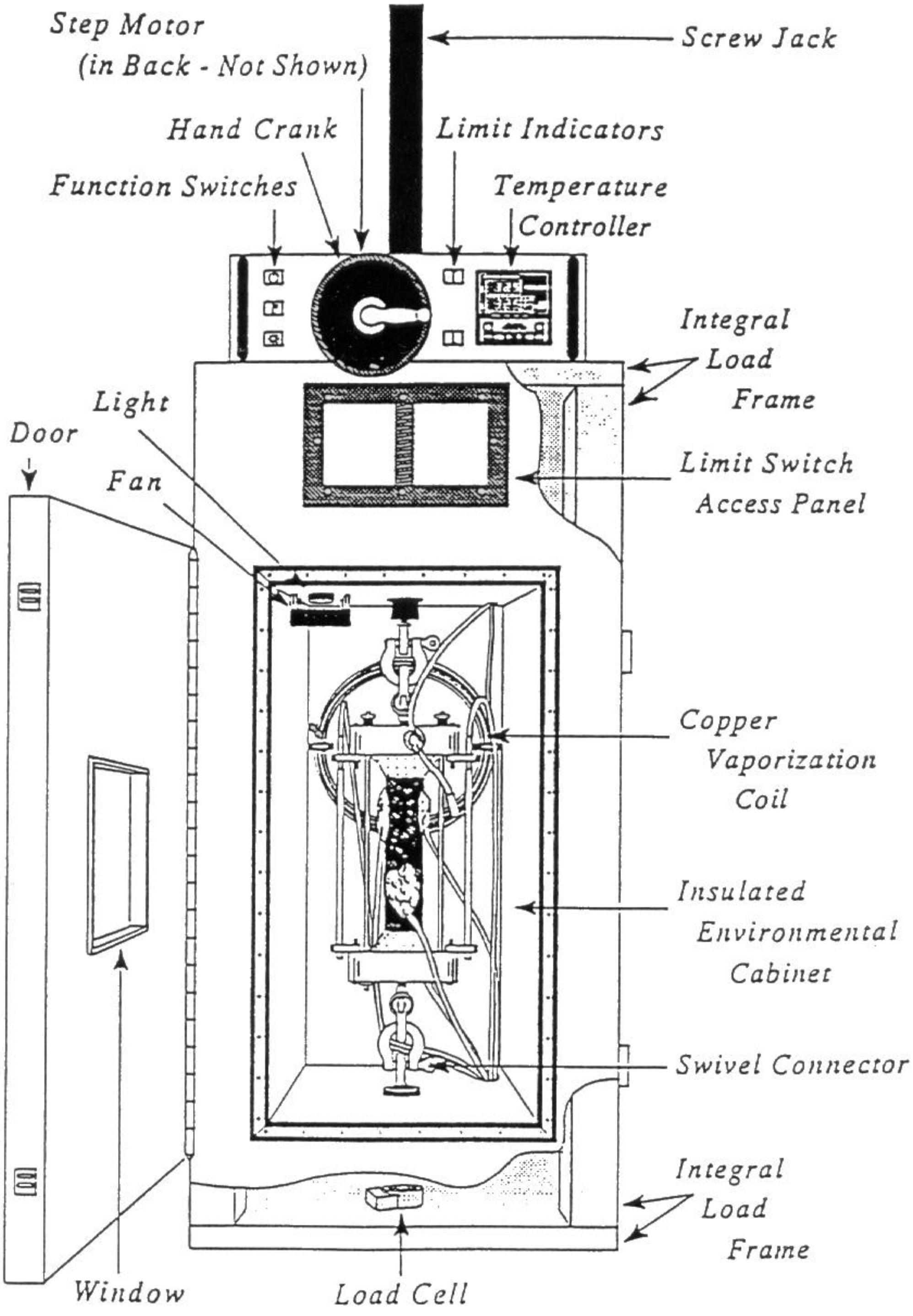

Step Motor
(in Back - Not Shown)
Screw Jack
Hand Crank
Limit Indicators
Function Switches
Temperature
Controller
Integral
Load
Frame
Light
Door
Fan
Limit Switch
Access Panel
Copper
Vaporization
Coil
Insulated
Environmental
Cabinet
Swivel Connector
Integral
Load
Frame
Window
Load Cell

Figure 2.2 TSRST Equipment Components [OEM, 1995]

changes in specimen length. A computer then prompts the step motor to pull the specimen back to its original length, which builds tensile stress in the specimen [Jung and Vinson, 1994b].

The temperature control system includes an environmental cabinet, a tank of liquid nitrogen (LN_2), a programmable temperature controller connected to a solenoid valve, a copper coil, a fan, and a resistance temperature device (RTD). The system cools as liquid nitrogen is vaporized through copper coils into the environmental cabinet. The temperature controller is programmed to cool at a specified rate, and controls the release of liquid nitrogen through the solenoid valve into the environmental cabinet. An RTD measures temperature inside the cabinet so the controller will know when to cool. A fan circulates air inside the cabinet to create a relatively even temperature distribution [Jung and Vinson, 1994b].

A data acquisition system records data such as temperatures from RTDs, load from the load cell, and change in specimen length from LVDTs. This information is used to send instructions to the step motor and for test data analysis. A computer logs data at a specified interval throughout testing, and computes parameters such as average temperature and tensile stress. The data acquisition system is controlled through a TSRST software package [OEM, 1995].

Various specimen sizes have been tested in the TSRST, with cross-sectional areas ranging from 625 mm^2 to 5,776 mm^2. Aspect (length/width) ratios have ranged from 4 to 20 [Jung and Vinson, 1994b]. Based on previous research, a cross-section of at least 2,600 mm^2 should be used [Janoo, Bayer, Vinson, and Haas, 1990]. Cooling rates used in tests have ranged from 3 to 30˚C/hr [Jung and Vinson, 1994b]. However actual cooling rates in the field have been found to be between 0.5 and 1.0˚C/hr [Janoo et al., 1990], and cooling rates in Canada seldom exceed 2.7˚C/hr [Fromm and Phang, 1972]. Most users of the TSRST have used a rate of 10˚C/hr to perform tests in a reasonable amount of time [Jung and Vinson, 1994b].

Specimens are cemented to aluminum end platens by the use of epoxy. A fillet of epoxy is created along the sides of the specimen to ensure an adequate bond between sample and platen. The epoxy is allowed to cure while the specimen and platens are attached to an alignment stand so the specimen will be correctly aligned. Before testing, spring-loaded alignment rods are attached through holes in the platens to compensate for weight of the hanging specimen assembly. Invar rods also are attached along with LVDT holders. The LVDTs rest on Invar rods and monitor the length of specimen. Swivel attachments are connected to both ends of the assembly and the specimen is hung in the environmental cabinet. The specimen may be precooled before insertion into the environmental cabinet or precooling may be completed within the cabinet. After securing the specimen in the cabinet, four platinum RTDs are attached around the specimen to record temperature data. The LVDTs are placed in their holders and the temperature control RTD is attached to the top platen so that it is suspended below the platen [OEM, 1995]. The specimen is then ready for precooling or, if already precooled, the actual test.

During the thermal stress restrained specimen test, the temperature in the environmental cabinet is dropped at a constant rate of 10˚C/hr. The specimen contracts as it cools, but the step motor pulls the specimen back to its original length as determined by LVDTs. As the step motor pulls, tensile stresses built within the specimen, until tensile stresses exceed the tensile strength of the material and the specimen breaks.

EFFECTS OF AGING ON LOW TEMPERATURE CRACKING

As the age of a pavement increases, so does the incidence of thermal cracking because asphalt cement becomes more brittle as it ages. This occurs as organic molecules in asphalt react with oxygen over the service life of the pavement. This oxidation changes the structure and composition of the molecules, making them more brittle and more subject to cracking. Another form of aging occurs during mixing and construction when asphalt cement is heated to high temperatures. This allows the volatile components of the cement to evaporate, which creates a stiffer asphalt [AI SP-1, 1995].

In previous TSRST results, fracture temperatures have increased along with degree of aging. Samples subjected to long term aging would break at warmer temperatures than those that had been short term aged [Jung and Vinson, 1994a].

CHAPTER SUMMARY

This chapter presented an overview of asphalt mix components and design, including the new SHRP Superpave mix design. Environmental conditions that affect asphalt pavements were covered. Past research studies on low temperature cracking and development of the thermal stress restrained specimen test were presented. The effects of aging on low temperature cracking also was considered. This information is important in developing the experiment design of this study to evaluate low temperature cracking of asphalt mixtures using the thermal stress restrained specimen test.

CHAPTER 3

DESIGN OF EXPERIMENT

INTRODUCTION

The main objective of this study was to evaluate low temperature cracking of typical asphalt mixes in Wyoming. To achieve this, the thermal stress restrained specimen test (TSRST) device was selected to evaluate low temperature cracking. In addition, the Georgia loaded wheel tester was used to evaluate rut resistance of asphalt mixes. Two newly-constructed interstate jobs were selected for inclusion in the experiment — Point of Rocks-West IM-80-3(121)120 on Interstate 80 and Kingsbury Road IM-90-(69)101 on Interstate 90. The projects were constructed in two different portions of Wyoming during the summer of 1996. This chapter summarizes the overall design of experiment for this research study and includes details about the asphalt mixes used in both jobs.

POINT OF ROCKS PROJECT

As shown in Figure 3.1, the Point of Rocks project is located approximately 30 kilometers east of Rock Springs in Sweetwater County on Interstate 80. Interstate 80 is a major east-west route in the United States with an Average Daily Traffic (ADT) of about 10,000 with 45 percnet truck traffic [Wyoming Department of Transportation, 1993]. Granite aggregate obtained from the Forever Pit was used in the asphalt mix along with recycled asphalt pavement (RAP). The gradation consisted of 80 percent virgin aggregate with 20 percent RAP. The virgin aggregate consisted of 55 percent coarse and 45 percent fines. Table 3.1 shows the combined aggregate gradation for this project. Five percent of asphalt cement was used in the mix, including the asphalt from the RAP. This meant that 4 percent of new asphalt cement was added. The new binder used in the mix was Exxon Polymer (Modified) AC-20. One percent hydrated lime also was added. The Marshall mix design for this mix was performed by WYDOT. Table 3.2 shows a mix summary while Appendix A shows mix design details.

