LABORATORY EVALUATION OF RUTTING IN ASPHALT PAVEMENTS

by

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PREFACE

In this research, the feasibility of using the Georgia Loaded-Wheel Tester (GLWT) to predict rutting in the laboratory was investigated. This research was performed in two phases. The first phase consisted of modifying the GLWT to handle 15.2 cm (6 in) cores, developing a laboratory compaction procedure for cores, determining the optimum laboratory testing conditions, and investigating the repeatability of the GLWT. The second phase of this research project included correlating rut depth values obtained with the GLWT to actual field rut depth values, utilizing the GLWT to evaluate the effects of the asphalt additive SOMAT on asphalt concrete mixes, and evaluating the rut resistance of Stone Matrix Asphalt (SMA).

Results from this study show that the GLWT is capable of predicting rutting in asphalt pavements prior to construction. In addition, results from the GLWT correlate well with results from more expensive European Testers.

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INTRODUCTION

Background

In the last decade, pavement rutting has become a considerable problem for many state highway agencies. Ruts are the depressions that occur in a pavement's wheel paths due to traffic loadings. Increased truck tire pressures and heavier axle loads are the two leading causes of this problem. Rutting stems from the permanent deformation in any of the pavement layers or the subgrade, usually caused by the consolidation or lateral movement of the materials due to traffic loads [1]. Consolidation is the further compaction of asphalt pavement after construction. This occurs when field compaction techniques are inadequate in compacting the Hot Mix Asphalt (HMA) to the proper level of air voids. All bituminous mixes will experience some consolidation, but problems develop when too much consolidation occurs. Lateral movement or plastic flow is most commonly associated with the use of excess asphalt cement. The asphalt cement reduces the friction between the aggregates that causes the traffic loads to be carried by the asphalt cement instead of the aggregates. Repeated loadings force the asphalt cement out of the voids between the aggregates and thus permanent deformations occur in the HMA. Significant rutting can lead to major structural failures and hydroplane potentials.

In the past, pavement engineers did not have adequate tools to detect bituminous mixes that were susceptible to rutting prior to construction. However, the recent usage of accelerated testing devices has proven to be very successful in predicting rutting.

Problem Statement

Most mix design procedures currently used by state highway agencies are reliable in eliminating extremely poor asphalt mixes. However, they offer little assistance in distinguishing among mixes with high, moderate, or even low rut resistance. With this dilemma, many state highway agencies are looking for reliable alternative methods to predict the rut resistance of asphalt mixes. Constructing test sections can determine if an asphalt mix has adequate rut resistance, but the procedure is expensive and requires years of field measurements and analysis. An alternative technique is to use laboratory accelerated rut testing devices where the rutting characteristics of an asphalt mix can be determined in a matter of days.

The state of Wyoming, like other states, has its share of pavement rutting. Predicting pavement rutting prior to construction is on top of the Wyoming Department of Transportation (WYDOT) priority list. Therefore, the University of Wyoming investigated the feasibility of using the Georgia Loaded-Wheel Tester (GLWT) to predict field rutting in the laboratory.

Objectives of Research

This research was performed in two phases. The principle objectives of the first phase were to:

1. modify the GLWT to test 15.2 cm (6 in) cores,
2. develop a laboratory compaction procedure for cores,
3. determine the optimum laboratory testing conditions, and
4. investigate the repeatability of the GLWT.

The objectives of the second phase were to:

1. correlate GLWT and field rut depth values for selected test sections,
2. utilize the GLWT in evaluating the effectiveness of asphalt additives used to increase the rut resistance of asphalt concrete mixes, and
3. evaluate the rut resistance of Stone Matrix Asphalt (SMA).

Report Organization

Chapter 2 of this report reviews the literature associated with rutting. The origination, development, and usage of accelerated pavement testing devices also are discussed. Chapter 3 describes the data collection process and overall evaluation strategies followed in this research. Chapter 4 discusses the modifications that have been made to the GLWT both at the University of Wyoming and elsewhere. Chapter 5 describes the correlations between laboratory and field rut depth values for selected test sections in Wyoming. Chapter 6 evaluates the usefulness of various types of mixes and additives to increase the rut resistance of bituminous pavements. Chapter 7 summarizes the tasks performed in this study, presents the conclusions, and includes recommendations for future research.

CHAPTER 2

LITERATURE REVIEW

Introduction

In the United States, there is approximately 4 million miles of roads of which 2.3 million miles are paved. Of the 2.3 million miles, two million miles are surfaced with asphalt. The remaining 0.3 million miles are composite or rigid pavements [2]. Between 1989 and 1990, the annual vehicle-miles of travel increased from 2.10 trillion to 2.15 trillion, or 2.4 percent.

Pavement engineers continuously look for ways to improve the service life and performance of flexible pavements. In some states, such as Wyoming, it is challenging to design an asphalt mix that performs well in both the summer and winter because of the extreme seasonal temperature variations. Nationwide, a cyclic trend of problems occur with asphalt mixes. In the past, cracking was a major problem on roadways. To overcome this problem, pavement engineers began specifying asphalt cements with lower viscosities to increase the flexibility of the pavements. Although the cracking problem was resolved, decreasing the viscosities of the asphalt cements led to plastic flow and rutting in hot summer months. Rutting is the current dilemma facing pavement engineers. For many years, researchers have been trying to find a solution to this dilemma by experimenting with different mix designs and using asphalt cement additives. This chapter describes some of the techniques used to predict and eliminate rutting.

Components of Asphalt Pavements

Highway engineering is a field that requires the effective use of materials manufactured by nature. Naturally-occurring soils serve as the foundation for highway pavements. Some serve faithfully and well while others cause problems at every opportunity. Nature's products are often used in pavement bases and asphalt mixtures with relatively minor refinements [3]. Asphalt concrete mixtures are comprised of primarily two components, aggregates and bituminous materials.

Aggregates used in most roadway construction are obtained from local sources due to their availability and low hauling cost. The characteristics of the aggregates determine how well they will behave in the asphalt mix. Aggregates in Hot Mix Asphalt (HMA) are generally classified by size as coarse aggregate, fine aggregates, and mineral filler. Coarse aggregates are those aggregates retained on the 4.75 mm (No. 4) sieve, fine aggregates are those aggregates which pass the 4.75 mm sieve, and mineral filler has at least 70 percent of the material passing the 0.075 mm (No. 200) sieve, according to the ASTM standards. Coarse and fine aggregates are generally required to be hard, tough, strong, durable, properly graded; to consist of cubical particles with low porosity; and to have clean, rough hydrophobic surfaces [4]. Aggregates that lack any of the above characteristics often lead to problems when used in HMA. Mineral fillers can include crushed fines, lime, portland cement, fly ash, carbon black, and sulfur. Fillers are useful to fill voids, reduce the optimum asphalt cement content, increase stability, improve the bonding between asphalt cement and aggregates, and meet specifications for aggregate gradation. Aggregate gradation is determined by sieve analysis to provide the distribution of particle sizes expressed as a percent of the total weight. The sizes of the aggregates used in an asphalt mix also are important. If the coarse aggregates are too small, the mix may be unstable. However, if the aggregates are too large they may segregate and cause problems with the workability of the mix. Generally, highway agencies have developed gradations that are suitable for local conditions based on field experience. Table 1 shows the Wyoming Department of Transportation gradation band, which incorporates gradations of suitable aggregates.

The second primary component of HMA is bituminous materials. Asphalt cement is man's oldest engineering material with recorded uses dating back to 6000 B.C. It is a dark brown to black cementatious material that is either naturally-occurring or produced by petroleum distillation. Asphalt most commonly used in flexible pavement construction can be divided into three types: asphalt cements, emulsified asphalt, and cutback asphalt [4].

Table 1. Wyoming Aggregate Gradation Band for Bituminous Pavements

Sieve Size	Percent Passing
19.0 mm (3/4 in)	90 - 100
12.5 mm (½ in)	60 - 100
9.5 mm (3/8 in)	60 - 85
4.75 mm (No. 4)	40 - 70
2.36 mm (No. 8)	25 - 55
600 μm (No. 30)	10 - 30
75 μm (No. 200)	2 - 11

Asphalt cements are obtained by distilling crude petroleum. At ambient temperatures, asphalt cements are black, sticky, semisolid, and highly viscous. They are strong and durable with excellent adhesive and waterproofing characteristics. Asphalt cements are primarily used in the construction of HMA by applying heat, which reduces the viscosity, and mixing with aggregates. After the HMA has cooled it is extremely durable and can handle high traffic loadings. Two of the most common grading systems for asphalt cement are penetration and viscosity gradings. The penetration grades are determined by ASTM D946 and can be categorized in five groups according to their decreasing stiffness: 40-50, 60-70, 85-100, 120-150, and 200-300. The viscosity grades are determined by ASTM D3381 and can be categorized in six groups: AC-2.5, AC-5, AC-10, AC-20, AC-30, and AC-40. The viscosity grade corresponds to the viscosity of the original asphalt cement where AC-40 is the most viscous. A similar viscosity grading was developed for asphalt cements aged in the rolling thin film oven (RTFO). The viscosities of these asphalt cements are much higher and are classified as: AR-1000, AR-2000, AR-4000, AR-8000, and AR-16000.

Emulsified asphalt are mixtures of asphalt cement, water, and an emulsifying agent. These asphalt are liquid at ambient temperature, which allow them to be applied at low temperatures. When the emulsified asphalt is mixed with the aggregates, the asphalt cement clings to the aggregate's surface and the water evaporates leaving a semi-rigid pavement.

Cutback asphalt are liquid asphalt, which are manufactured by adding petroleum solvents to asphalt cements. When these asphalt are added to aggregates, the asphalt cement adheres to the aggregates and the solvents dissipate. These asphalt have low viscosities and are applied at lower temperatures than asphalt cements. Cutback asphalt are being phased out of service because they emit volatiles into the atmosphere, waste energy in their production, and pose safety problems with their flammable solvents [4].

Asphalt Cement Additives

Many additives are currently being studied to determine their improvements to the rheological and/or adhesive properties of nature's own asphalt cement. An asphalt cement additive is defined as a material that would normally be added to/or mixed with the asphalt before or during mix production to improve the properties and/or performance of the resulting binder and/or mix [3]. An additive that increases the stability or rut resistance will most likely decrease the flexibility of the mixture and increase the probability of cracking. An additive that is capable of reducing the temperature susceptibility of an asphaltic mixture may be expected to be the panacea. A generic classification of asphalt modifiers is presented in Table 2.

Rubbers

Rubber has been used for a number of years as latex or crumb rubber to improve certain properties of HMA mixtures. It can be used to increase the stability of a mix thus increasing the rut resistance. It also can increase the flexibility of a HMA, which reduces the low temperature cracking. The use of rubber in a HMA requires raising the temperatures at which it is mixed and compacted. The addition of crumb rubber increases the viscosity of the asphalt cement. Asphalt cements modified with rubber from passenger car tires have higher viscosities than those modified with industrial tires. Researchers contribute these findings to lower percentages of natural rubber and higher percentages of synthetic rubbers in passenger car tires [5]. According to Khedaywi [6], the addition of rubber to asphalt cement decreases the stability and increases the flow.

Plastics

Plastics are generally used to improve the rut resistance of asphalt mixes. However, they also have been used to decrease the temperature susceptibility of mixes and reduce low temperature cracking.

Fibers

Fibers have been used to increase the stiffness of mixes, which increases the resistance to rutting. The fibers also provide tensile strength to asphalt mixtures, which reduces reflective cracking. Fibers have also been used to increase the durability of HMA by increasing the optimum asphalt content.

Oxidants

Oxidants are another type of additive that improve the rut susceptibility of asphalt pavements by increasing the stiffness. However, problems do develop with oxidants because the increased stiffness promote low temperature cracking.

Table 2. Generic Classification of Asphalt Modifiers

<table>
<tr><th>Type</th><th colspan="2">Example</th></tr>
<tr><td>1. Filler</td><td colspan="2">Mineral filler:
crusher fines, lime, portland cement, fly ash
Carbon Black
Sulfur</td></tr>
<tr><td>2. Extender</td><td colspan="2">Sulfur
Lignin</td></tr>
<tr><td>3. Rubber
a. Natural latex
b. Synthetic latex
c. Block copolymer
d. Reclaimed rubber</td><td rowspan="3">P
O
L
Y
M
E
R
S</td><td>Natural rubber
Styrene-butadiene (SBR)
Styrene-butadiene-styrene (SBS)
Recycled tires</td></tr>
<tr><td>4. Plastic</td><td>Polyethylene
Polypropylene
Ethyl-vinyl-acetate (EVA)
Polyvinyl chloride (PVC)</td></tr>
<tr><td>5. Combination</td><td>Blends of polymers in 3 and 4</td></tr>
<tr><td>6. Fiber</td><td colspan="2">Natural:
Rock Wool
Man-Made:
Polypropylene
Polyester
Fiberglass</td></tr>
<tr><td>7. Oxidant</td><td colspan="2">Manganese salts</td></tr>
<tr><td>8. Antioxidant</td><td colspan="2">Lead compounds
Carbon
Calcium salts</td></tr>
<tr><td>9. Hydrocarbon</td><td colspan="2">Recycling and rejuvenating oils
Hardening and natural asphalt</td></tr>
<tr><td>10. Antistrip</td><td colspan="2">Amines
Lime</td></tr>
</table>

SOURCE: Roberts, F.L., Prithvi Kandhal, E. Ray Brown, Dah-Yinn Lee, and Thomas Kennedy. (1991). Hot Mix Asphalt Materials, Mixture Design, and Construction. Lanham, MD: NAPA Education Foundation.