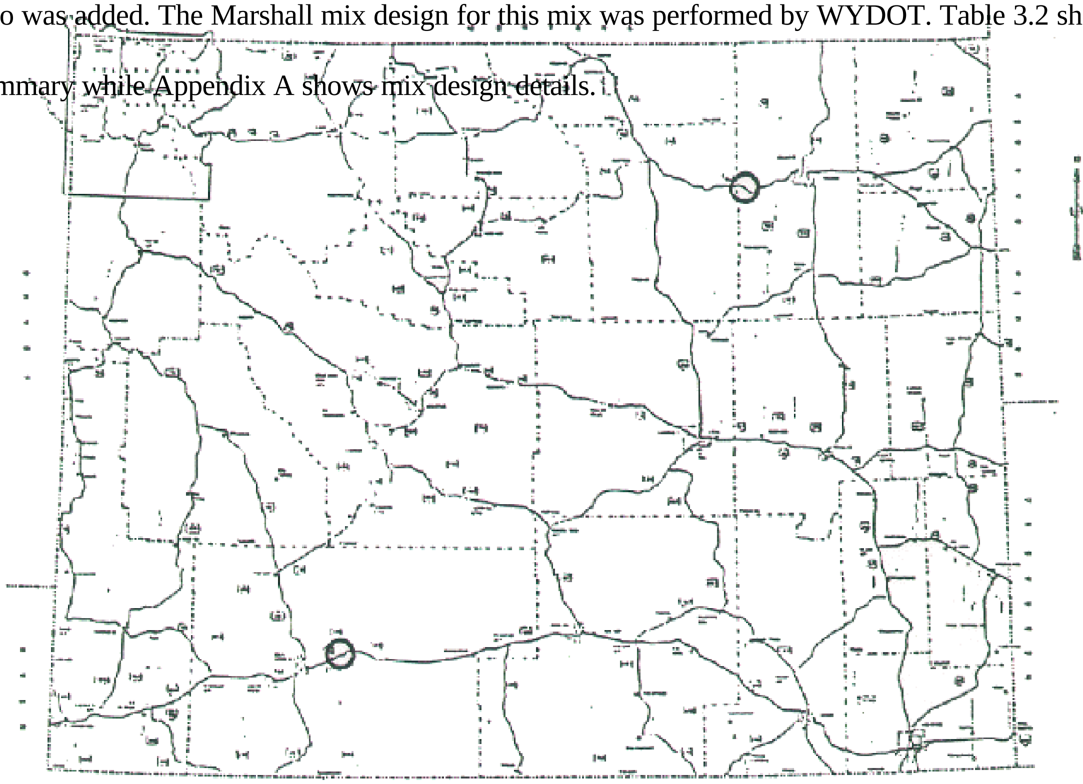


Figure 3.1 Locations of Test Sections

Materials from this project were obtained in June 1996. Adequate samples of the following were collected: coarse and fine aggregates, RAP, asphalt cement, and HMA from the paver. After paving and compaction, two 380 X 380 mm slabs were taken from the roadway near Milepost 121 by using a jackhammer. Figure 3.2 shows one of the slabs. A paving fabric was used under the asphalt layer, which helped in removing the slabs.

TABLE 3.1 Aggregate Gradations for Point of Rocks Asphalt Mix

Sieve	Coarse (+4)	Fines (-4)	RAP Average	Combined
25mm (1")	100		100	100
19mm (3/4")	93		98	97
12.5mm (1/2")	50		93	77
9.5mm (3/8")	30	100	87	67
4.75mm (# 4)	5	98	61	50
2.36mm (# 8)	1	63	43	32
1.18mm (# 16)	1	32	31	18
600μm (# 30)	1	19	24	12
300μm (# 50)	1	13	19	9
150μm (# 100)	1	8	13	6
75μm (# 200)	0.4	3.8	8.4	3.2

TABLE 3.2 Marshall Mix Design Results at Optimum Asphalt Content for Point of Rocks Project

	Point of Rocks Mix
Marshall Blow Count	75
Density at Optimum AC (kg/m³)	2287
Air Voids (%)	5.8
Marshall Stability (kg)	1989
Marshall Flow	10

KINGSBURY ROAD PROJECT

As shown in Figure 3.1, the Kingsbury Road project is located approximately 30 kilometers west of Gillette in Campbell County on Interstate 90 The traffic level is relatively light for this Interstate highway. The ADT in 1993 was 3,720 with 15 percent trucks [Wyoming Department of Transportation, 1993]. Limestone aggregate from the Pete Lien Pit near Sundance was used in this project, along with filler from the Reeves Pit near Buffalo. The aggregate combination consisted of 45 percent coarse, 40 percent fines, and 15 percent filler. Table 3.3 summarizes aggregate gradations for this project. The asphalt content of this mix was 4.9 percent. The binder used on the project was Cenex AC-20. One percent hydrated lime also was added to the mix. WYDOT performed the mix design for this project. The summary of the mix at optimum asphalt content is summarized in Table 3.4 while the whole mix design is shown in Appendix A.

Figure 3.2 Pavement Slab Taken from Point of Rocks Project

Materials for the laboratory testing were obtained in August 1996. Fine and coarse aggregates, and filler were sampled from stockpiles since paving had not yet begun. Asphalt cement was obtained from WYDOT in Cheyenne. A 460 X 460 mm slab of the 100 millimeter lift was taken from the roadway by WYDOT employees after paving, along with core samples obtained with a core drill.

TABLE 3.3 Aggregate Gradations for Kingsbury Road Asphalt Mix

Sieve	Coarse (+4)	Fines (-4)	Filler	Combined
25mm (1")	100			100
19mm (3/4")	95			98
12.5mm (1/2")	51		100	78
9.5mm (3/8")	27	100	100	67
4.75mm (# 4)	3	87	97	51
2.36mm (# 8)	1	57	77	35
1.18mm (# 16)	1	31	53	21
600μm (# 30)	1	20	39	14
300μm (# 50)	1	13	27	10
150μm (# 100)	1	10	17	7
75μm (# 200)	0.7	7	9.1	4.5

TABLE 3.4 Marshall Mix Design Results at Optimum Asphalt Content for Kingsbury Road Project

	Kingsbury Road Mix
Marshall Blow Count	75
Density at Optimum AC (kg/m³)	2424
Air Voids (%)	3.4
Marshall Stability (kg)	1702
Marshall Flow	11

LABORATORY TESTING PROGRAM

After identifying test sections to be included in the experiment, a testing program was developed, which included field and laboratory components. The following section describes the components in detail.

To evaluate the characteristics of asphalt mixes in this experiment, two primary laboratory tests were used. The thermal stress restrained specimen test (TSRST) determined the low-temperature properties of each mix including temperature and tensile stress at fracture due to thermal cracking. While low temperature cracking was the main factor in this study, the Georgia loaded wheel tester (GLWT) also was used to determine the rutting resistance of each mix. The main objective of any pavement engineer is to obtain a balanced mix that offers good resistance to low temperature cracking and rutting. By performing the TSRST and GLWT tests, the performance of asphalt mixes at high and low temperatures could be observed.

Another objective of this study was to evaluate effects of aging on mix performance. Two forms of aging were used in this experiment. Short-Term Oven Aging (STOA) was performed in accordance to the standard test method SHRP M-007, Standard Method of Test for Short- and Long-Term Aging of Bituminous Mixes described in Harrigan, Leahy, and Youtcheff (1994). In this procedure, the asphalt mix is placed in pans directly after mixing and spread out thinly. The pans are then placed in a 135˚C oven for four hours, after which the mix is compacted. STOA is done to simulate aging that takes place while HMA is being mixed at the plant and placed in the field. The second type of aging is Long-Term Oven Aging (LTOA). Samples subjected to LTOA were further aged according to SHRP M-007. In this aging, the compacted samples are to be placed in an 85˚C oven for 120 hours or five days. This is done to simulate aging that takes place over the service life of the pavement.

Samples tested in the TSRST were obtained from four sources: field slabs, uncompacted mix from the paver compacted in the lab, unaged mix that was lab mixed and compacted, and STOA mix that was mixed and compacted in the lab. Most lab compacted samples were compacted at the Colorado Department of Transportation using a linear kneading compactor. Additional samples were compacted at the University of Wyoming using a press. Compaction details can be found in Chapter 4. By using the samples, the difference between lab mixes and field mixes could be observed, along with the effects of aging. A summary presenting the condition of samples tested in the experiment can be found in Table 3.5.

TABLE 3.5 Conditions of Samples Used in Experiment

Sample	Mixing		Compaction			Aging	
	Field	Lab	Field	Lab Rolled	Lab Press	STOA	STOA+ LTOA
Field Cores	X		X				
Paver Mix A	X			X			
Paver Mix B	X				X		
Lab Mix A		X		X			
Lab Mix B		X			X		
Lab Mix C		X		X		X	
Lab Mix D		X			X	X	
Lab Mix E		X		X			X
Lab Mix F		X			X		X

The following mixes were used to make samples for the GLWT: paver mix, unaged lab mix, STOA lab mix, and STOA + LTOA lab mix. The samples were compacted in the gyratory compactor according to the compaction method given in SHRP M-002, Standard Method of Test for Preparation of Compacted Specimens of Modified and Unmodified Hot Mix Asphalt by Means of the SHRP Gyratory compactor, which is found in Harrigan, Leahy, and Youtcheff (1994). Also, field cores from the project were obtained from WYDOT and tested in the GLWT. Information from the tests were compared with results from the TSRST.

FIELD DATA

In the spring of 1997, field data were obtained from both test sections. A pavement condition survey was performed at each site to determine the amount of low-temperature cracking that had occurred over one winter. This was done by randomly selecting at least eight sites along each project and recording the amount, type, and severity of cracking in a measured area of pavement. Also, temperature data from locations near each project were obtained from the Wyoming Water Resource Center located at the University of Wyoming to determine the temperatures that pavements were subjected to during the winter months. Equations from the Asphalt Institute were used to determine pavement temperatures. This data also were compared with findings from laboratory tests to see if the TSRST could be used to predict low-temperature cracking.

DATA SUMMARY AND EVALUATION

Data such as densities, fracture temperatures, and tensile strengths were recorded from TSRST testing along with other data described in this chapter. Rut depths were obtained from GLWT testing. Densities of all samples were evaluated and compared to WYDOT specifications. Statistical analyses were performed on densities, fracture temperatures, and tensile strengths of TSRST samples. Also, the correlation of fracture temperatures to rut depths for the various types of samples was explored. Temperature data were compared to TSRST results and pavement condition surveys to determine if any correlations were evident. The analysis of data from this study was then used to form conclusions and recommendations.

CHAPTER SUMMARY

This chapter has presented the objectives of this low temperature cracking study and how they were achieved through laboratory testing and field evaluations. Descriptions of the Point of Rocks and Kingsbury Road test sections were given, including locations, mix designs, sample collection. Laboratory tests used in this study and field data collected for analysis were included. How data were used and analyzed to form conclusions for this study also were presented.

CHAPTER 4

TESTING AND DATA COLLECTION

INTRODUCTION

Laboratory and field evaluations were performed in this study to observe low temperature cracking characteristics of asphalt mixes. The focus of laboratory testing was on the thermal stress restrained specimen test, which concentrates on temperatures and stresses in asphalt mixes when low temperature thermal cracking occurs. The Georgia loaded wheel tester also was used to examine high temperature characteristics of rutting in pavements. Background, procedures, and results of the tests are presented in this chapter.

Field evaluations were performed on the test section sites so that lab and field performance could be compared. Field data collected included pavement distress surveys, pavement condition index calculations, and field temperature data. Methods of data collection and results are given in this chapter.

THERMAL STRESS RESTRAINED SPECIMEN TEST

Thermal cracking due to low temperatures is a problem in many parts of the world. Researchers have been studying thermal cracking for years, and have tried various methods to evaluate low temperature behavior of asphalt mixes. Data from the evaluations have been used in thermal cracking models developed to predict low-temperature cracking, such as COLD [Finn et al., 1986], University of Florida model [Ruth et al., 1982], Texas A&M model [Lytton et al., 1983], and University of Texas model [Shahin and McCullough, 1972]. Some tests used to provide data for the models include indirect tension test, direct tension test, direct tensile creep test, flexural bending test, thermal stress restrained specimen test, and coefficient of thermal expansion and contraction test. According to Vinson et al. (1990), only the thermal stress restrained specimen test and coefficient of thermal expansion and contraction test simulate actual field conditions and directly measure stress-temperature relationships.

The thermal stress restrained specimen test (TSRST) was first introduced in the 1960s when Monismith et al. (1965) stated that thermal cracking could be simulated in a laboratory. A specimen was attached to a fixed frame to keep the sample length constant during cooling while stress, strength, and temperature data were recorded. Initially the frame was made of Invar steel to reduce change in length of the frame as temperature decreased. However, this fixed frame method was not successful as frame deflections during loading would keep the sample from failing [Kanerva, Vinson, and Zeng, 1994]. To overcome this, Arand (1987) built a displacement feedback loop into the system to constantly correct specimen length during the test. This prevented stress relaxation in the specimen during the test due to a flexing frame and allowed sample failure. Further development of the TSRST has been done at Oregon State University under SHRP contract A-003A and by OEM, Inc. of Corvallis, Oregon. A complete TSRST system is shown in Figure 4.1.

Figure 4.1 Thermal Stress Restrained Specimen Test Apparatus

Test Objectives

The objective of the TSRST is to obtain low-temperature characteristics of asphalt concrete mixes, such as the temperature and stress at which thermal cracking occurs, by subjecting a specimen to an accelerated test that measures thermal cracking performance. The results enable the asphalt mix designer to predict how a mix will perform in the field before paving a road, thus eliminating poor performing pavements that waste valuable tax dollars. Various mixes can be tested in a relatively short time period and with information from other accelerated performance tests, the most superior mix can be determined.

The basis of the thermal stress restrained specimen test is to cool an asphalt concrete specimen at a specified rate, which will cause the specimen to shrink. As shrinking occurs, the specimen is pulled back to its original length by the device, which builds tensile stress in the asphalt concrete. This continues until the tensile stress that has accumulated reaches the tensile strength of the sample and specimen breaks.

Test Samples

In this study, both field and lab samples were tested in the TSRST. Field samples were obtained from slabs taken from the pavements in both test projects. The slabs were cut with a jackhammer in the field after finishing the lay down operation. Later in the lab, slabs were cored and sawed to obtain samples suitable for testing in the TSRST machine. All samples were approximately 23 centimeters long. In addition to the field samples, some samples were compacted in the lab. Initially, a press at the University of Wyoming was used to compact a few 100 X 100 X 360 mm beams. These beams later were cored to obtain TSRST samples. All additional specimens were compacted by the Colorado Department of Transportation (CDOT) in Denver by means of the linear kneading compactor shown in Figure 4.2. The slabs compacted at CDOT were 500 X 180 X 100 mm and are shown in Figure 4.3. Three types of mixes were prepared and tested from each project, with two samples for each type. These mixes were: mix made at the job site and taken from the paver, HMA mixed in the lab and compacted without aging, and a mix that was mixed in the lab and STOA before compaction.

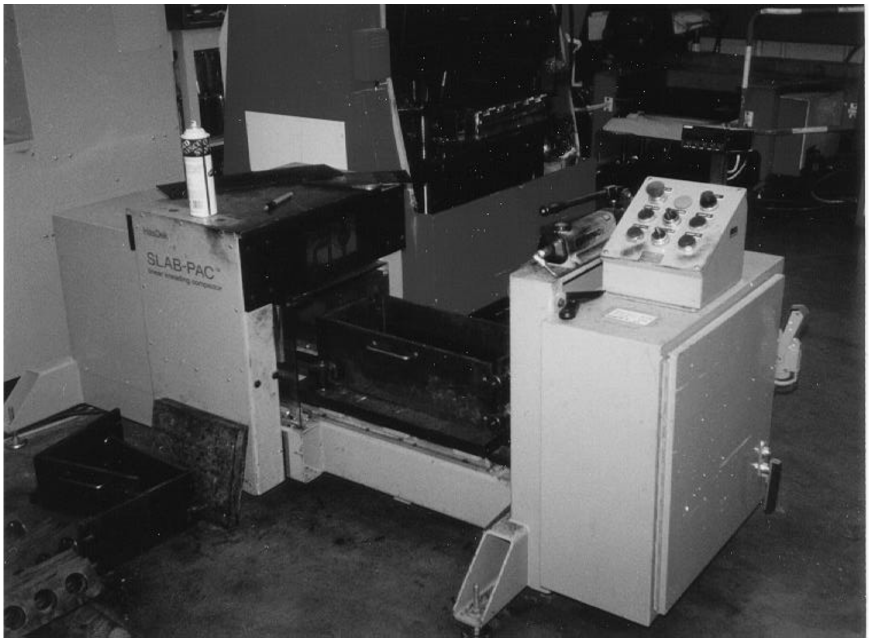


Figure 4.2 Linear Kneading Compactor located at CDOT

Figure 4.3 Beams compacted by CDOT Linear Kneading Compactor

With the exception of field slab samples from the Point of Rocks project, all specimens were 5.08 centimeter cores approximately 23 centimeters long. The Point of Rocks slabs were cut into prisms about 40 X 50 mm in cross section. This was done using a diamond core bit and a diamond saw blade. Densities of the samples were determined prior to testing using Method A of the AASHTO T166-88 procedure, Standard Method of Test for Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens [AASHTO, 1990]. Figures 4.4 and 4.5 show prism and core samples.

Specimen sizes used for testing do not match those set forth in AASHTO TP10, Standard Test Method for Thermal Stress Restrained Specimen Tensile Strength, because the AASHTO standard does not reflect commonly-used procedures for the TSRST. The main with the AASHTO TP10, which is a provisional standard, was with the procedure specified specimen size. A specimen diameter of 63.5 millimeters and a length of 254 millimeters is specified, while 51 millimeter diameter cores currently are being used by CDOT and others and shorter lengths are being used for convenience [Ashenbrener, 1995; Whiting, 1997]. SHRP funded studies such as Jung and Vinson (1993) also were performed using specimens with dimensions smaller than those specified by AASHTO.

Figure 4.4 Core Samples for TSRST

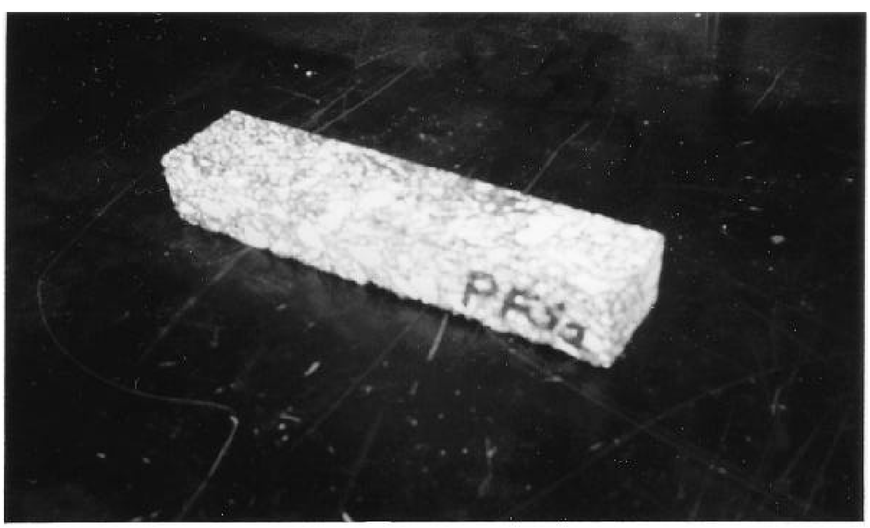


Figure 4.5 Prism Sample for TSRST

TSRST Test Procedures

Test procedures for the thermal stress restrained specimen test (TSRST) consisted of two parts: specimen set up and testing. Procedures suggested by OEM, Inc. were the basis for testing along with AASHTO TP10. Samples were attached to two aluminum platens using a two-part epoxy, Devcon steel filled putty and hardener. This was done in an alignment stand that would keep specimen and platens in proper alignment as shown in Figure 4.6. Poor alignment could result in bending stresses in the sample, which could alter results [Jung and Vinson, 1994b]. Nine parts putty to one part hardener was used to create the epoxy as according to manufacturers directions. Sample alignment was measured using a small steel ruler and adjustments were made accordingly. Holes in the top and bottom platens were aligned with rods. The epoxy was allowed to cure overnight.