Antioxidants

Antioxidants improve the durability of HMA by reducing the oxidation. These additives are believed to increase the rutting of asphalt mixtures because they reduce the aging that naturally occurs.

Hydrocarbons

Hydrocarbons are normally used to increase the stiffness of a HMA by adding a more viscous asphalt cement to a soft asphalt cement. They also can increase the durability of a HMA by adding a softer asphalt cement to a stiffer one.

Antistripping Agents

Antistripping agents are used to increase the bond between asphalt cement and aggregate. These additives may be added to mixes that are moisture susceptible to reduce stripping. Lime is the most widely used antistipping agent [4].

Shale Oil Modified Asphalt Treatment

Since 1986, The New Paraho Corporation has been engaged in the development of a new shale oil modified asphalt paving product, called SOMAT. The SOMAT binder is prepared by blending 10-15 percent by weight of the Shale Oil Modifier (SOM) with virgin asphalt.

Effectiveness of Asphalt Additives

Several studies have been performed to determine the effectiveness of asphalt additives. A study conducted by the Texas Transportation Institute at Texas A&M University looked at the effects of five different asphalt cement additives on rutting and cracking. The five additives that were included in the study were:

1) Ethylene Vinylacetate (EVA)
2) Polyethylene (finely dispersed)

3) Block Copolymer Rubber or styrene-butadiene-styrene (SBS)

4) Latex (styrene-butadiene rubber)

5) Carbon Black

The following three damage indicators were used in the study: rut depth, roughness, and cracking. It was concluded that the most effective additives in reducing rutting were EVA, polyethylene, SBS, and Carbon Black [3].

In a study conducted in Wyoming by Farrar et al., six asphalt cement modifiers were tested in the field and by using the Hamburg rutting device. The modified binders consisted of two types of SBS blocked copolymer, carbon black, SBR latex, polypropylene fiber, and ethylene vinylacetate copolymer resin. The field performance of these additives were compared to control sections for five years. This study indicates that all test sections performed well except the one containing carbon black, which had medium to severe rutting. It also was noted that the control mix was ranked third in performance above some modified asphalt cements [7].

A field study was conducted by The New Paraho Corporation to assess the in-service performance characteristics of SOMAT. Characteristics of SOMAT were compared with those of conventional and other modified asphalt now in commercial use [8]. The study compared the SOMAT to alternative modified binders by analyzing the contribution of each to reducing the following distresses: water stripping, thermal cracking, fatigue cracking, rutting, and oxidative aging. The addition of Shale Oil Modifier (SOM) to the binder increases its adhesive qualities and therefore, enhances the ability of the asphalt cement to bond to the aggregate. The additional bonding strength significantly improves the ability of a HMA to reduce stripping. SOMAT has a higher stiffness than conventional asphalt at high service temperatures and is more flexible at low service temperatures. These characteristics give SOM modified bituminous pavements greater protection against both low-temperature cracking and rutting. When the SOMAT pavements were tested using the repeated-load indirect tensile method they withstood significantly more cycles than asphalt pavements modified with SBR and styrene butadiene block copolymer. This shows the potential increase in resistance to fatigue cracking of the SOM modified asphalt. Initial results from the addition of SOM suggest that it also provides improved resistance to long-term aging, or binder embrittlement. From a construction standpoint, SOMAT pavements are much less tacky than polymer modified asphalt which make them easier to place and clean-up [8].

Stone Matrix Asphalt (SMA)

Although modifying asphalt cement generally increases the resistance of HMA to rutting, it is not the only solution to reduce rutting. Special asphalt mixes have been developed, such as the Stone Matrix Asphalt (SMA), to decrease the potential rutting problem in bituminous pavements. SMA is a gap-graded hot mixture that maximizes the binder and coarse aggregate contents [3]. It was developed in Europe more than 20 years ago. However, it is only recently being tested in the United States. Table 3 shows typical target gradation specifications for SMA mixes [9]. Table 3 also includes a typical 19mm (3/4 in) dense graded asphalt mix gradation used in Wyoming [10]. It is apparent when looking at the two gradations that SMA mixes are gap-graded while the typical dense graded mixes are more well-graded.

In Europe, SMA mixes have been successful in resisting permanent deformations. The high resistance to deformation is due mainly to the stone-on-stone contact or "stone skeleton" that occurs when the large aggregates support each other in the compacted state. Figure 1 illustrates the stone-on-stone contact which occurs in SMA mixes. In a study conducted by Brown and Mallick, it was found that the stone-on-stone contact occurs when the percentage of aggregate passing the 4.75 mm sieve is 30 percent or less. The stone-on-stone contact is considered to begin at either of the following two points: when the voids in coarse asphalt (VCA) plot begins to curve as the amount of material passing the 4.75 mm sieve is reduced, or when the voids in mineral aggregate (VMA) plot begins to increase as the percent passing the 4.75 mm sieve is reduced [11]. The following equations are used to calculate the VMA and VCA:

$$VMA = 100 - (V_{agg}/V_{Tot}) * (100)$$

$$VCA = 100 - (V_{CA}/V_{Tot}) * (100)$$

where:

VMA = Voids in the Mineral Aggregates

VCA = Voids in the Coarse Aggregates

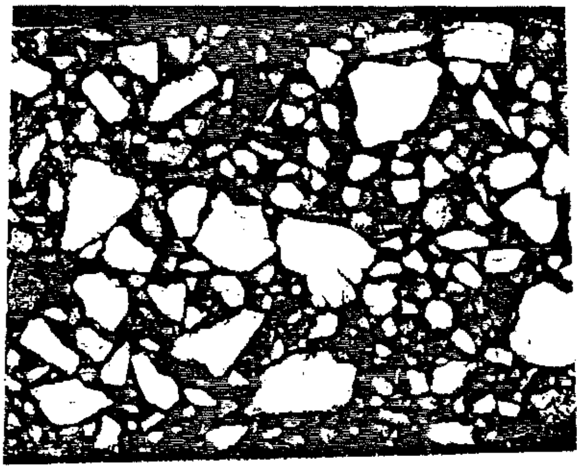
V_{agg} = Volume of aggregate, calculated using bulk specific gravity

V_{Tot} = Total volume of compacted mixture

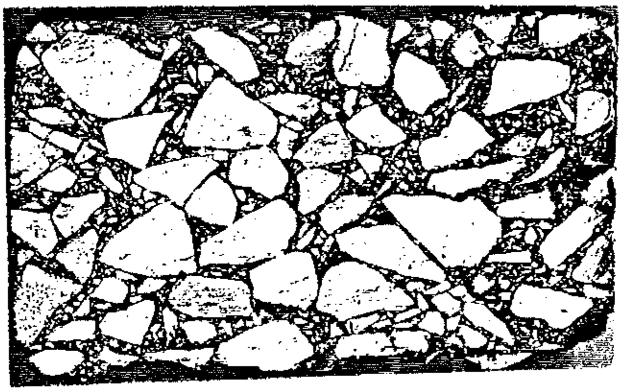
V_{CA} = Volume of coarse aggregate, calculated using bulk specific gravity

Table 3. Typical SMA and 19 mm (3/4 in) Dense Graded Mix Gradations

	Percent Passing	
Sieve Size	**Typical Dense Graded Mix**	**SMA Mix**
19.0 mm (3/4 in)	90 - 100	100%
12.5 mm (½ in)	60 - 85	85 - 95
9.5 mm (3/8 in)	------	75 (maximum)
4.75 mm (No. 4)	40 - 60	20 - 28
2.36 mm (No. 8)	25 - 45	16 - 24
600 µm (No. 30)	10 - 30	12 - 16
300 µm (No. 50)	------	12 - 15
75 µm (No. 200)	2 - 8	8 - 10



Dense Graded Asphalt Mix



SMA Mix

Figure 1. Illustration Comparing Dense Graded Asphalt to SMA

Current Asphalt Mix Design Procedures

Since the beginning of the century, pavement researchers have been trying to develop standard practices for mixing asphalt cements with aggregates. In 1912, a book was published by Richardson that described the first method for determining the optimum asphalt content in a mix design [12]. The procedure was called the "Pat Test." This procedure was performed by pressing a fine grained surface mix against manilla paper and observing the stain. A dark stain meant the mix was too rich, a light stain meant the mix was too lean, however a medium stain meant the mix design was "just right." Several other asphalt mix design procedures were later developed. Currently, most state highway agencies use the Hveem or Marshall mix design procedures to determine the optimum asphalt content. The superpave mix design procedure, which was recently developed by the Strategic Highway Research Program (SHRP), is being evaluated for possible implementation in the near future.

Hveem Mix Design Procedure

The Hveem mix design was developed by Francis Hveem, a resident engineer in California, in the 1930's. Hveem realized that as the diameter of the particles decreased in oil mixes so did the oil film thickness. This led to the development of the kerosene equivalent test. Later, Hveem developed the Hveem stabilometer to evaluate the ability of mixes to resist the shear forces applied by wheel loads. A cohesiometer was developed with the stabilometer to measure the cohesive strength across the diameter of a compacted core. By 1959, the Hveem procedure was in its final form for designing HMA mixes [4].

Marshall Mix Design Procedure

The Marshall mix design was developed by Bruce Marshall of the Mississippi Highway Department around 1939. In 1943, the Corps of Engineers Waterways Experiment Station (WES) began to study the use of the Marshall apparatus to select the optimum asphalt content and densities for pavements used at airfields. The Marshall apparatus was appealing because it could easily be transported to the field for quality control. It has since been modified for use in laboratories throughout the United States. In a 1984 survey, it was found that 38 states used some version of the Marshall method while 10 states used the Hveem method [4].

SHRP Superpave Mix Design Procedure

The Superpave mix design system is a comprehensive method of designing paving mixes tailored to the unique performance requirements dictated by the traffic, environment (climate), and structural section at a particular pavement site. It facilitates selecting and combining asphalt binder, aggregate, and any necessary modifier to achieve the required level of pavement performance [13]. The superpave mix design is applicable to all bituminous layers with a wide variety of pavement materials and characteristics. It analyzes mixes for resistance to the following three distresses: permanent deformation, fatigue cracking, and low-temperature cracking. The objective of the superpave mix design system is to define an economical blend of asphalt binder and aggregate that yields a paving mix having sufficient asphalt binder, sufficient voids in the mineral aggregate (VMA) and air voids, sufficient workability, and satisfactory performance characteristics over the service life of the pavement. In contrast to the Marshall and Hveem mix design methods which are empirical, the SHRP mix design method originated solely from performance-based properties.

Pavement Rutting

Ruts are the depressions which occur in a pavement's wheel paths due to traffic loads. Rutting stems from the permanent deformation in any of the pavement layers or the subgrade, usually caused by the consolidation or lateral movement of the materials due to traffic loads [1]. Consolidation is the further compaction of asphalt pavement after construction. This occurs when field compaction techniques are inadequate in compacting the HMA to the proper level of air voids. All bituminous pavements will have some consolidation take place but problems occur when too much consolidation occurs. Under normal conditions a 10.16 cm (4 in) layer compacted to 7-8 percent air voids during construction will consolidate to 4-5 percent air voids in two to three summers and develop rut depths approximately 0.30 cm (0.12 in). Higher rut depths may illustrate that the deformations are occurring in an underlying layer due to shear failure.

Compacted HMA with voids higher than 8 percent are prone to rapid oxidation, which leads to cracking and raveling. Compacted HMA with voids lower than 3 percent are highly susceptible to rutting and shoving. HMA should be compacted to an air void level between 3 percent and 8 percent because repeated vehicular loadings will cause the HMA to eventually compact to its natural air void percentage (3-5 percent). Most consolidation and therefore rutting occurs in the wheel paths where the majority of the loadings take place.

Rutting also occurs because of lateral plastic flow in the wheel paths. The most common cause of plastic flow is the use of excess asphalt cement. The asphalt cement reduces the friction between the aggregates, which causes the traffic loads to be carried by the asphalt cement instead of the aggregates. Repeated loadings force the asphalt cement out of the voids between the aggregates and thus permanent deformations occur in the HMA. Lateral flow is minimized by reducing the asphalt cement content, using larger aggregates, using textured aggregates that have high friction factors, and providing adequate compaction to the HMA during construction [4]. Increasing the viscosity of the asphalt cement also reduces the amount of rutting. Methods of increasing the viscosity are: using a higher grade of asphalt, using asphalt cement modifiers, or increasing certain mineral fillers. However, more viscous asphalt cements are prone to cracking which is equally detrimental to rutting.