Figure 4.6 TSRST Specimen in Alignment Stand

After curing, the specimen and platens were precooled in an environmental chamber. Precooling brings the sample temperature to between 2˚C and 4˚C and takes 30 to 90 minutes [OEM, 1995]. The environmental chamber was used for precooling to save time by reducing precooling time needed in the TSRST machine. This procedure allowed more specimens to be tested per bottle of liquid nitrogen and for precooling of a sample while testing another.

Spring-loaded alignment rods were installed on the assembly, leaving a 2.5 mm gap when the spring was compressed. Invar rods and LVDT holders were attached and aligned, and ball swivel connectors were screwed into each end of the assembly. This assembly was then hung in the environmental cabinet of the TSRST by using the top clevis, and position of the specimen was adjusted with the hand crank so that the bottom clevis could be connected. A gap was left in the bottom clevis so that no tension was applied to the sample before testing began. Next, four platinum RTDs were attached to the specimen using clay. An RTD was placed on each side of the sample, and they were spaced from top to bottom. LVDTs were placed in their holders and adjusted to give a reading near 0.000 mm, and a temperature control RTD was hung from the assembly so it was suspended from the top platen. As shown in Figure 4.7, the setup was now ready for precooling.

After setting the temperature controller according to manufacturers directions, liquid nitrogen was turned on. Specimens were precooled until all four RTDs on the sample had readings between 2 and 4˚C. Data were then entered into the TSRST computer program, such as filename, time interval for data collection, and sample cross-sectional area. After verifying all settings and readings were correct, the temperature controller was set to begin the test temperature ramp. The servo motor was then turned on to allow length correction of the specimen.

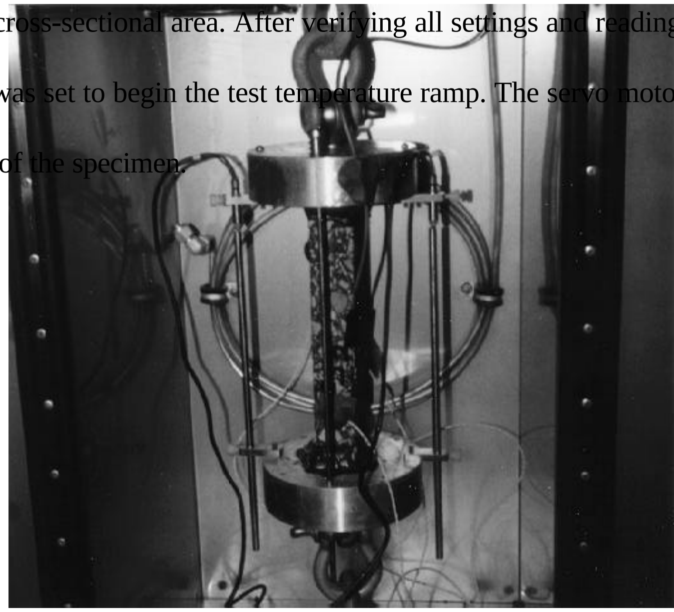


Figure 4.7 Sample Ready for Thermal Stress Restrained Specimen Test

During the test, the temperature controller drops the temperature 10˚C per hour in the environmental cabinet. As temperature drops and the sample shrinks, LVDTs detect a change in length and the step motor pulls the specimen back to its original length. The load cell attached to the bottom clevis in the bottom of the frame indicates tensile load placed on the specimen throughout testing. The data acquisition system scans and records the load, temperatures of the four RTDs, LVDT readings, and test time at specified intervals throughout the test. One-minute intervals were initially used, but the interval was increased to two minutes to reduce the large amount of data recorded. Testing would continue until sample failure, which generally took three or four hours.

According to AASHTO TP10, recorded items include average temperature at failure, load at failure, $\delta S/\delta T$, which is the slope of the tensile stress vs. temperature curve, and time to failure. Ultimate strength of the specimen can be determined from the load at failure and the cross-sectional area. Description of the failure, such as location, shape, and amount of aggregate breakage, were also recorded.

Test Results

The thermal stress restrained specimen test was performed on 23 samples. Eight of the samples were from the I-90 Kingsbury Road project; the other 15 samples were from the I-80 Point of Rocks project. More samples were tested from the I-80 project to determine effectiveness of variable methods for sample preparations. Sample test results are shown in Appendix B and summaries of TSRST test results are shown in Appendix C.

TSRST results from the Kingsbury Road project are shown in Table 4.1. This table summarizes densities of field slab and paver mix samples along with fracture temperatures and tensile strengths. Densities for lab-mixed samples are slightly lower than field samples, with STOA being the lowest. Fracture temperatures of field compacted samples were slightly lower than the lab compacted samples. Tensile strengths had some variations. A broken TSRST sample is shown in Figure 4.8, and a typical graph of temperature versus tensile stress during the test is shown in Figure 4.9.

TABLE 4.1 I-90 Kingsbury Road TSRST Results

<table>
<tr><th colspan="3">Sample Condition</th><th>Density (kg/m³)</th><th>Tensile Strength (kg/cm²)</th><th>Fracture Temperature (˚C)</th><th>Slope dS/dT (kg/m²/ ˚C)</th></tr>
<tr><td rowspan="6">Lab Compacted</td><td rowspan="2">Paver Mix</td><td>1</td><td>2406</td><td>22.5</td><td>-25.8</td><td>133358</td></tr>
<tr><td>2</td><td>2412</td><td>31.7</td><td>-27.8</td><td>17999</td></tr>
<tr><td rowspan="2">Unaged Lab Mix</td><td>1</td><td>---</td><td>17.4</td><td>-26.0</td><td>11390</td></tr>
<tr><td>2</td><td>2364</td><td>25.4</td><td>-24.5</td><td>15538</td></tr>
<tr><td rowspan="2">STOA Lab Mix</td><td>1</td><td>2308</td><td>21.7</td><td>-26.9</td><td>11249</td></tr>
<tr><td>2</td><td>2318</td><td>21.9</td><td>-23.7</td><td>13499</td></tr>
<tr><td rowspan="2">Field Compacted</td><td rowspan="2">Field Slab</td><td>1</td><td>2414</td><td>29.2</td><td>-28.0</td><td>14272</td></tr>
<tr><td>2</td><td>2414</td><td>26.4</td><td>-29.3</td><td>16663</td></tr>
</table>

Figure 4.8 A Broken TSRST Sample

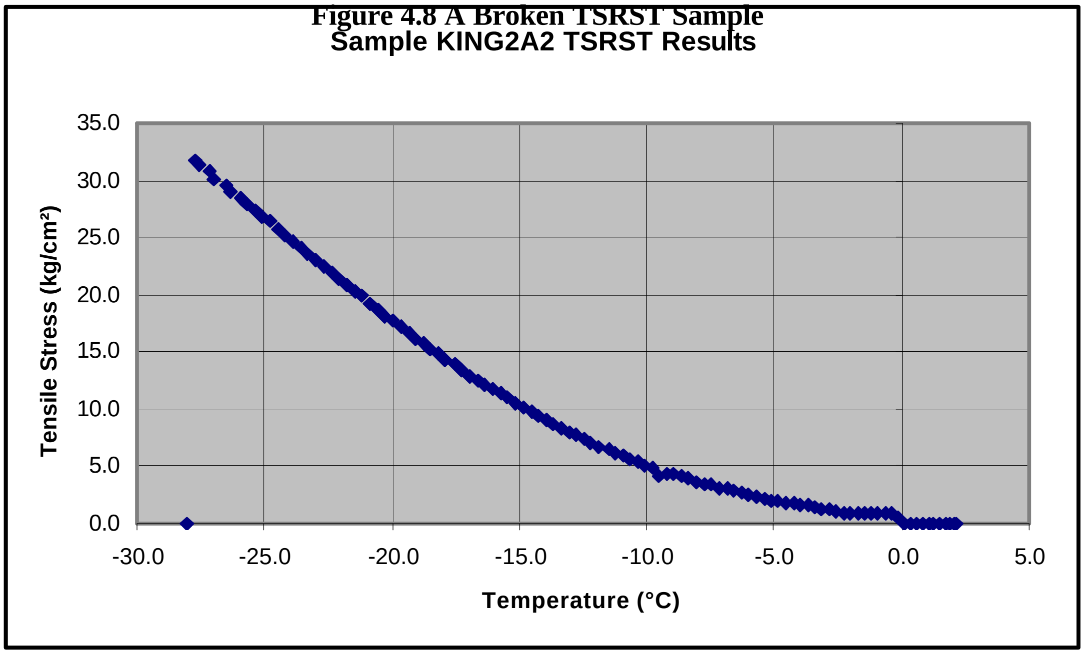
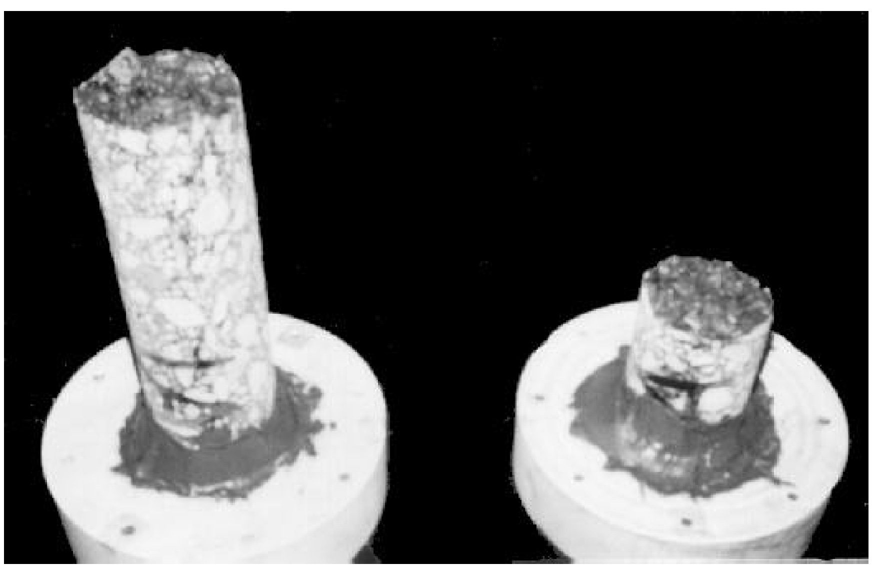


Figure 4.9 Typical TSRST Temperature vs. Tensile Stress Results

According to WYDOT specifications for determining adequate compaction, sample lots must have an average density of at least 92 percent of the maximum, with a range of 8 percent or less to be acceptable [WYDOT, 1996]. For the I-90 Kingsbury Road project the maximum density was 2510 kg/m³, as determined by WYDOT's Materials Branch. The average density for lab compacted Kingsbury samples was 94.1 percent with a



range of 4.1 percent, acceptable valuesunder WYDOT specifications. The pay factor for such densities is 0.888, which would be a pay deduction if a contractor had these densities in the field. The average density of the two field samples was 96.2 percent, which is good.