A fine line exists between significant rut depths vs. normal consolidation. According to Jimenez, the critical rut depth is 0.95 cm (3/8 in) [14]. It is not until this point that ponding water will cause hydroplaning. According to Roberts et al., the cross-slope of the pavement section is the controlling factor in determining when a rut is deep enough to accumulate water and poses a safety threat. In any instance, significant rutting can lead to major structural failures and hydroplane potentials.

Before excessive rutting can be effectively repaired so it will not reoccur, the cause or causes must be established. A full-depth analysis of the pavement structure should be analyzed prior to any repair. If rutting is caused by overstressing the subgrade soil because the pavement structure is too weak for the loads, leveling the wheel paths and a thin overlay does not permanently solve the rutting problem. Problems may be occurring because there is not adequate drainage under the pavement structure. In fact, the necessity to conduct a structural evaluation to find a solution can not be overemphasized.

Accelerated Pavement Testing Devices

Several accelerated rut testing devices were developed more than half a century ago to determine deficiencies in pavements. Although some of these original testers are no longer being used, their concepts have been refined to simulate actual traffic characteristics. By subjecting samples of flexible pavement to these accelerated testing devices, the general performance of the pavement can be discovered prior to actual construction. If an asphalt mixture performs well in one of these devices then it is likely to have adequate performance in the field. Currently, several accelerated rut testing devices and prototypes are being used throughout the world. Each testing device has its own unique operating mechanism and specification for pavement performance. These pavement testing devices can be classified generally as being large, medium, or small [15].

The large "traffic simulators" are generally constructed to perform testing outdoors. If testing is conducted indoors a larger than average laboratory is required. Waterways Experiment Station (WES) was one of the first sites where a large testing device was used. Test sections at WES were 3.66 m (12 ft) wide and up to 91.44 m (300 ft) long [15]. Generally, test sections are constructed as oval or circular test tracks. The test track encompasses a central hub to which all the loading apparatuses are connected.

Loads up to 266.9 kN (60,000 lb) can be applied to loading apparatuses, which are generally capable of simulating actual traffic by allowing the wheels to wander. Pavements are subject to millions of loading applications and pavement distresses are measured with actual field equipment at various intervals. The circular or oval test tracks have up to a one-mile circumference and numerous advantages. First, test tracks are large enough to construct with normal field procedures. This provides the most accurate modeling process possible. Second, continuous unidirectional trafficking can be applied to test sections which better simulates field conditions. The continuous movement reduces the amount of time lost in accelerating and decelerating on straight test sections. Continuous movement also increases the speed at which the loading apparatus can achieve and maintain. Another advantage of the circular or oval test track is that several wheel apparatuses can be added to increase the number of loading applications applied to a pavement in a given time period. One advantage of testing outdoors is that the pavement samples exposure to all elements of the weather. One major disadvantage to the large rut testing devices is the cost. The cost of large testers is in the millions of dollars. Another disadvantage to the large testers is the time required to test pavements may exceed one year.

Medium pavement testers operate on a much smaller scale than the large testing devices but still utilize normal construction procedures for the asphalt pavements. The medium size testers are probably the most diverse of the three groups. Some of these testers are moved to test in situ while others are permanently fixed in large buildings and have more controlled testing environments. The Heavy Vehicle Simulator (HVS) used in South Africa can be transported under its own power to the test site. Loads up to 10,200 kg (22,500 lb) can be applied bidirectionally through a mounted tire at speeds up to 10 km/h (6.2 mi/h) [15].

The Accelerated Loading Facility (ALF) is another temporary testing facility that can be transported to different test sites via truck. The ALF was developed by the Australians. The design was later sold to the Federal Highway Administration (FHWA) and a prototype was built in the United States at the FHWA Turner-Fairbanks Research Laboratory. Loads up to 7260 kg (8 tons) can be applied unidirectionally through the wheel assembly. Programmable transverse distribution of load passes simulates the random non-uniformity of actual traffic patterns [16]. The cost of an ALF is in excess of one million dollars. Another device is called the Accelerated Testing System (ATS). This device was developed by Purdue University in cooperation with the Indiana Department of Transportation (INDOT). This testing facility is fixed in a large laboratory and spans a pit where prototype scale pavement sections are installed. The pit is underlain by a system of heaters that can raise the temperature of the pavement to simulate summer pavement temperatures. This system is capable of applying loads up to 9070 kg (20,000 lb) bidirectionally at rates up to 8 km/h (5 mi/h). The apparatus can move laterally to allow wandering of the wheel assembly [15].

Small pavement testers are generally installed in laboratories. All aspects of these testers are scaled down to simulate the effects of traffic. The mixes tested in these devices are laboratory compacted in contrast to the normal construction methods used with large and medium testing devices. However, pavement researchers have developed many compaction apparatuses and procedures to simulate field compaction. A study conducted by Tim Aschenbrener of the Colorado Department of Transportation shows that laboratory cores compacted with the French plate compactor and the linear kneading compactor are more stable than those compacted under normal construction practices [17]. Several small pavement testing devices are described in the remainder of this chapter. Each testing device has its own design and unique features that distinguish it from the others.

The French Rutting Tester

The French Rutting Tester is considered to be a "European Torture Test" because of the grueling conditions to which pavements are exposed. The French tester evaluates the resistance to permanent deformation on slabs 50 by 18 cm (19.7 by 7.1 in) and 2 to 10 cm (0.8 to 3.9 in) thick [18]. Two slabs can be tested simultaneously in the French tester. Slabs are prepared with the Laboratoire Central des Pons et Chaussees (LCPC) Plate Compactor. Prior to testing, slabs are loaded with 1,000 cycles at room temperature. This is taken as the "zero" reading. Next, the samples are heated to testing temperature for 12 hours before testing begins. During testing the slabs are loaded with 5,000 N (1124 lbs) by a pneumatic tire inflated to 0.6 MPa (87 psi). Tests are typically conducted at 60°C (140°F). Rut depth measurements are taken at 100; 300; 1,000; 3,000; 10,000; 30,000 and possibly 100,000 cycles. A successful test will have a rut depth less than 10 percent of the slab thickness after 30,000 cycles. A pair of slabs can be tested in approximately nine hours. The cost of the French Rutting Tester and LCPC Plate Compactor is $185,000 [18].

The Hamburg Wheel Tracking Device

Another "European Torture Test" is the Hamburg Wheel Tracking Device which was developed in Germany to determine the moisture susceptibility of asphalt pavements. The size of the slabs tested is 25 by 28 cm (9.8 by 11 in) and 6 to 9 cm (2.4 to 3.5 in) thick. Slabs are prepared with the Linear Kneading Compactor. Two slabs can be tested simultaneously. This device is similar to the French Rutting Tester except that the slabs are immersed in a 50°C (122°F) water bath and loaded by a steel wheel. The wheel is loaded with 705 N (158 lbs). The machine is automated and records the deformations after each cycle. The inverse slope of the rut depth vs cycle curve is calculated simultaneously. Figure 2 is an illustration of a typical rut depth vs. cycle curve. This plot is relatively straight up to the stripping inflection point, which is the point where stripping begins to occur on the sample. The straight segment of the line is called the creep slope because it is caused by the plastic flow of the asphalt cement. The plot is relatively straight past the stripping inflection point except with a much higher slope. This segment of the plot is considered the stripping slope because all of the rutting is due to stripping instead of plastic flow. A successful test will have less than a 4 mm (0.16 in) rut depth after 20,000 cycles. In Colorado, it has been determined that deformations up to 10 mm (0.39 in) are acceptable for their pavements. A pair of slabs can be tested in approximately six hours. The cost of the Hamburg Wheel Tracking Device is $45,000 [18]. Tim Aschenbrener has shown that results from the Hamburg Wheel Tracking Device correlate well with results obtained in the field. The Colorado Department of Transportation (CDOT) is so confident in their results that they have started awarding bonuses to contractors based on results from the Hamburg Torture Test [17].

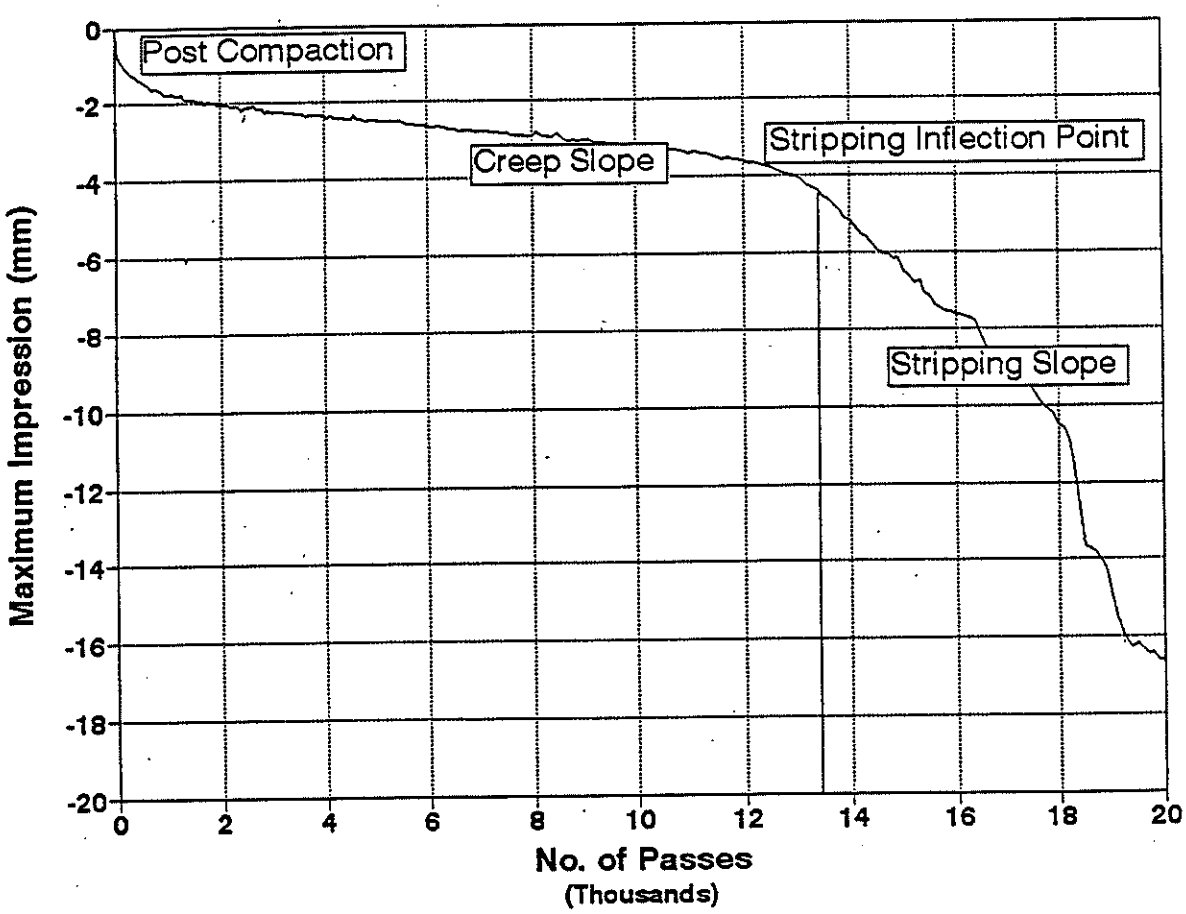


Figure 2. Definition of Results from the Hamburg Wheel-Tracking Device

Georgia Loaded-Wheel Tester

In 1985, the Georgia Department of Transportation (GaDOT) initiated a research study on loaded wheel testers. GaDOT contracted Georgia Tech to develop a testing device that would meet their needs. Shortly thereafter, the GaDOT realized that the original Georgia Loaded-Wheel Tester (GLWT) could meet their needs if a few minor modifications were made. The GLWT, shown in Figure 3, was developed by Benedict Slurry Seals, Inc. to test asphalt slurry seals.

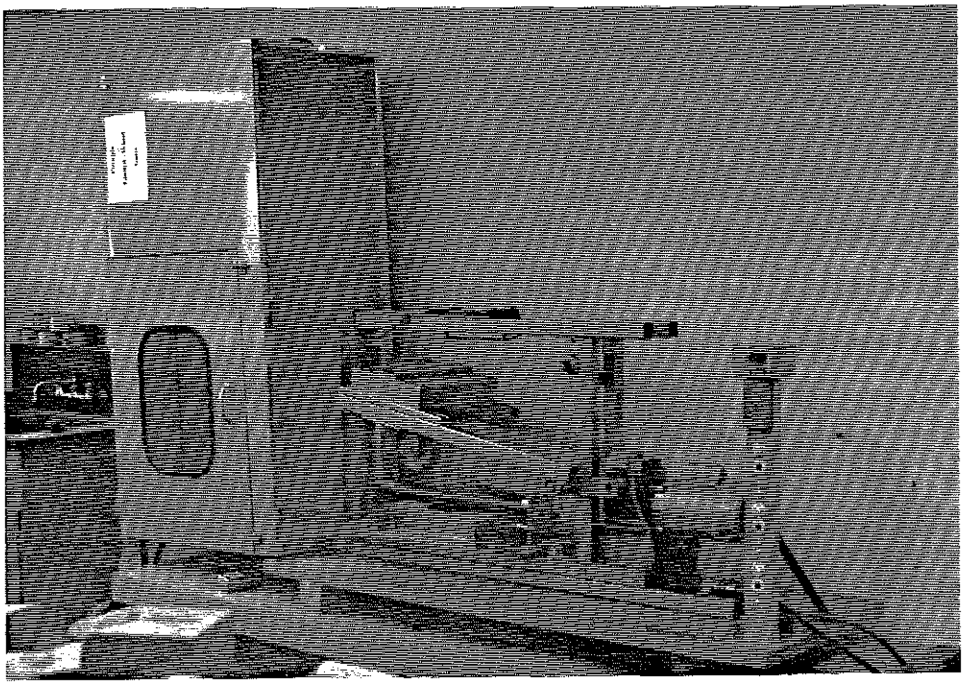


Figure 3. The Georgia Loaded-Wheel Tester

The GLWT test beams are 7.62 x 7.62 x 38.1 cm (3 x 3 x 15 in) and can be compacted with a variety of methods. Prior to testing, samples are preheated to the testing temperature. The samples are placed in the GLWT and confined by the sample holding molds. A 2.54 cm (1 in) diameter hose inflated to 0.69 MPa (100 psi) is placed on top of the sample and the loading device is lowered onto the sample.

The loaded-wheel is loaded with 45.4 kg (100 lb) of steel weights. This combination of weight and pressure has been shown by Collins to produce contact pressures of approximately 100 psi [19]. The loaded-wheel straddles the rubber hose and traverses back and forth over the sample. At a predetermined number of cycles the loaded-wheel automatically stops and rut depths are recorded. Typically, samples are considered to "pass" the GLWT if the rut depths are less than 0.762 cm (0.30 in) after 8,000 cycles. A cycle is one loading in each direction. One sample can be tested in approximately 3 ½ hours. The GLWT has been shown by Lai and Lee to potentially distinguish between levels of rut resistance in asphalt mixes [20]. Researchers from the GaDOT have conducted extensive research with the GLWT and have been thoroughly impressed. Following this research, the GaDOT incorporated the GLWT in their mix design procedure with the Marshall mix design procedure [19].

Other Small Rut Testing Devices

The Simple Shear Testing Device was developed at the University of California at Berkeley. Several prototypes are currently being tested. The cost of this accelerated tester is $150,000. This device is being considered by the Strategic Highway Research Program (SHRP) to predict permanent deformation characteristics of asphalt pavements. The Environmental Conditioning System (ECS) was developed at Oregon State University. The cost for this device is $45,000. The ECS is being considered by SHRP to predict moisture sensitivity characteristics [18].

The Rolling Wheel Machine was built by the Royal Dutch/Shell Group. The test sections are constructed between concrete walls on a 3 m (9.8 ft) circular track. Loads up to 2,000 kg (4,410 lb) are applied hydraulically to the wheels as they rotate around a center hub [15].

Chapter Summary

Significant rutting can lead to major structural failures and potential hydroplaning. Traditional methods for examining the characteristics of bituminous pavements are not adequate for determining rutting resistance. Several types of additives are currently used to improve the performance of asphalt. The effectiveness of these modified asphalt and special types of asphalt mixes are currently being evaluated with accelerated pavement testers.

CHAPTER 3

DESIGN OF EXPERIMENT

Introduction

In this research project, extensive data were collected in the field and laboratory to fulfill the objectives presented in Chapter I. The study was performed in two phases. Figure 4 summarizes the overall tasks performed in both phases. The main objective of the first phase was to modify the testing conditions of the Georgia Loaded-Wheel Tester (GLWT). The modified test conditions of the GLWT were later used in Phase II to examine the rut resistance of selected field cores and two special asphalt mixes. The first mix was prepared with the asphalt additive SOMAT (Shale Oil Modified Asphalt Treatment) while the second mix was a Stone Matrix Asphalt (SMA). This chapter describes all the tasks performed in both phases of this research study.

Tasks Performed in Phase I

Modification of the GLWT and Test Specimen

In August of 1993, the University of Wyoming purchased the Georgia Loaded-Wheel Tester from Benedict Slurry Incorporated. The GLWT was enclosed by an insulated box to maintain constant testing temperatures. The apparatus was fabricated to test 7.62 x 7.62 x 38.1 cm (3 x 3 x 15 in) beams. The beams were very heavy and awkward to compact so the feasibility of testing 15.2 cm (6 in) cores was investigated. The preparation of cores is less time consuming and requires less material than beams. In addition, obtaining cores from the field is much easier than removing beams from an existing pavement. After a thorough investigation, cores were accepted for testing in the GLWT at the University of Wyoming.

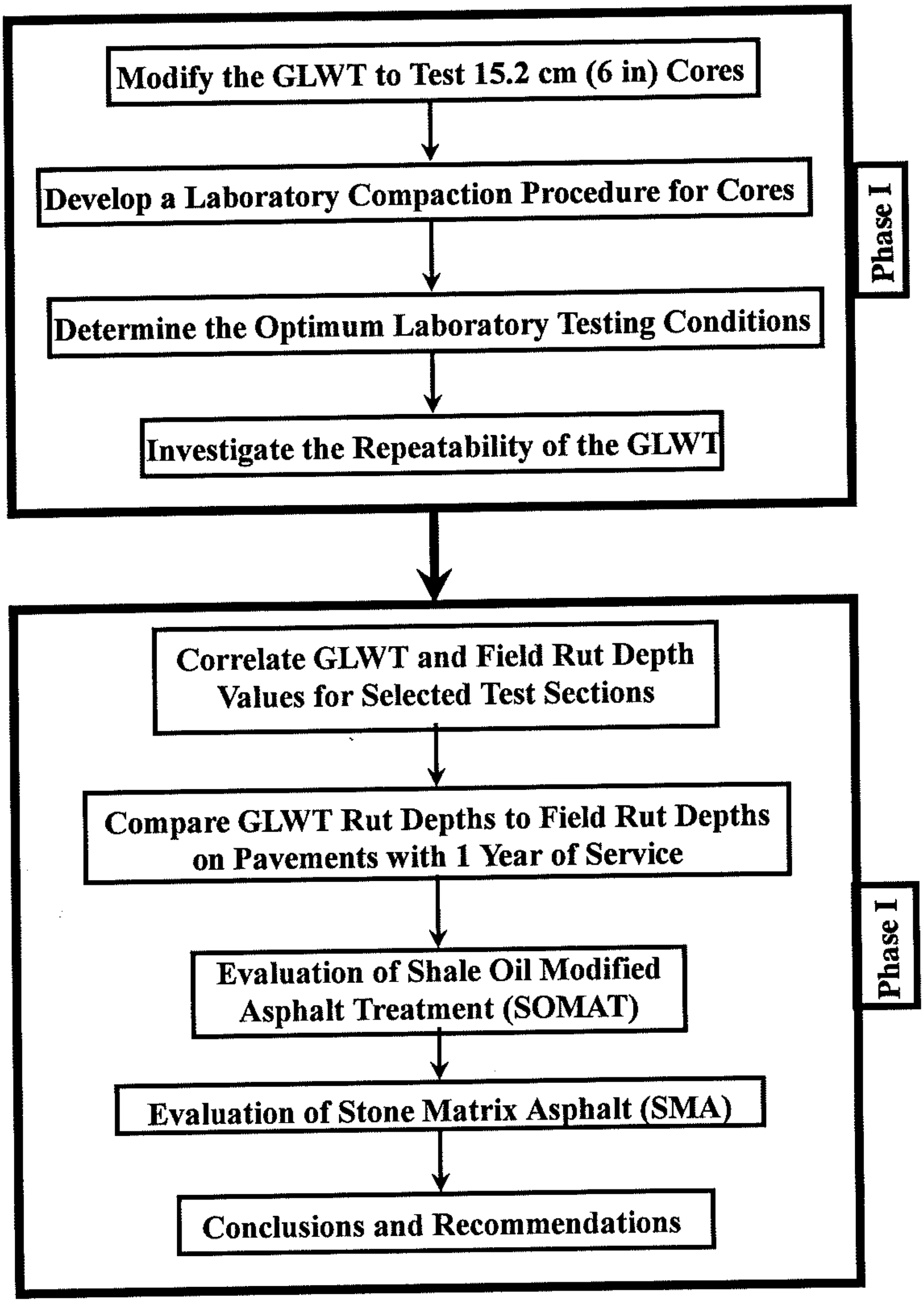


Figure 4. Overall Strategies Followed in this Research Project

Developing a Laboratory Compaction Procedure for 15.2 cm Cores

Some of the samples tested in this research study were compacted in the laboratory. Therefore, it was necessary to develop a standard laboratory compaction procedure that would produce densities similar to those experienced in the field by normal construction methods. Initially, the standard procedure for preparation of test specimens of bituminous mixtures by means of California Kneading Compactor (AASHTO T247-89) was followed. This procedure produced densities much higher than those obtained in the field. Therefore, this procedure was modified until design densities were achieved. The final process for compacting the samples is thoroughly discussed in Chapter 4 of this report.

Determining the Optimum Testing Conditions

It was found in the literature review that the majority of agencies using the GLWT conduct testing at 40.6°C (105°F), but recommended testing at higher temperatures. Testing conducted at the University of Wyoming evaluated several temperatures. After the preliminary testing was finished, the testing temperature of 46.1°C (115°F) was determined to produce rut depths that correlated best with field rut depth values.

Testing the Repeatability of the GLWT

After modifying the GLWT, several identical laboratory cores were prepared with the standard compaction technique developed in this project. These cores were tested in the GLWT under identical conditions. The results concluded that the GLWT produced repeatable results on similar cores. The results from the repeatability study are discussed in detail in Chapter 4 of this report.

Tasks Performed in Phase II

Selection and Evaluation of Field Test Sections

Thirteen pavement test sections were selected in the State of Wyoming for inclusion in this experiment. These sections were selected according to their geographic locations and rut depth severity level. The sites represented all regions in the State of Wyoming. The Wyoming Department of Transportation (WYDOT) collected cores from all test sections. These cores were immediately tested in the GLWT at the University of Wyoming. The results from the GLWT were later correlated to actual field rut depths.

Evaluating Pavement with One Year of Service

Loose HMA was collected from behind the paver on a primary highway project. This mix was reheated and compacted in the laboratory. The rut depths predicted by the GLWT were compared to the field rut depths after one year of service. These results are presented in Chapter 5.

Evaluating SOMAT

Loose HMA containing Shale Oil Modifiers (SOM) was collected from behind the paver on an I-90 interstate project. HMA also was collected for an adjacent control section. The rut depths from the modified and control asphalt mixes were compared after testing in the GLWT. This evaluation examined the benefits of using SOM binder. Results of this study are presented in Chapter 6.

Evaluating SMA

The Wyoming DOT built several SMA test sections on I-80 in 1994. One control and two SMA mixtures were collected from behind the paver on the I-80 experiment. These mixes were reheated and compacted in the laboratory. In addition to the loose HMA, cores were later obtained from all experimental sections. The GLWT rut depth results from the SMA mixes were compared to results obtained from testing the mixes in European rut testing devices. The results are presented in Chapter 6.

Chapter Summary

This chapter presented the data collection and overall evaluation strategies followed in each phase of this research project. All of the data for each of these individual studies were collected and summarized in a computerized data base. The different mixes and modifiers were evaluated according to their ability to reduce rutting. The results and recommendations from each study are presented in Chapter 7. Overall, each task provided a method to thoroughly satisfy the objectives of this research project.

CHAPTER 4

MODIFICATIONS TO THE GEORGIA LOADED-WHEEL TESTER

Introduction

The original Georgia Loaded-Wheel Tester (GLWT) was developed by Benedict Slurry, Inc. for the GaDOT Materials Testing Laboratory to test asphalt slurry seals. It included a loaded-wheel driven by an electric motor, a weight holding box, and a mounting plate for the asphalt specimen [21]. Minor modifications were made to the original GLWT to test asphalt samples [19]. Modifications were made in the following areas: test specimen, loading wheel, temperature control, and rut profile measurements. This chapter looks at the modifications that were made to the GLWT at the University of Wyoming and elsewhere.

Modifications to the Test Specimen

The original GLWT was designed to test asphalt slurry seals. Generally, slurry seals have a 0.635 cm (0.25 in) maximum aggregate size [4]. Typical Plant Mix Pavements (PMP) often have aggregates exceeding 2.54 cm (1 in). To accommodate the larger aggregates, the size of the GLWT test specimen had to be increased. A beam 7.62 cm wide, 7.62 cm high, and 38.1 cm long (3" x 3" x 15") was chosen since the maximum aggregate size used in Georgia asphaltic concrete mixes is slightly less than 3.81 cm (1.5 in). This agreed with the general rule of thumb that the mixture thickness should be approximately twice the diameter of the largest stone in the mix [19]. Figure 5 shows a photograph of a standard GLWT sample. Research at the University of Wyoming concentrated on modifying the GLWT to test 15.2 cm (6 in) diameter cores with a height of 7.62 cm (3 in) for a variety of reasons. First, extracting cores from existing pavements is much easier than extracting beams. Second, 15.2 cm cores are easier to compact in the laboratory and require less time and material to prepare. Third, cores are more readily available and are commonly removed for standard laboratory testing.