TSRST results for the Point of Rocks samples are shown in Table 4.2. The highest densities and tensile strengths and lowest fracture temperatures were observed in samples made from field slabs, followed by samples from ready mix, unaged lab mix, and STOA lab mix. Tests on two STOA lab mix samples were voided due to malfunctions with the TSRST step motor. In the tests, corrections were not made for the length of the shrinking sample for an extended period of time. The step motor suddenly tried to correct for different length by stretching the sample rapidly. Within minutes the sample failed under the increasing load.

Three samples tested in the TSRST were compacted at UW using a 45,000 kg press. This was done by placing mix in a 100 X 100 X 360 mm steel mold with a steel spacer on top of the mix. A Tinius-Olsen press was used to compact the mix by loading the spacer to 36,300 kg and releasing the load twice, then loading to 36,300 kg and holding at that load for five minutes. The compacted asphalt beam was then cored to obtain a five centimeter diameter core sample. Results from these samples also are given in Table 4.2.

TABLE 4.2 I-80 Point of Rocks TSRST Results

Sample			Density (kg/m^3)	Tensile Strength (kg/m^2)	Fracture Temperature (˚C)	Slope dS/dT (kg/m^2 /˚C)
Lab Compacted Linear Kneading Compactor	Paver Mix	1	2284	28.7	-25.2	17858
		2	2287	31.3	-26.8	17577
	Unaged Lab Mix	1	2286	27.3	-24.9	16874
		2	2252	23.9	-24.3	15397
	STOA Lab Mix	1	2204	---	-24.2	---
		2	2206	16.9	-21.4	10476
		3	2179	---	-25.0	---
		4	2188	17.8	-25.8	7312
Field Compacted	Field Slab	1	2318	35.7	-27.6	23904
		2	2281	31.3	-27.4	19616
		3	2302	34.2	-27.2	26014
		4	2332	37.0	-28.1	22428
Lab Compacted UW Press	Paver Mix	1	2209	19.7	-27.6	7734
	Unaged Lab Mix	1	2239	25.4	-23.7	17014
	STOA Lab Mix	1	2241	23.2	-25.8	13640

To determine if sample densities were adequate, samples from the I-80 Point of Rocks project were divided into four groups. The first group, consisting of two samples from paver mix and two from unaged lab mix, had an average density of 93.1 percent, a range of 1.5 percent, and a corresponding pay factor of 1.00. This pay factor indicates that a contractor would receive full payment for work of this quality. The second group, which included four short-term aged lab samples, had an average density of 89.7 percent which is below 92 percent and is not acceptable. This confirms that there were compaction problems with the aged mixes. The third group, which were field slabs, had an average density of 93.3 percent and a range of 2.1 percent, which gives a pay factor of 1.00. The fourth group, comprised of samples compacted with the Tinius-Olsen press at UW, had an average density of 91.1 percent, which also is not acceptable. It appeared that some aggregate breakage may have occurred during compaction of the samples.

GEORGIA LOADED WHEEL TEST

Accelerated tests to evaluate rutting resistance of flexible pavements have been around for many years and come in all shapes and sizes. Full-scale testing on test roads performed by traffic simulators have been used to predict rutting, along with portable methods such as the Accelerated Loading Facility (ALF). The methods involve full-scale pavements and high costs. However, smaller devices that can be used in a laboratory have been developed in various parts of the world. The French Rutting Tester and the Hamburg Wheel Tracking Device have been used extensively to determine rutting and stripping characteristics. Other tests include the Simple Shear Testing Device from the University of California at Berkeley, Environmental Conditioning System from Oregon State University, and the Rolling Wheel Machine developed by the Royal Dutch/Shell Group [Miller, 1995].

The Georgia Loaded-Wheel Tester was developed in 1985 by the Georgia Department of Transportation (GaDOT) and Georgia Tech to evaluate rutting characteristics of Georgia highways. This device allows small samples to be tested at temperatures similar to those found in the field. Studies have found that the GLWT can predict the level of rutting resistance in an asphalt cement mix [Lai and Lee 1990; Miller, 1995]. GaDOT has since used the GLWT extensively and now include the test in their mix design procedure [Miller, 1995].

Test Objectives

The Georgia loaded-wheel tester, shown in Figure 4.10, is an accelerated test used to determine rutting resistance of asphalt mixes before using the mixes in the field. This allows for experimentation of different mixes in the lab to produce pavements that perform better in the field. Since asphalt binders are temperature susceptible, their viscosities decrease with an increase in temperature. As a result, rutting typically occurs when pavement temperatures are elevated, such as during summer months. The GLWT allows pavement engineers to heat samples during the test to simulate field conditions. The Georgia loaded-wheel test consists of a weighted wheel running back and forth over a pressurized rubber tube on the sample, simulating a tire running over pavement. Rut depths are recorded after various numbers of cycles, which characterizes the rutting resistance of the mix.

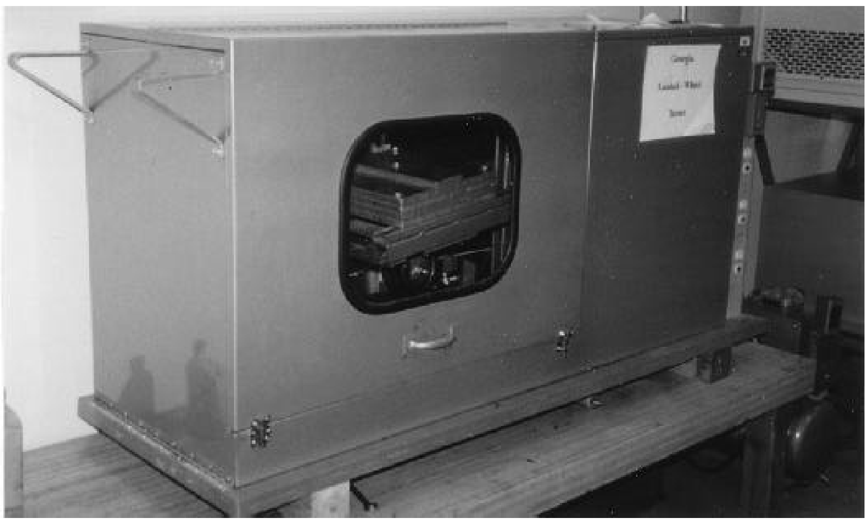


Figure 4.10 Georgia Loaded Wheel Tester

Test Samples

In the past, asphalt cement beams were used for testing in the GLWT. However, procedures were developed at the University of Wyoming to use 150 millimeter cores in the test. Cores are easier to handle, obtain, and compact than beams, and less material would be needed for testing [Miller, 1995]. A Superpave

Gyratory Compactor used for Superpave mix design procedures was used to compact cores in the laboratory. The gyratory compactor manufactured by Troxler and used at the University of Wyoming is shown in Figure 4.11. When performing a Superpave design, samples are compacted for a design number of gyrations. Gyratory compactors also have the capability to compact a sample to a given height, which makes GLWT testing easier since precast concrete spacers used to hold the sample match the height of the sample itself. Cores taken from the Kingsbury Road and Point of Rocks projects also were tested in the GLWT. These cores were obtained by WYDOT after pavement construction.

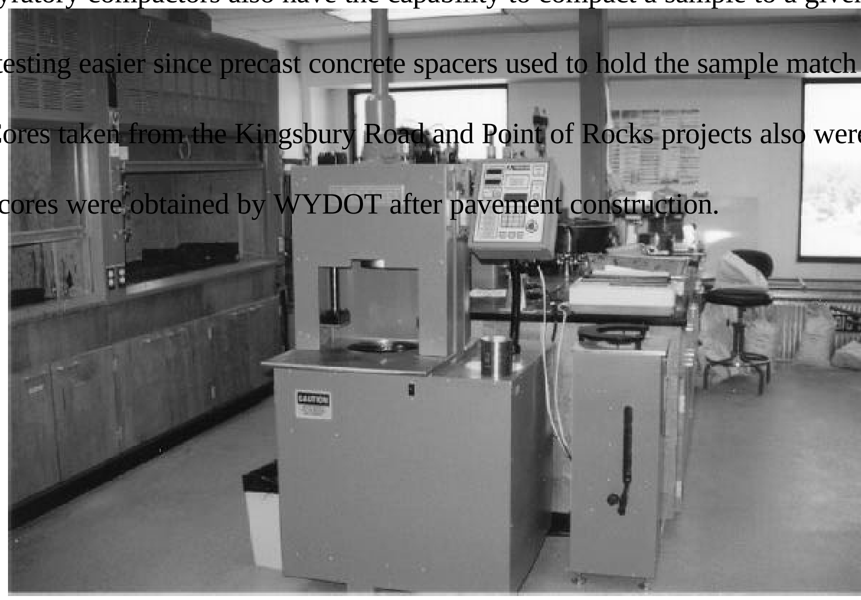


Figure 4.11 Gyratory Compactor used at the University of Wyoming

Additional samples were made from mix taken from the paver during construction and from cores cut from completed pavements of the I-80 Point of Rocks and I-90 Kingsbury projects. The only difference between the samples was the method of compaction, so it was expected that results of paver mix and field core samples would be similar. Likewise, lab mix that had been short-term oven-aged was expected to simulate new pavement. Lab mixes that had not been aged with those that had been STOA and LTOA were tested to determine effects of aging on GLWT samples.