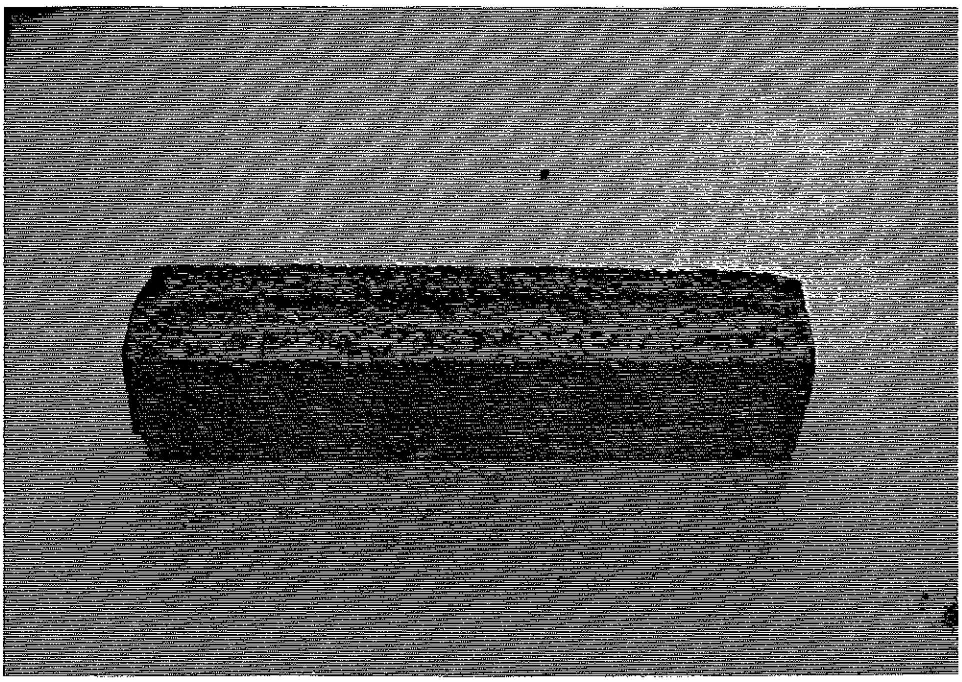


Figure 5. Standard GLWT Test Sample

Testing 15.2 cm (6 in) Cores in the GLWT

The initial attempt to test cores was performed by placing three samples simultaneously in the GLWT as shown in Figure 6. This endeavor was quickly abandoned due to excessive rocking of the cores during testing. To overcome the rocking problem, fresh concrete was placed around individual cores and allowed to set before testing. This procedure, shown in Figure 7, was too time consuming.

Single cores were ultimately tested by placing pre-cast concrete spacers on both sides of the cores to accommodate the 30.5 cm (12 in) travel path of the loaded-wheel. Several spacers were made in advance to accommodate slight fluctuations in core heights. Figure 8 shows this standard procedure for testing cores in the GLWT at the University of Wyoming.

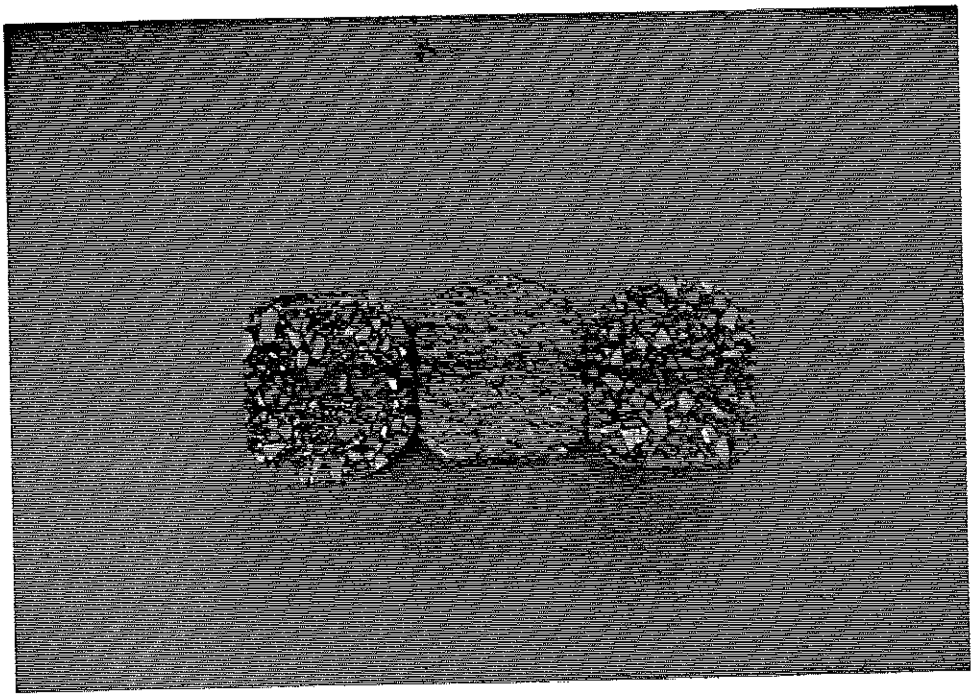


Figure 6. Testing Three Cores Simultaneously

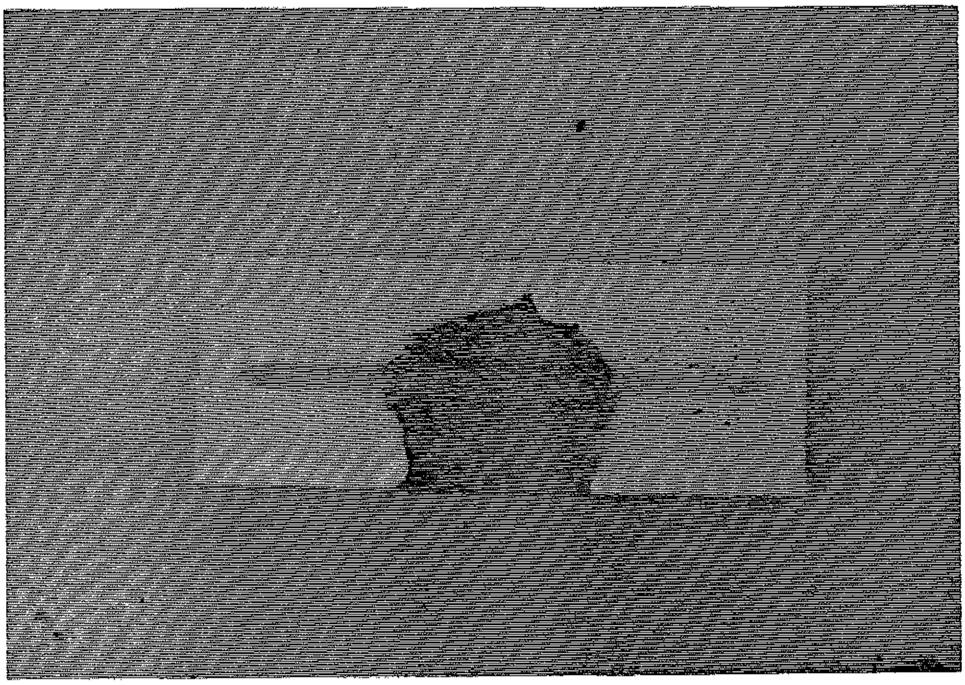


Figure 7. Placing Concrete around Individual Cores

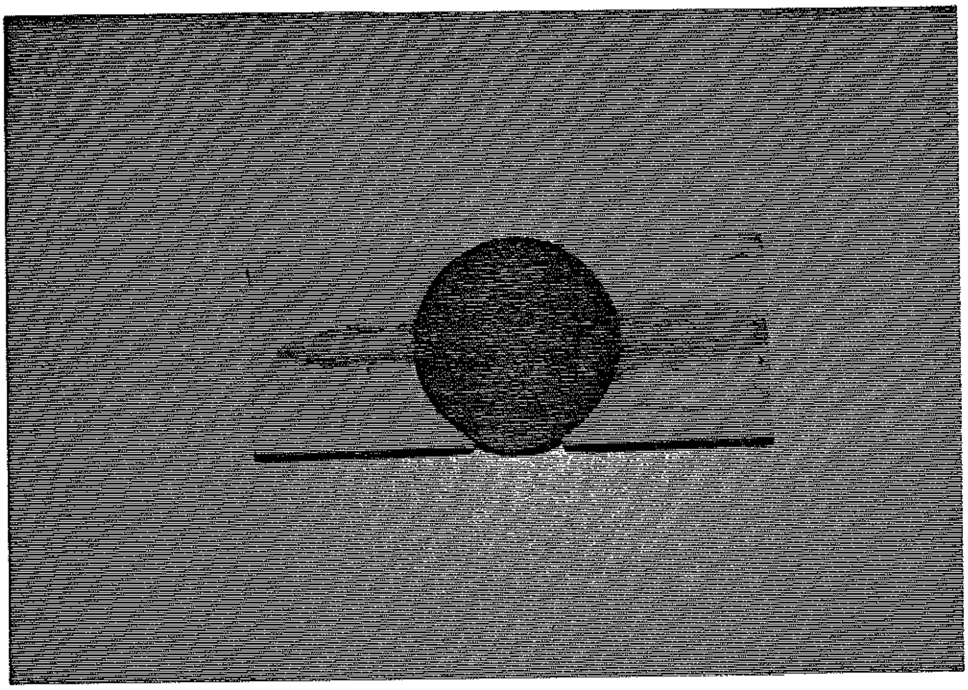


Figure 8. Standard Procedure for Testing Cores at UW

Modifications to the Compaction Procedure

One of the primary causes of rutting is consolidation, which is the further compaction of HMA from loadings after construction. Naturally, with lower levels of voids or higher densities, a compacted asphalt mix will experience less consolidation. A wide variety of densities will produce a diverse range of rut depths. For this reason, it was necessary to develop a compaction procedure that could produce uniform densities for similar mixes.

The Original Compaction Procedure

According to Collins et al. [19], the first attempts to compact beams for testing in the GLWT were performed with a California Kneading Compactor. The standard California Kneading Compactor's compaction foot was modified with a square foot to compact the asphalt samples. The molds were moved manually to insure uniform compaction on each of two or three lifts. This procedure was abandoned because the temperature of the asphaltic mixture dropped too much by the end of the compaction process. A thorough investigation was later conducted by Lai [21] on the compaction of the beam specimens statically with a hydraulic press. The advantage of using a universal testing device like the hydraulic press is that they are readily available in most laboratories. The final mold assembly has inside dimensions 38.1 cm long, 7.62 cm wide, and 12.7 cm high (15 x 3 x 5 in). The molds are constructed from 1.27 cm (0.5 in) steel, and weigh approximately 40.4 kg (89 lb) when they are filled with an asphalt mixture. The molds have a removable bottom so beams can be extruded. These molds are very awkward to handle during the compaction process because they are hot and heavy.

FHWA Round-Robin Testing

A round-robin test was conducted in cooperation with six DOT's. Materials for a fine-grained asphalt mix were obtained from the Georgia DOT (GaDOT) and shipped to all participating agencies. Each DOT compacted the mixtures with the standard static compaction method described in the previous section. The samples were tested in the GLWT and it was concluded that different laboratories produced significantly different sample densities and rut depths. Therefore, an alternative method was sought for compacting the samples [22].

Compaction Procedure Followed by GaDOT

Two methods currently used for compacting asphalt beam specimens at the Georgia DOT are the linear kneading and the rolling compaction processes. Both of these methods use a rolling compactor. In the linear kneading compaction process, a series of 125 mm (4.92 in) thick steel plates are placed on top of a loose asphalt mix. Vertical loads up to 5443 kg (12,000 lb) are applied through a loaded wheel that rolls across the steel plates. The rolling compaction process is similar to the kneading compaction process except instead of steel plates, a semi-rigid nylon liner is placed between the loading wheel and the loose asphalt mix. Figures 9 and 10 illustrate how each of these compaction processes are accomplished. Each of these processes have their advantages and disadvantages. For example, the kneading compaction is faster and produces more uniform surface densities than the rolling compaction. However, the kneading compaction process produces samples with higher densities on the top portion and a higher percentage of fractured aggregates. The rolling compaction process produces results that are more characteristic of actual field compaction processes [23].

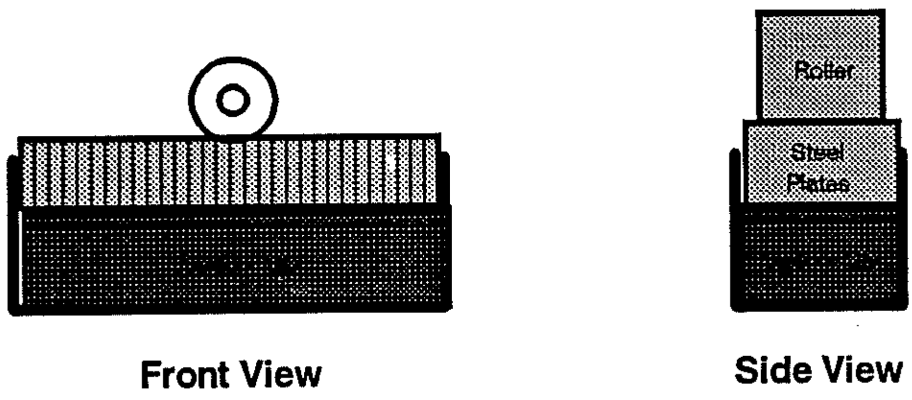


Figure 9. Linear Kneading Compaction Process

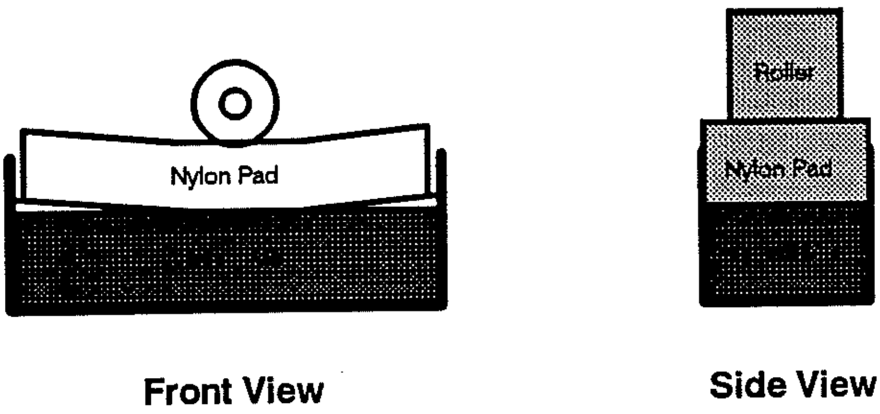


Figure 10. Linear Rolling Compaction Process

Compaction Procedure Developed at the University of Wyoming

The preliminary investigation of the GLWT at the University of Wyoming followed the static compaction procedure on several beams. When the testing of these beams was completed it was thought that testing cores would be more beneficial for several reasons. First, the compaction procedure for beams is extremely time consuming and results in beams that have inconsistent densities. Second, much more material is needed for the compaction of beams. Finally, the beams have a high percentage of voids on the surfaces which results in errors while taking rut depth measurements. Therefore, the remainder of this study was concentrated on testing 15.2 cm cores.

The AASHTO specification T 247-89 initially was used to compact cores in the laboratory. This specification called for approximately 20 blows at 1.7 MPa (250 psi) to get the sample into its semi-compacted state. Then 150 blows are applied at 3.4 MPa (500 psi) to complete compaction in the California kneading compactor. This procedure achieved densities approximately 10 pcf higher than those typically occurring in the field with the same gradation and asphalt cement. Several trial batches were prepared and different compaction techniques were used to lower the densities of the test specimens. The following compaction procedure was found to best simulate field densities:

<u>Step</u>	<u>Activity</u>
1)	Batch the total amount of aggregate at the prescribed gradation in a single lift. Total weight of the aggregate, fillers, and asphalt cement should be determined from the desired final unit weight and sample size.
2)	Heat the aggregate and the asphalt cement to the appropriate temperature, using the same temperature for making Marshall samples.
3)	Preheat the molds, kneading compaction foot, static pedestal, steel shim, and all other equipment necessary to mix the asphalt cement.
4)	Thoroughly mix the aggregates and asphalt cement to insure all aggregate surfaces are covered.
5)	Remove the mold from the oven and place it in the kneading compactor. Place a paper disc in the bottom of the mold.
6)	Attach the funnel to the mold and tighten.
7)	Place the preheated asphaltic mix in the feeder trough and transfer one half of the mix into the mold. Tamp the asphaltic mixture 20 times in the center of the mold and then 20 times around the perimeter of the mold with a round-nose steel rod.
8)	Transfer the remainder of the mix to the mold and tamp it in the previous manner.
9)	Apply 100 blows at a pressure of 2.4 MPa (350 psi) for a duration of 0.5 seconds to the loose mix.
10)	Remove the paper disk from the bottom of the sample and set the preheated steel shim and pedestal on top and bottom of the sample, respectively.
11)	Place the mold and specimen back in the oven and reheat for 1 hour.

12) Remove the sample from the oven and place it in the universal testing machine.

13) Apply a load of 4,536 kg (10,000 lb) for one minute and remove from the tester. Allow the sample to cool overnight.

14) Using the Marshall extraction device, place the steel shim on top of the sample and extract the sample from the mold. Allow the sample to cure for a minimum of 24 hours at room temperature.

15) Heat the core to the testing temperature by placing it in the preheating box or the GLWT at least 12 hours prior to testing.

Modifications to the Testing Temperature

The original version of the GLWT was installed in an air tight temperature controlled room. The temperature in the room was maintained at 35°C (95°F), which corresponds to the mean summer temperature in Georgia. The setup restricted the use of the GLWT to places where temperature controlled rooms were available. Having the GLWT located in a room with an elevated temperature also created miserable conditions for technicians to work. To make the GLWT more portable, a temperature controlled environmental chamber was added that could maintain temperatures up to 51.7°C (125°F). The addition of the environmental chamber allowed pavement researchers at GaDOT to raise the testing temperature to correspond with summer pavement temperatures. The GaDOT raised the standard testing temperature in the GLWT to 40.6°C (105°F) [19].

In the research at the University of Wyoming, the 40.6°C testing temperature did not appear to be severe enough to predict pavement rutting. This was determined after comparing laboratory rut depth measurements to field rut depth measurements. Tests were then conducted at 48.9°C (120°F) and after testing only a few cores it was determined that this temperature was too high. All of the samples tested at 48.9°C failed or approached failure. At 46.1°C (115°F), field rut depths correlated well with laboratory rut depths. Therefore, 46.1°C was selected for the remainder of the testing.

Modifications to the Base Plate

Modifications had to be made to the GLWT apparatus prior to testing the 15.2 cm (6 in) cores. New holes were drilled in the base plate so the sample-holding molds could be used with cores instead of beams.

Modifications to the Loaded-Wheel

The original GLWT device had a 2.54 cm (1 in) wide, 7.62 cm (3 in) diameter aluminum wheel fitted with a hard rubber tire. According to Lai and Lee [20], the tire would exert nonuniform contact pressures. To eliminate this problem an inflatable rubber tire was developed in the laboratory to replace the hard rubber tire. The first version was a 20.32 cm (8 in) diameter aluminum wheel with a 2.54 cm diameter high pressure hose wrapped around the rim. After several tests were performed, problems with the new wheel were recognized. Due to the reciprocating action of the arms, the wheel would rapidly accelerate and decelerate at the ends of the beam. This rapid change in momentum would cause excessive rutting on the asphalt surface and premature wear on the rubber tire. For these reasons, this version of the wheel system was abandoned [21].

The loaded-wheel apparatus was once again modified by replacing the rubber tire assembly with an inflated, stationary, rubber hose 2.54 cm in diameter on which the 7.62 cm (3 in) loaded metal wheel traverses. The hose is clamped to brackets at each end of the beam and maintains a longitudinal alignment along the center of the beam. This eliminates most of the excess rutting on the ends of the test specimens caused by the skidding and shoving actions of the tire.

Modifications to the Rut Depth Measuring Device

The original rut profile measuring device was an aluminum channel 7.6 cm wide and 38.1 cm long. The flanges of the channel would rest on the sample holding molds while measurements were taken. Seven slots were machined into the top of the channel at center and in 5.08 cm (2 in) increments off center. Measurements were taken by placing a dial indicator in the machined slots and moving it orthogonally across the wheel path. The largest deformation in each of the seven slots was recorded. This device was not capable of measuring deformations on 15.2 cm cores because the channel was only 7.62 cm wide. Therefore, a similar device was constructed using a 15.2 cm channel. Both of the channels were free to slide along the sample holding molds, which limited the accuracy of the devices. Tabs were welded to the front of the channel that limited the movement of the channels and increased the repeatability dramatically. Another problem that was experienced with this particular design of measuring device was insuring that the dial indicator was perpendicular while transversing the wheel path. Slight deformations in the wheel path would catch the tip of the dial indicator and cause it to tilt and produce inflated readings. With these devices the measurements were subject to human errors.

A measuring device was developed in the laboratory at the University of Wyoming to provide standardized rut depth measurements. The measuring device is a 63.5 cm (25 in) long aluminum dowel, 3.175 cm (1.25 in) in diameter, machined on the ends to slide into the hose clamping brackets. Three dial indicators are permanently attached to the dowel with set screws to take a center measurement and measurements 5.08 cm (2 in) off center in both directions. Figure 11 shows the rut-depth measuring device developed at the University of Wyoming.

Testing Cores in the GLWT

At least 12 hours prior to testing, each core is placed in the preheating box of the GLWT. To begin testing, a preheated core is secured in the temperature controlled GLWT. The measuring device is placed in the hose mounting brackets and initial dial indicator readings are recorded. The measuring device is removed and the rubber hose is placed in the mounting brackets. The hose is inflated and maintained at 690 kPa (100 psi) with a compressor and regulator. With the hose tightened to the mounting brackets, the loaded-wheel is lowered on top of the hose. When the door is closed the testing can proceed. Cycles are recorded with an internal electronic counter. At a preset number of cycles, the loaded-wheel automatically stops. The hose is removed and the dial indicator readings are recorded. Rut depths are recorded at 1,000; 4,000; and 8,000 cycles. A core can be tested in approximately 3.5 hours.

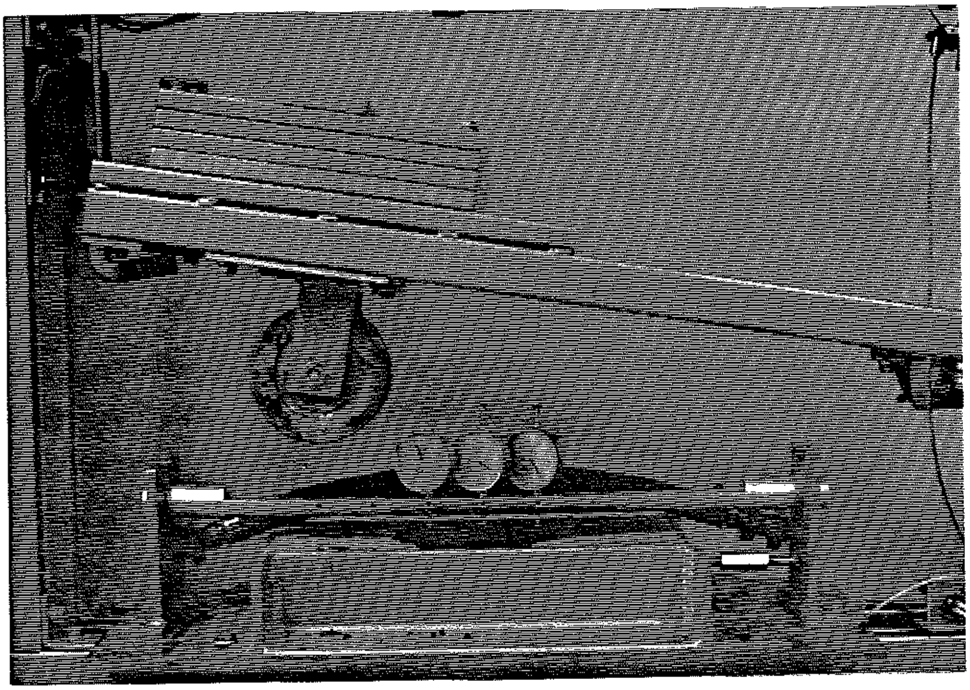


Figure 11. The Modified Rut Depth Measuring Tool

Testing the Repeatability of the GLWT

Following the modifications made to the GLWT at the University of Wyoming, a repeatability study was conducted to determine if the GLWT produced similar rut depths on similar cores. Twelve samples were prepared with an asphalt mix design from a previously constructed interstate project. The aggregates for the mixes were obtained from the same pit used in the project by Wyoming DOT quality control personnel. The aggregate gradation is shown in Table 4. Sinclair AC-20 asphalt cement was used in preparing all of the cores. The standard technique described in Chapter 4 was followed in compacting all the cores. Testing was conducted in the GLWT at 46.1°C (115°F).

Table 4. Aggregate Gradation for the Repeatability Study

Sieve Size	Percent Passing
19.0 mm (3/4 in)	100
12.5 mm (½ in)	99
9.5 mm (3/8 in)	68
4.75 mm (No. 4)	41
2.36 mm (No. 8)	25
600 μm (No. 30)	12
75 μm (No. 200)	4

Table 5 shows the results from the testing after 1,000; 4,000; and 8,000 cycles. The coefficients of variation for the measurements were 0.243, 0.215, and 0.213 after 1,000; 4,000; and 8,000 cycles, respectively. These levels of variance are expected since the cores were hand mixed and individually prepared in the laboratory. The average rut depth increased from 0.230 cm (.091 in) after 1,000 cycles to 0.381 cm (0.15 in) after 8,000 cycles. The standard deviation increased from 0.056 cm (0.022 in) to 0.081 cm (.032 in). This increase in standard deviation was a result of the increased variation of individual samples from the mean rut depth as the average rut depth increased. These results indicate that as the samples were exposed to repeated loadings, the differences became apparent and some samples developed more rutting than others. The results from the repeatability study at 46.1°C (115°F) are shown graphically in Figure 12.