Test Procedure

Before testing was performed, the GLWT environmental cabinet was preheated with a core to be tested. The temperature used to simulate field pavement temperatures during testing was 46.1˚C. This temperature was found to be severe enough to predict rutting and is similar to temperatures found in field pavements [Miller, 1995]. A core was placed in precast concrete spacers, which were tightened into place. Initial readings using the rut depth measuring device were taken. A rubber hose with air pressure of 689 kPa was placed in brackets that hold the hose stationary above the sample.The wheel assembly, to which 45.4 kg of steel weights are attached, was then lowered onto the hose. A motor moves the wheel assembly back and forth across the hose on the sample. One cycle consists of a back and forth motion of the wheel. The GLWT ran for 1,000 cycles, after which rut depths were measured using a rut depth measuring device. Rut depths were again recorded after 4,000 and 8,000 cycles. If the total rut depth after 8,000 cycles is less than 7.62 mm, the sample has passed the test.

Test Results

The Georgia loaded wheel test was performed on 11 samples from each project for a total of 22 samples. This included field cores taken from both projects and samples compacted in the UW lab using the gyratory compactor. All testing took place at the University of Wyoming. Test results are summarized in Appendix D.

GLWT results for Kingsbury Road samples are given in Table 4.3. All samples tested in the GLWT had acceptable rut resistance. Among laboratory prepared mixes, those that had been aged had smaller rut depths than the unaged samples. However, rut depths on lab-prepared mixes did not correspond with paver mix samples or field cores. It was expected that results from the paver mix, lab mix STOA, and field core samples would all correspond, but this was not the case. The field cores had the greatest rut depths of all samples.

TABLE 4.3 I-90 Kingsbury Road GLWT Results

Sample	Average Density (kg/m^3)	Average Rut Depth (mm)
Mix from Paver	2462	2.65
Unaged Lab Mix	2425	2.24
STOA Lab Mix	2439	0.81
STOA + LTOA Lab Mix	2444	0.81
Field Cores	2434	4.56

Densities of Kingsbury Road GLWT samples were quite good when compared to WYDOT standards. The samples were broken into three groups, with the first made up of paver mix and unaged lab mix samples. In comparison to the maximum density of 2510 kg/m^3, which was determined by the Materials Program at WYDOT, average density of the first group was 97.3 percent with a range of 2.2 percent. The corresponding pay factor for the densities are 1.10, which means that the densities achieved in this lot were high and consistent. The second group was made up of aged lab mixes, with all being short-term oven aged and some also being long-term oven aged. The samples had an average density of 97.3 percent of maximum with a range of 0.9 percent, which also has a pay factor of 1.10. The third group consisted of field cores, which had an average density of 97.0 percent with a range of 0.6 percent. Again, the pay factor worked out to be 1.10, which indicates that the contractor was entitled to a bonus according to the density of the samples. Overall, densities of the samples compacted in the gyratory compactor were very similar to samples taken from the field.

Rut depths of the Point of Rocks samples do not vary significantly among different sample types except for field cores as shown in Table 4.4. There also does not appear to be a trend in rut depth measurements with respect to aging. Rut depth measurements in all samples from the Point of Rocks project other than field cores were small, which indicates that this particular mix has great rut resistance properties. This was expected since nearly half the traffic on the I-80 Point of Rocks project is truck traffic and a strong mix was needed by WYDOT to prevent rutting in this section.

TABLE 4.4 I-80 Point of Rocks GLWT Results

Sample	Average Density (kg/m^3)	Average Rut Depth (mm)
Mix from Paver	2322	1.09
Unaged Lab Mix	2311	1.02
STOA Lab Mix	2316	1.50
STOA + LTOA Lab Mix	2301	1.07
Field Cores	*2253	*4.56

* Numbers affected by 19 mm wearing surface course

The field cores from the Point of Rocks project included a 19 mm wearing course on the surface. It appeared that rutting during the Georgia loaded wheel test may have been due to compaction of the wearing course, which was an open graded mix that does not possess much structural strength.

When looking at the densities of I-80 Point of Rocks samples, three groups were used. The first group was paver mix samples, which had an average density of 94.8 percent. No pay factor was computed due to a small group size, but densities were good. The second group was all six lab prepared samples. They had an average density of 94.3 percent and a range of 1.1 percent, which gives a pay factor of 1.0. The third group was the field cores, which had an average density of 92.1 percent, a range of 0.9 percent, and a corresponding pay factor of 0.583. The low densities of the field cores was due to a 19 mm wearing course, which comprised almost one-third of the core sample. Field slabs collected before the wearing course was added had excellent densities, indicating that addition of the wearing course was the cause of lower densities. Overall, the gyratory compactor used at the University of Wyoming created samples with consistent densities and appeared to do a good job of reproducing densities found in field pavement slabs.

FIELD EVALUATION

After obtaining results from thermal stress restrained specimen tests in the lab, comparisons had to be made with field performance of pavements at both projects. This was done by performing pavement distress surveys on each project. Methods used in this study for evaluating pavement distress are found in Distress Identification Manual for the Long-Term Pavement Performance Project [Strategic Highway Research Program, 1993], which provides methods of pavement distress categorization according to type, severity, and quantity. Also, the Pavement Condition Index (PCI) for each project was determined using the U.S. Army's PAVER procedure [Shahin and Kohn, 1981]. Data from pavement condition surveys were compared with actual temperature data taken from near the project sites. Temperature data were obtained from the Wyoming Water Resource Center located at the University of Wyoming.

In this study, pavement distress surveys focused on transverse cracking of pavements from the I-80 Point of Rocks and I-90 Kingsbury Road projects. Generally, transverse cracks are a result of thermal cracking due to low temperatures. Since pavements in this study were less than one-year-old when surveyed, other distresses, such as rutting or fatigue cracking, were not present. Crack severity was classified as low, moderate, or high. Low severity cracks have a mean width less than 6.4 mm. Moderate severity cracks have widths between 6.4 and 19 mm, while high severity cracks are wider than 19 mm.

Since performing a distress survey over an entire project would be time consuming, only samples of each project were surveyed. According to PAVER procedures from Shahin and Kohn (1981), a minimum of five samples should be surveyed, with more samples being included as pavement condition variations increase. It was determined that at least eight samples from each project should be surveyed, as there was not much variation expected in the condition of the new pavements. Data from the random samples taken throughout the project were then used to calculate the PCI for each pavement. The PCI for a pavement can range from 0 to 100, with the rating decreasing as a pavement deteriorates.

Each sample consisted of two 3.6 m lanes, 0.6 m of inside shoulder, 1.8 m of outside shoulder, at a length of 30.5 m along the roadway. This provided a sample area of 297 m^2, which is within the PAVER guidelines of 232±93 m^2 . Sample locations were chosen by dividing project length into even pieces and systematically picking samples spaced evenly throughout the project. This would ensure unbiased sample selection that would not be affected by field conditions.

The Point of Rocks project is about 16 kilometers long, so one sample per 1,600 meters was surveyed. The first sample location began approximately 800 meters from the west end of the project, as measured by a car odometer. Each consecutive sample was then located an additional 1,600 meters east, for a total of nine samples. Two sample locations were changed due to guardrail along the highway, which did not allow a place for a vehicle to be safely pulled off the roadway. The sample sites were moved to the nearest safe location. Samples were marked off using a hand odometer, and pavements were surveyed visually and data recorded. Location, length, and severity of each crack was recorded on data sheets for nine sample areas.

With the Kingsbury Road project length of eight kilometers, samples had to be spaced approximately 800 meters apart. The same procedure used for the Point of Rocks survey was used here, except that samples were spaced at 800 meter intervals throughout the project. Surveys were performed on westbound lanes only, as eastbound lanes had not yet been constructed. Eleven samples were observed on the project. An example of low temperature cracking from the Kingsbury test section is shown in Figure 4.12.

Figure 4.12 Low Temperature Cracking at Kingsbury Road Test Section

Results of the pavement condition surveys are presented in Appendix E. A summary of the results is given in Table 4.5. It was apparent that only minimal thermal cracking had occurred over the winter and spacings between cracks were large. For example, cracks in the Kingsbury Road project appeared to be spaced about 75 meters apart, meaning that most 30.5 meter long survey samples would not include cracking. Cracks that did appear on this project were completely across the road. Cracks in the Point of Rocks project occurred more frequently, but generally were short in length. No cracks completely traversing the road were observed in Point of Rocks samples. It also was noted that for both projects, all cracks observed were of low severity — no medium or high severity cracks existed in any survey samples. As a result of relatively small quantities of cracks with minimal severity, PCIs of these pavements were quite high, which would be expected from a new pavement.

TABLE 4.5 Pavement Condition Survey Results

	Point of Rocks Project	**Kingsbury Road Project**
Number of Samples	9	11
Number of Cracks	27	4
Total Crack Length (m)	68	36
Pavement Condition Index	98.7	99.4
Condition Rating	Excellent	Excellent

TEMPERATURE DATA

Temperature data were collected for sites located as close as possible to each project. Data from station 487845 located at the Rock Springs Airport were used for the Point of Rocks project, while data from station 483855 located 14 kilometers east-southeast of Gillette were used for the Kingsbury Road project. While locations of the stations were approximately 30 to 50 kilometers from the project sites, it must be understood that Wyoming is a rural state and these stations are the closest available that provide reliable data on a daily basis. Daily maximum and minimum temperatures covering August 1996 to April 1997 were collected to ensure that the lowest temperatures were included. Some observations of the temperature data are shown in Table 4.6 while complete data for the 1996-97 winter can be found in Appendix F.

Pavement temperatures and air temperatures are generally different but related. Asphalt Institute SP-1 (1995) contains the following equation, which calculates minimum pavement design temperature as a function of the low air temperature:

$$T_{min} = 0.859\ T_{air} + 1.7^{\circ}$$

where T_{min} = minimum pavement design temperature in ˚C
T_{air} = minimum air temperature in average year in ˚C.

Minimum pavement design temperatures were calculated and are presented in Appendix F. A summary of the pavement temperatures also are given in Table 4.6.

TABLE 4.6 Field Temperature Observations

	Station 487845 Rock Springs		Station 483855 Gillette	
	Air	**Pavement**	**Air**	**Pavement**
Total Observations	270	270	270	270
50th Percentile Temp (˚C)	-3	-1	-3	-1
Percentile Below 0˚C	63.5	53.5	65.0	55.7
Percentile Below -15˚C	3.3	1.1	14.8	6.3
Lowest Temperature (˚C)	-26	-20	-35	-28

CHAPTER SUMMARY

This chapter has presented testing and data collection procedures used in this study. The thermal stress restrained specimen test and Georgia loaded wheel tester were the laboratory tests used for this analysis. Background, objectives, procedures, and results of the tests were presented. Methods and results from field evaluations were included, such as pavement distress surveys and temperature data. Data analysis on field and lab results will be presented in the next chapter.