After the repeatability of the GLWT was verified, testing was conducted on cores from the field test sections and a variety of bituminous mixtures. Chapter 5 and 6 discuss the findings from these studies.

Table 5. Average Rut Depth Measurements from the Repeatability Study at 46.1°C (115°F)

	AVERAGE RUT DEPTH AFTER 1000 CYCLES (cm)	AVERAGE RUT DEPTH AFTER 4000 CYCLES (cm)	AVERAGE RUT DEPTH AFTER 8000 CYCLES (cm)
	.211	.312	.373
	.180	.264	.305
	.264	.401	.450
	.168	.198	.221
	.363	.432	.483
	.152	.224	.251
	.221	.323	.401
	.249	.353	.429
	.224	.312	.409
	.234	.335	.419
	.274	.386	.445
	.216	.315	.386
Mean	.230	.321	.381
Coefficient of Variation	.243	.215	.213
Standard Deviation	.056	.069	.081

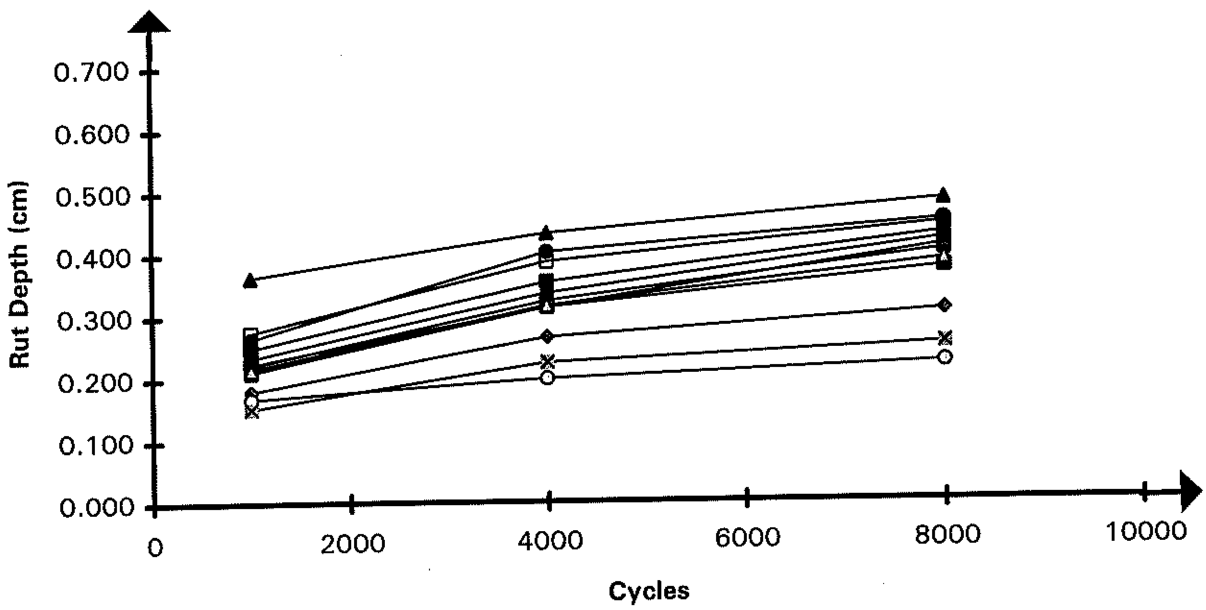


Figure 12. GLWT Results for Cores Tested at 46.1°C (115°F)

Chapter Summary

In this chapter, the modifications that have been made to the GLWT were presented. Several of these modifications were performed at the University of Wyoming. Modifications were made in the following areas: loaded-wheel, rut depth measuring device, test specimen, and testing temperature. The repeatability of the modified GLWT also was examined in this chapter.

CHAPTER 5

CORRELATING FIELD AND LABORATORY PERFORMANCE

Introduction

As described earlier, the Georgia Loaded-Wheel Tester (GLWT) was used at the University of Wyoming to evaluate the susceptibility of various types of asphalt mixes to rutting. After establishing the validity and repeatability of the GLWT, cores were extracted from several primary highways throughout the State of Wyoming and tested in the GLWT. Later, laboratory and actual field rut depth measurements were correlated. In addition, HMA was collected from behind the paver on a primary highway project. The loose HMA was reheated, compacted, and tested in the GLWT. This chapter presents the results from the above tests and provides a comprehensive discussion and analysis of the data gathered.

Selection of Field Test Sections

Thirteen pavement test sections were selected in the State of Wyoming for inclusion in this experiment. Table 6 and Figure 13 summarize the locations of all test sections. These test sections were selected according to their geographic location and rut depth severity level. The sites were selected from several locations in the state to get a representative sample of pavement types and in situ conditions.

Data Collection

In the summer of 1992 and spring of 1993, extensive field data were collected on all test sections included in this experiment. This field evaluation included pavement and subgrade coring, deflection measurements, and condition surveys. The following additional data were collected on each of the sites: field rut depth measurements, age, elevation, equivalent daily 18-kip axle loadings, highest monthly mean temperature, and type of surface treatment.

Table 6. Location of Test Sections

Number on State Map	Project	Roadway	Test Site Milepost
1	P-23-03	US-287	423.5
2	P-25-04	US-26 & US-85	97
3	P-34-09	US-310 & WYO-789	251
4	P-34-10	US-16, 20 & WYO-789	202
5	P-40-13	US-18 & US-20	35
6	P-20-15	US-26 & WYO 789	120
7	P-20-17	WYO 789	96
8	P-30-18	US-26	108
9	P-12-20	US-30	93
10	P-12-21	US-30	61
11	P-12-26	US-30	36
12	P-12-27	US-30	33
13	P-12-28	US-30 & WYO 89	16

The Wyoming version of the South Dakota Road Profiler is normally used by WYDOT to measure rut depths throughout the state. Rut depths for the individual test sites were obtained from the Wyoming Rut Depth Report [24]. The rut depth data were an average of 2,640 measurements per mile. These rut depths were classified in the following three levels:

High: > 0.84 cm (0.33 in)

Medium: 0.38 - 0.84 cm (0.15 - 0.33 in)

Low: < 0.38 cm (0.15 in)

The rut depths for the test sites are shown in Table 7.

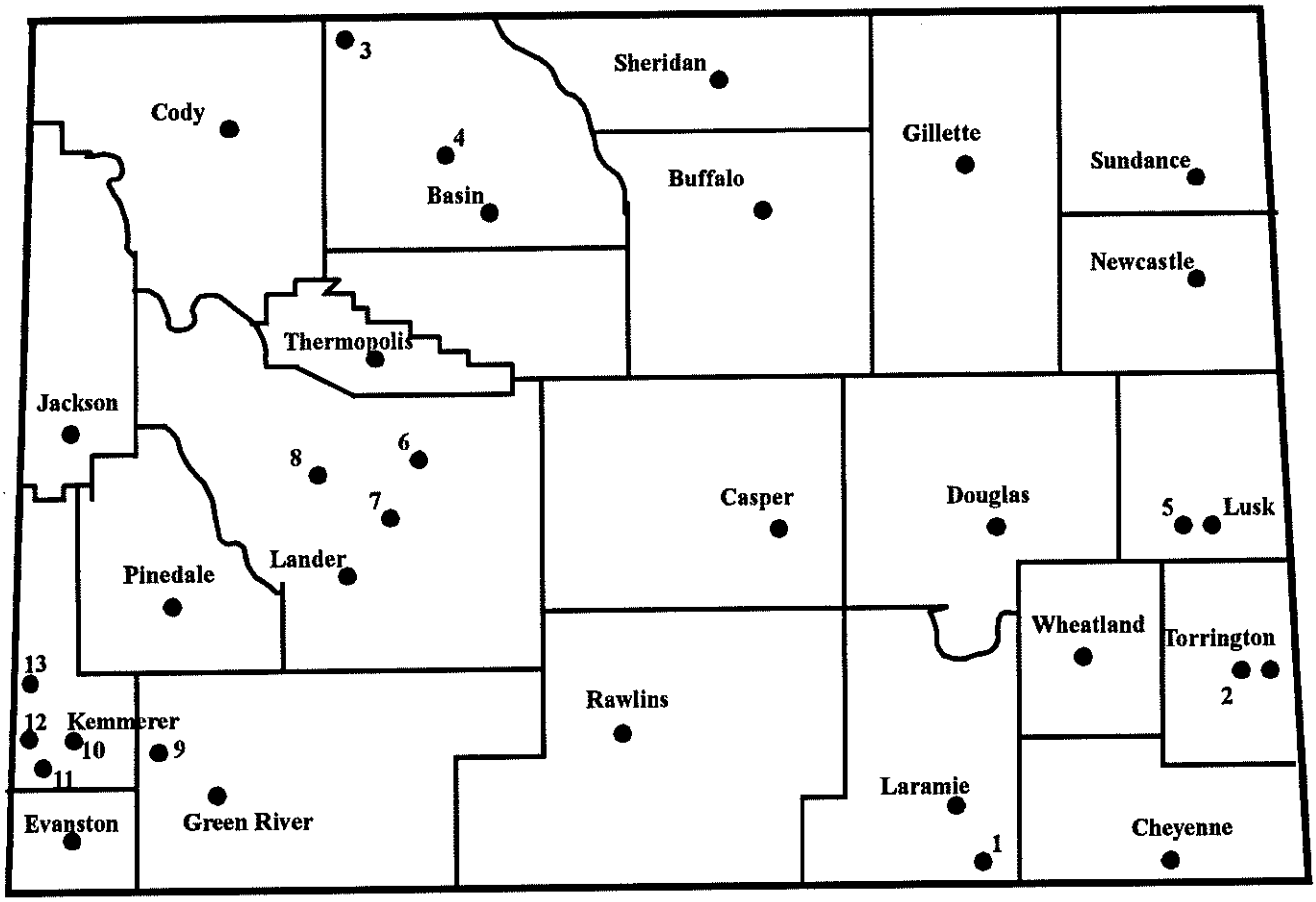


Figure 13. Geographic Location of Field Test Sections

Table 7. Field Conditions for Test Sections

Project	Average Field Rut Depth (cm)
P-23-03	0.18
P-25-04	0.84
p-34-09	0.33
p-34-10	0.28
p-40-13	0.53
p-20-15	0.13
P-20-17	0.46
P-30-18	0.84
P-12-20	0.41
P-12-21	0.53
P-12-26	0.51
P-12-27	0.61
P-12-28	0.46

The traffic volumes for all test sections were collected from the 1993 Vehicle Miles as average annual daily truck traffic [25]. The average annual daily truck traffic volumes were converted to equivalent daily 18-kip axle loads (EDLAs). The traffic volumes for all of the sites are summarized in Table 8. SHRP has designated seven different levels for analysis of traffic volumes [25]. These seven levels of traffic volumes are shown in Table 9. Traffic volumes of all test sections fell in only two of the seven levels. Four of the sites were in Level II, while the remaining sites were in Level III.

The ages and maintenance records for all the sites were collected from data files of the Materials Branch of the Wyoming DOT. In addition, the elevations of all the test sites in Wyoming were extracted from a topographic map. The sites were divided into two categories, 1,158-1,676m (3,800-5,500 ft) and

1,676-2,316 m (5,500-7,600 ft). Table 10 summarizes the ages and elevations of all test sections included in the experiment.

Table 8. Traffic Volumes for Test Sections

Project	Average Annual Daily Traffic	Average Annual Daily Truck Traffic	EDLA'S [1]
P-23-03	4555	1045	199
P-25-04	6475	635	121
P-34-09	1720	240	46
P-34-10	3360	370	70
P-40-13	2110	360	68
P-20-15	3310	620	118
P-20-17	2730	440	84
P-30-18	1265	145	28
P-12-20	3005	995	189
P-12-21	3120	1050	200
P-12-26	2400	830	158
P-12-27	2400	830	158
P-12-28	2550	860	163

[1]EDLA's = Equivalent Daily Loaded Axles

The highest average monthly mean summer temperatures for all the sites were collected from the weather bureau. These temperatures were an average of the highest daily temperatures for the entire month. The month with the highest average temperature for each site was used in this study. The variation in average temperatures among the sites were only a few degrees. Some of the pavements in the study had surface treatments (Plant Mix Wearing Course (PMWC) or chip seal), while others were a single layer of dense graded asphalt. The samples were broken into two categories, those which had some type of surface treatment and those which did not.