CHAPTER 5

DATA ANALYSIS

INTRODUCTION

Following data collection as well as the field and laboratory testing described in previous chapters, results were summarized and evaluated. Statistical analyses using one-way ANOVA and general linear model methods were performed on data to determine the effect of sample preparation on the TSRST results. This chapter summarizes all the statistical findings in addition to comparisons performed on field and laboratory data.

STATISTICAL ANALYSIS

A statistical analysis was performed on laboratory test data obtained in this study. One-way analysis of variance (ANOVA) was performed separately on TSRST and GLWT data for both Point of Rocks and Kingsbury samples. The analysis of variance method looks at the variance of a regression analysis and partitions the error into as attributed to the regression and error terms. ANOVA procedures allow easy calculation of an F statistic which is used to decide if a response is significant [Netter, Kutner, Nachtsheim, and Wasserman, 1996]. This study utilized the ANOVA method of regression analysis to determine if sample type, such as field slab, paver mix, unaged lab mix, or STOA lab mix, made a difference in density, fracture temperature, tensile strength, or rut depth. A simple regression analysis was conducted to determine the relationship between density and fracture temperature for TSRST samples. In addition, general linear models were used to determine if sample project had effects on density, fracture temperature, tensile strength, or rut depth of samples. Using a general model allows for many types of regression relationships, such as polynomial regression, transformed variables, qualitative predictor variables, and interaction effects [Netter et al., 1996]. The MINITAB computer package was used for all statistical calculations.

In an effort to compare low and high temperature properties of asphalt mixes included in this study, fracture temperatures from the TSRST were compared with rut depths from the GLWT for each sample type. This was done by simply plotting fracture temperature versus rut depth to see if the results were correlated. The plotting method used was rather unconventional, but this was necessary since rut depths and fracture temperatures came from completely different samples and could not be compared with conventional statistical methods.

Analysis on TSRST Data

The focus of laboratory testing for this study was on the thermal stress restrained specimen test (TSRST). As a result, most of the data analysis focused on results from this test. Statistical results are summarized in Tables 5.1 and 5.2, while complete statistical results can be found in Appendix G.

One-way ANOVA analysis was performed on TSRST data to determine if the type of sample used for each project effected density results. Statistical results can be found in Appendix G, and Table 5.1 presents a summary of ANOVA findings. A 95 percent confidence level (α level = .05) was used for all statistical tests. This analysis concluded that sample densities were dependant on sample type whether it is field slab, paver mix, unaged lab mix, or STOA lab mix.

TABLE 5.1 ANOVA Summary of Sample Type Significance

Response	Significance of Sample Type (α level = .05)			
	Kingsbury		Point of Rocks	
	Significant	p-value	Significant	p-value
Density	Yes	.001	Yes	.000

Response	Significance of Sample Type (α level = .05)			
Fracture Temperature	No	.223	No	.060
Tensile Strength	No	.420	Yes	.001

TABLE 5.2 General Linear Model Significance Summary

Response	Significance of Project (α level = .05)	
	Project	**p-value**
Density	Yes	.000
Fracture Temperature	No	.160
Tensile Strength	No	.168

Densities of the different sample types for the I-90 Kingsbury Road project indicated that there were differences among various types of samples, such as field slab, paver mix, unaged lab mix, and STOA lab mix. Although all precautions were taken to simulate field conditions in the laboratory, field and laboratory samples had different densities. In addition, short-term oven aging which is meant to simulate the aging that takes place during mixing and construction, was expected to provide results similar to the field mixes. However, lab prepared samples, especially those that had been STOA, had lower densities than field prepared mixes. It should be mentioned here that field slab and paver mix samples did have similar densities.

Density ANOVA results from the I-80 Point of Rocks samples were similar to those found in the I-90 Kingsbury Road samples. Densities of lab prepared samples were less than those of field samples, while the densities of field mixed samples were similar. Even when the linear kneading compactor was used to compact samples it did not effectively duplicate field densities. Short-term oven aging before compaction significantly reduced sample quality with respect to density.

As shown in Table 5.2, the general linear model indicated that densities were different for samples from each project. This was expected, as each project had a different density according to the job mix formula.

The density analysis indicated that samples taken from field mixes have better densities than samples made from lab mixes. Methods used to prepare TSRST samples in the lab could not simulate field densities. If TSRST samples with field densities are needed, they should come from HMA that has been mixed in the field. Other methods of laboratory sample preparation and compaction may more closely approximate field compaction. For example, densities of Georgia loaded wheel test samples prepared in the gyratory compactor were similar to those of field samples. However, modifications would be necessary to create TSRST samples in the gyratory compactor as it cannot currently accommodate current TSRST sample lengths.

As shown in Table 5.1, fracture temperatures for TSRST samples appeared to be similar regardless of sample type. Although fracture temperatures varied slightly from one sample to another, the variations statistically were not significant. This indicates that even though sample densities were slightly different, the fracture temperatures in the TSRST were nearly the same. This conclusion would allow the preparation of samples in the lab to test mixes before they are made in the field. As shown in Table 5.2, mixes from the Kingsbury Road and Point of Rocks projects had similar fracture temperatures. This indicates that both asphalt mixes should have similar resistance to low temperature cracking in the field.

Tensile strengths achieved by samples in the TSRST appeared slightly higher in samples made from field slabs. However, there were significant amounts of variation in recorded results. This is mainly due to the method of data collection for the TSRST device. Test data are collected at specified intervals, such as every two minutes. The last stress recorded before fracture was used as the fracture stress. This incorporates an error, depending on how much longer the sample took to break. Also, random differences in mix composition and aggregate position could create weak spots in a sample.

When tensile strengths at fracture were analyzed statistically, ANOVA concluded that strength was not dependent on sample type for the Kingsbury project while strength was dependent on sample type for the Point of Rocks project. The general linear model as shown in Table 5.2 suggests that there was no difference in tensile strength between the Point of Rocks and Kingsbury Road projects. This confirms past studies indicating that fracture strengths were rather difficult to reproduce [Jung and Vinson, 1993].

Aging of asphalt mixes in this study affected results from the TSRST. Unaged lab mixes had slightly lower fracture temperatures than STOA lab mixes. Although laboratory aging did make a difference in fracture temperatures, aging did not result in samples with performance similar to field samples.

A simple regression analysis was conducted to determine a relationship between density and fracture temperature for TSRST samples. A test to determine if linear relationships were similar for each individual project indicated that there was no difference between the sites. As a result, the analysis combined samples from both projects. The resulting regression analysis produced a relationship between density and fracture temperature that had a p-value of 0.028 and an R^2 value of 24 percent, which confirms that a relationship exists but is not strong.

Statistical Analysis on GLWT Data

To evaluate relationships between low temperature cracking and rutting in asphalt mixes, the Georgia loaded wheel tester was used to determine rutting characteristics of various mixes used in the study. Rut depths from GLWT samples were analyzed using the same statistical methods described above. Results from the analyses are shown in Tables 5.3 and 5.4. Field cores from the Point of Rocks project were not included in the statistical analysis due to the wearing surface course. As shown in Table 5.3, there were significant variations in rut depths among samples from the Kingsbury Road project. However, samples from the Point of Rocks project had similar rut depths. This indicates that the method used to make samples for extremely stiff mixes does not significantly affect the GLWT results. However for a softer mix, mixing and compaction methods can make a difference in GLWT results. For the most reliable results, field cores should be tested in the GLWT. Overall, no sample from either project failed in the GLWT, indicating that the mixes had adequate rut resistance.

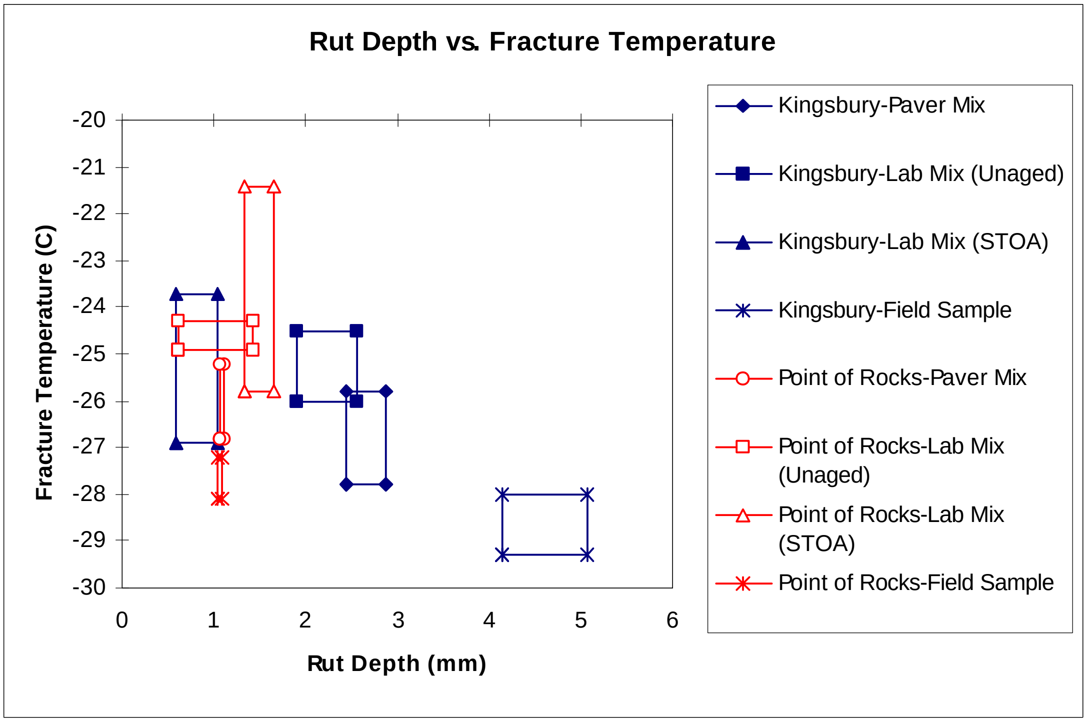
TABLE 5.3 ANOVA Summary of Sample Type Significance for GLWT Samples

Response	**Significance of Sample Type (α level = .05)**			
	Kingsbury		**Point of Rocks**	
	Significant	**p-value**	**Significant**	**p-value**
Rut Depth	Yes	.000	No	.464

TABLE 5.4 General Linear Model Significance Summary for GLWT Samples

Response	**Significance of Project (α level = .05)**	
	Project	p-value
Rut Depth	Yes	.023

A rut depth vs. fracture temperature plot was prepared by using the maximum and minimum data values from each type of sample for both projects. The maximums and minimums were combined to plot a box, which would indicate the range of values for each sample type. This rut depth vs. fracture temperature plot can be seen in Figure 5.1. It is clear from this plot that the Kingsbury Road samples had a linear relationship between rut depth and fracture temperature. As rut depths increase, the fracture temperatures decrease. This signifies a trade-off in asphalt mix characteristics because the low-temperature property



improves as the high-temperature property deteriorates. However, this relationship is not easily apparent in the Point of Rocks samples, as their rut depths were similar.

Figure 5.1 Rut Depth vs. Fracture Temperature Plot

ANALYSIS OF FIELD DATA

Field data were collected in the forms of pavement condition surveys and temperature data. As discussed in previous chapters, pavement condition surveys were used to calculate a pavement condition index (PCI) for each test section. Both projects had PCI values near 99, which indicates excellent pavement condition. This was expected as both pavements were less than one-year-old. Distress surveys indicated that the Point of Rocks section had more total cracking, although no observed cracks completely crossed the roadway. The Kingsbury section had less total cracking, but virtually every crack observed was completely across the highway.

Pavement distress surveys and pavement condition index (PCI) calculations performed on both I-80 Point of Rocks and I-90 Kingsbury Road test sections did not show a significant difference in pavement conditions. Because of the difference in temperatures experienced at both sites, it was not possible to determine if one field pavement had performed better than the other. Further study of these test sections after additional service could indicate if this is the case. Also, a test of different mixes used at the same location could indicate if a ranking of TSRST results would match pavement performance.

As stated previously, temperature data were obtained from sites near both projects. Only daily minimum temperature data were analyzed for this study. Ranking data from coldest to warmest quickly showed that Gillette had a significantly higher number of days below -15˚C (0˚F) than did Rock Springs, even though the numbers of days below freezing were similar for both sites. It also was apparent that the minimum recorded temperature for Gillette, -35˚C, was quite colder than the -26˚C minimum for Rock Springs.

Point of Rocks Lab and Field Comparisons

Although it is not statistically possible to compare TSRST results with field survey data, general observations and result comparisons were made. The lowest temperature recorded during the winter of 1996-97 at the Rock Springs airport was -26˚C. It was assumed that temperature readings from the recording station are similar to those experienced at the project. Thermal cracking occurred on the project, although cracks had not extended across the entire roadway. Most survey samples had cracks present, but they were generally on the shoulder or across one lane. Temperatures at which Point of Rocks field slab samples cracked in the TSRST averaged -27.6˚C, as seen in Table 4.6. This is just slightly below the actual low temperature experienced in the field, and well below the lowest pavement temperature. Samples made from Point of Rocks paver mix broke at an average of -26˚C. Lab mixed samples, unaged and short-term aged, broke at slightly warmer temperatures. From TSRST results it would be expected that some thermal cracking would have occurred, but the amount of cracking would not be extensive since temperatures did not drop well below the average fracture temperature. This correlates with distress surveys performed at the project, in which no cracks propagated completely across the pavement.

Kingsbury Road Lab and Field Comparisons

The lowest temperature recorded at the Gillette weather station over the winter of 1996-97 was -35˚C, with four occasions dropping below -30˚C. While low temperature crack spacings were quite large, cracks that had formed were completely across the highway. According to field slabs tested in the TSRST, the average fracture temperature was -28.7˚C. This would indicate that the pavement had been subjected to critical fracture temperatures on several occasions, and pavement temperatures would have reached this critical value. Results of distress surveys correspond to TSRST results as the entire roadway width has cracked.

Point of Rocks vs. Kingsbury Road

A general comparison of the two projects included in this study was made. This would explain differences in results that were observed due to different materials, environment, and construction. The Point of Rocks project used a polymer modified AC-20 asphalt and granite aggregate, where the Kingsbury road project used plain AC-20 asphalt and limestone aggregate. Material use would suggest that Point of Rocks pavements would be more resistant to low temperature cracking due to stronger asphalt and aggregate. However, thermal stress restrained specimen tests indicated that statistically both Kingsbury Road and Point of Rocks projects had similar resistance to thermal cracking. Overall test results indicate that HMA from the Point of Rocks project were generally stiffer than HMA from the Kingsbury Road project. This is supported by both TSRST and GLWT results.

CHAPTER SUMMARY

Statistical analyses confirmed that TSRST sample densities were dependent upon which project they came from and how they were made. However, fracture temperatures of the samples were not statistically dependent on type and were similar regardless of density. Tensile strengths were type dependent in one asphalt mix and not the other, suggesting that tensile strength may not be a good way of characterizing low temperature properties. Rut depths were type dependent in the softer Kingsbury Road mixes, but not in the stiffer Point of Rocks mixes. This indicated that different methods of mixing and compaction are more significant in softer mixes. Aging did appear to make a difference in test results for both the TSRST and GLWT, however the aged samples did not simulate field samples as anticipated. A plot of rut depths from the GLWT vs. fracture temperatures from the TSRST indicated that a linear relationship is present, with low-temperature properties improving as high-temperature properties deteriorated.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTION

This study of low temperature cracking in asphalt mixes was comprised of laboratory and field components. The thermal stress restrained specimen test (TSRST) and Georgia loaded wheel test (GLWT) were used in the laboratory to perform testing on asphalt samples from two WYDOT asphalt paving projects. The TSRST was used to evaluate the effectiveness of testing laboratory and field samples and to determine if laboratory results compare well with field performance of asphalt pavements. Aging effects on asphalt mixes also were observed. The GLWT was used to examine the high temperature rutting characteristics of asphalt mixes. These rutting characteristics were compared with low temperature characteristics obtained from the TSRST. Field data were recorded by conducting pavement condition surveys on the test sections and by collecting temperature data near each project. Using all data, the field performance of asphalt pavements was compared to laboratory test results. Statistical analyses were performed on laboratory test data to back up observed correlations between sample types, projects, and results.

CONCLUSIONS

Based on the testing and analysis performed in this study, the following conclusions can be made:

1. The thermal stress restrained specimen test is effective in evaluating low temperature cracking properties of asphalt mixes. Testing field samples in the device produces results to evaluate constructed asphalt pavements, while testing laboratory samples produces results to evaluate asphalt mixes before construction. Results for fracture temperatures were statistically equal regardless of sample type. Laboratory prepared samples had slightly warmer fracture

temperatures, but there was no statistical difference based on sample type even though the samples had statistically different densities.

2. Current laboratory compaction methods cannot simulate field densities. This is due mixing and compaction procedures, as field mixed samples compacted in the lab also had densities slightly below those found in field compacted samples.

3. Tensile strength should not be used to characterize the low temperature cracking resistance of an asphalt mix. Even though field slab samples had slightly higher tensile strengths than other samples tested in the TSRST, there were significant variations in strengths recorded in the various tests. Some of the variations were due to the data collection method, which recorded stress at specified intervals. Past studies have concluded that tensile stress results were somewhat difficult to reproduce, which was confirmed in this study.

4. Current asphalt mixes used in Wyoming have adequate rut resistance. The rut depths of the Kingsbury Road samples had statistically significant variations based on sample type, but were well within the criteria of the Georgia loaded wheel tester. The Point of Rocks samples had minimal rutting and rut depths for different sample types were similar. It was apparent that the Point of Rocks asphalt mix was quite stiff and the Kingsbury Road asphalt mix somewhat softer. Differences of sample type were more evident in the softer mix.

5. There is a trade-off of high and low temperature performance in asphalt pavement mixes. As low temperature performance improves, high temperature performance deteriorates. Results from the TSRST and GLWT were used to make a plot of rut depth vs. fracture temperature, which indicated that there was a linear relationship between rut depth and fracture temperature among the various sample types used in this study.

6. Additional field surveys are needed to determine the low temperature performance of the asphalt mixes observed in this study. Only slight low temperature cracking had occurred at both test

sections over their first winter in service. While the Point of Rocks section near Rock Springs had some cracking, temperatures at the site over the 1996-97 winter did not fall far below fracture temperatures recorded in TSRST testing. The Kingsbury Road section near Gillette had cracking completely across the roadway as temperatures at this site dipped well below the fracture temperatures recorded in TSRST testing on several occasions. These pavements will have increased thermal cracking after additional years of service if normal temperatures are experienced.

7. The degree of aging of a sample had a significant effect on laboratory test results. However, laboratory aging did not simulate aging that occurred during mixing and construction of HMA pavements.

RECOMMENDATIONS

1. While TSRST results were similar for samples tested despite slight density variations, a more efficient compaction method is needed. Compacting mixes with the linear kneading compactor at CDOT was time consuming and did not produce samples with densities similar to field samples. Possibly a method using the gyratory compactor could be developed using a larger sample size.
2. Although field samples can provide the most realistic results in the TSRST, laboratory samples can provide similar results despite lower densities. Therefore, it is recommended that field samples should be used when available and laboratory prepared samples should be used to predict performance prior to construction.
3. The field performance of both I-90 Kingsbury Road and I-80 Point of Rocks projects should be monitored for additional years of service to determine low temperature characteristics. One winter is not enough to fully evaluate low temperature cracking resistance. Data collected over a longer time period will enable field and laboratory results to be fully correlated.

4. Further study is necessary to determine if laboratory aging is necessary to simulate aging that occurs during field mixing and compaction. The method and degree of laboratory aging also should be investigated.

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APPENDIX A: Job Mix Formulas

APPENDIX B: TSRST Sample Results

APPENDIX C: TSRST Results Summaries

APPENDIX D: GLWT Results Summaries

APPENDIX E: Pavement Condition Index Calculations

APPENDIX F: Temperature Data

APPENDIX G: Statistical Data