Table 9. SHRP Traffic Levels

Traffic Level	EDLA'S [1]
I	< 27
II	27 - 82
III	82 - 274
IV	274 - 822
V	822 - 2,740
VI	2,740 - 8,220
VII	> 8,220

[1]EDLA's = Equivalent Daily Loaded Axles

Testing Field Cores

In the summer of 1992 and spring of 1993, three full-depth cores were extracted from between the wheel paths at each test section. Overall, a typical cross-section of the pavement structure included an asphalt concrete layer, a granular or treated base (asphalt or cement), and the underlying subgrade soil. Initially, the cores were used to determine actual pavement thicknesses and later for testing in the GLWT. Table 11 shows the variations in pavement thicknesses among the test sections. The upper 7.62 cm (3 in) of each core were tested in the GLWT. The bulk specific gravities were determined for all cores using the standard method of test for bulk specific gravity of compacted bituminous mixtures (AASHTO T 166-88). The heights and densities of the cores tested in the GLWT are summarized in Table 12.

Table 10. Ages and Elevations of Test Sections

Project	Age (Years)	Elevations (m/ft)
P-23-03	14	2,316/7,600
P-25-04	15	1,250/4,100
P-34-09	13	1,296/4,250
P-34-10	24	1,160/3,800
P-40-13	7	1,524/5,000
P-20-15	6	1,463/4,800
P-20-17	11	1,524/5,000
P-30-18	51	1,677/5,500
P-12-20	5	1,920/6,300
P-12-21	10	2,134/7,000
P-12-26	6	1,920/6,300
P-12-27	8	1,920/6,300
P-12-28	15	1,860/6,100

The initial set of field cores were tested in the GLWT at 40.6°C (105°F). Statistical analyses concluded that correlation between the GLWT rut depths and field rut depths were inconclusive. Therefore, a second set of field cores were tested at 46.1°C (115°F). Statistical evaluations at this temperature indicated better correlation between field and laboratory rut depths. The statistical results are described in Chapter 5. Table 13 summarizes the rut depth measurements after 8,000 cycles for the cores tested at each temperature. The mean rut depths for all samples after 8,000 cycles were 0.26 cm (0.10 in) and 0.41 cm (0.16 in) at 40.6°C and 46.1°C, respectively.

Table 11. Pavement Thicknesses of Test Sections

Project	Average Pavement Thickness (cm)
P-23-03	17.1
P-25-04	10.8
P-34-09	12.1
P-34-10	14
P-40-13	19.1
P-20-15	15.2
P-20-17	18.4
P-30-18	15.9
P-12-20	13.3
P-12-21	15.2
P-12-26	12.1
P-12-27	11.4
P-12-28	17.1

Correlating Field and Laboratory Rut Depths

After testing the first set of cores at 40.6°C (105°F) attempts were made to correlate field rut depths to those obtained in the GLWT. The cores were analyzed according to age, elevation, traffic volumes, and even geographic location. Statistical analyses were performed on the data using the statistical software package Minitab. Results from the analyses were p-values larger than the α-level of testing (0.05) and low coefficients of multiple determination (R-square). The p-values larger than the α-level supported the fact that field rut depths were not equal to rut depths obtained with the GLWT. The low R-square values indicate that for the regression equations obtained, only small percentages of the variation in the field rut depths is explained with results from the GLWT. For these reasons, it was concluded that the correlations between the GLWT rut depths and field rut depths at 40.6°C (105°F) were inconclusive.

Table 12. Heights and Densities of Field Cores

Project	Cores Tested at 40.6°C (105°F)		Cores Tested at 46.1°C (115°F)	
	Height (cm)	Density (kg/m^3)	Height (cm)	Density (kg/m^3)
P-23-03	7.6	2,265.0	7.6	2,263.4
P-25-04	8.1	2,353.1	8.3	2,332.3
P-34-09	8.4	2,319.5	8.1	2,324.3
P-34-10	7.6	2,338.7	7.0	2,329.1
P-40-13	7.0	2,357.9	7.6	2,337.1
P-20-15	8.3	2,330.7	7.0	2,338.7
P-20-17	7.5	2,329.1	7.3	2,327.5
P-30-18	7.5	2,271.4	8.3	2,295.4
P-12-20	8.1	2,287.4	8.9	2,269.8
P-12-21	7.8	2,277.8	7.6	2,277.8
P-12-26	7.6	2,284.2	7.3	2,293.8
P-12-27	7.6	2,289.0	8.3	2,289.0
P-12-28	7.1	2,319.5	6.4	2,327.5

Attempts were later made to correlate field and laboratory rut depth data for the cores tested at 46.1°C (115°F). Several regression models were used incorporating linear, quadratic, and cubic relationships. However, the regression models obtained had only limited success in describing the variations in the field rut depths with the results from the GLWT. In addition to the low values of coefficient of multiple determination, the majority of the predictor variables were correlated, which skewed the results. For this reason, it was necessary to group the cores according to their individual attributes.

Table 13. Average Laboratory Rut Depths for Field Cores after 8000 Cycles

PROJECT	AVERAGE LABORATORY RUT DEPTH AT 40.6°C (105°F)(cm)	AVERAGE LABORATORY RUT DEPTH AT 46.1°C (115°F)(cm)
P-23-03	0.191	0.452
P-25-04	0.193	0.466
P-34-09	0.213	0.218
P-34-10	0.592	0.445
P-40-13	0.262	0.526
P-20-15	0.406	0.462
P-20-17	[a]	0.528
P-30-18	0.277	0.762[b]
P-12-20	0.191	0.297
P-12-21	0.130	0.201
P-12-26	0.284	0.328
P-12-27	0.086	0.175
P-12-28	0.310	[a]

[a]Cores failed during testing
[b]Test was stopped after rutting exceeded 0.762 cm (0.3 in)

Traffic volumes of all test sections fell in only two of the seven SHRP levels. Four of the sites were in Level II, while the remaining sites were in Level III. Therefore, the traffic volumes were concluded to be similar for all test sections, which resulted in excluding them from the analysis. Since all of the test sections were over five years old it was concluded that the majority of rutting had already occurred. Therefore, the ages of the sites had minimal effects on their performance in the GLWT. The highest monthly mean temperature was recorded at each site from the National Weather Bureau. Based on these temperatures, there was only a variation of a few degrees between the sites. For this reason, the temperature was not considered to be important in predicting rutting in the field.

A study conducted in Colorado with the Hamburg Wheel Tester analyzed 39 different test sites according to their traffic volumes, temperature, and elevation. That study concluded that elevation is a significant factor in predicting the amount of rutting in pavement [26]. Since Colorado shares a common boundary and many topographical similarities with Wyoming, the conclusions from the Colorado study were shared by the University of Wyoming.

The data set was split into two categories based on the elevations of the sections and on pavement surface types. Statistical analyses were then performed separately on each data set. All test sections at elevations between 1,158 m and 1,676 m (3,800 ft and 5,500 ft) were grouped together. There were six sites in this elevation range. The following regression model was obtained for this category:

$$\textbf{Rut Depth} = -3.71 + 1.50*A + 0.461*H \qquad \textbf{(Eq. 1)}$$

where :

Rut Depth : Predicted Field Rut Depth (cm)

A : Average laboratory rut depth after 8000 cycles at 46.1°C (115°F) (cm)

H : Height of the field core tested in the GLWT (cm)

The R-square for this regression model was 92.6 percent. Figure 14 illustrates the relationship between field rut depths and rut depths predicted by this model.

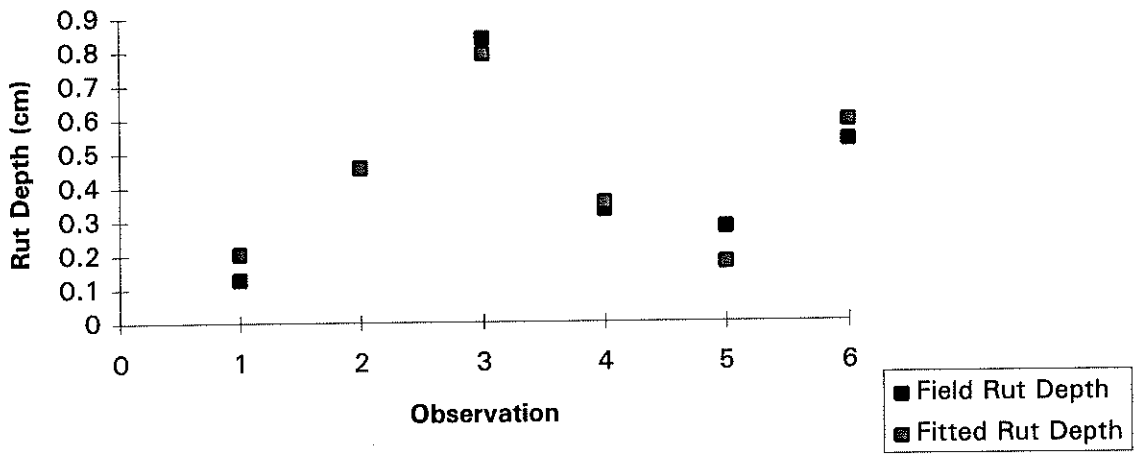


Figure 14. Actual vs. Predicted Rut Depths using Model 1

A similar regression model was developed for elevations between 1,676 m and 2,316 m (5,500 ft and 7,600 ft). The model equation for these higher elevations is:

Rut Depth = 0.776 + 0.39*A - 1.39*B **(Eq. 2)**

where :

Rut Depth : Predicted Field Rut Depth (cm)

A : Average laboratory rut depth after 8,000 cycles at 46.1°C (115°F) (cm)

B : Center laboratory rut depth after 8,000 cycles at 46.1°C (115°F) (cm)

The R-square coefficient for this model was 91.9 percent. Five projects were used in the determination of this regression model.

Next, the field test sections were divided into two categories according to their surface type. The cores were classified as either having a surface treatment (wearing course or chip seal) or not having a separate surface treatment (single layer of dense graded asphalt). Statistical analysis was first performed on the data from cores with no surface treatments. The following regression equation resulted from this analysis:

$$\textbf{Rut Depth} = -4.32 + 1.63 \cdot A + 0.53 \cdot H \qquad \textbf{(Eq. 3)}$$

where :

Rut Depth : Predicted Field Rut Depth (cm)

A : Average laboratory rut depth after 8,000 cycles at 46.1°C (115°F) (cm)

H : Height of the field core tested in the GLWT (cm)

The R-square coefficient for this model was 97.3 percent. Four cores were used in the determination of this regression model. Finally, similar analysis was performed on test sections with surface treatments. The following linear model was found:

$$\textbf{Rut Depth} = 0.784 + 0.59 \cdot A - 1.60 \cdot B \qquad \textbf{(Eq. 4)}$$

where :

Rut Depth : Predicted Field Rut Depth (cm)

A : Average laboratory rut depth after 8,000 cycles at 46.1°C (115°F) (cm)

B : Center laboratory rut depth after 8,000 cycles at 46.1°C (115°F) (cm)

The R-square coefficient for this model was 93.4 percent. Seven test sections were used in the determination of this regression model. The previous models have been developed and recommended for cores between 7.0 and 8.9 cm high.

Problems Experienced while Testing Field Cores

Problems developed while testing a few of the surface treated cores in the GLWT. Some of the more rigid surface treatments fractured. In these cases, longitudinal cracks developed on the surface of the cores adjacent to the hose after repeated loadings. If cracking developed in the early stages of testing then the rutting was a function of structural failure instead of plastic flow. Another problem was noticed with cores that had extremely rough surfaces. When repeated cycles were applied to the samples some of the larger surface aggregates tilted or shifted. The rut depth measuring devices takes measurements at identical locations each time. The movements possibly inflated the initial dial indicator readings which caused all successive rut depth measurements to be inflated. The rut depth measurements in these samples are masked by the shifting surface aggregate.

Analyzing HMA with Observed Field Performance

In the spring and summer of 1993, there was a complete reconstruction of five miles of primary highway US-19 between Pinedale and Daniel Junction. A typical cross-section of the new roadway consists of 24 inches of imported sub-base, six inches of crushed base, four inches of Type 1 (3/4 in.) Plant Mix Pavement (PMP), and 3/4 inch Plant Mix Wearing Course (PMWC). Quality Control personnel from WYDOT collected samples of the PMP directly from the paver and recorded the locations where each sample was taken. This HMA was then taken to the University of Wyoming for testing in the GLWT. Initially, the HMA had to be reheated to a temperature sufficient to be workable. Then the appropriate amount of material to compact four 7.62 cm (3 in) high, 15.24 cm (6 in) diameter cores was separated from the entire sample. This material was then broken into four separate samples, sealed in air tight containers, and heated to approximately 132°C (270°F) prior to compaction. The standard compaction procedure described in Chapter 4 was followed for each of the cores. After the cores were cured for several days, they were heated for a minimum of 12 hours at 46.1°C (115°F) prior to testing in the GLWT. Rut depth measurements were recorded at 1,000; 4,000; and 8,000 cycles for each core. Table 14 shows the density of each core in addition to the average rut depth developed after 8,000 cycles.

Table 14. Densities and Rut Depths for Laboratory Compacted HMA

Sample	Density (kg/m^3)	Rut Depth after 8000 Cycles at 46.1°C (cm)
1	2,388.4	0.168
2	2,386.8	0.249
3	2,375.5	0.338
4	2,370.7	0.058

The next step in this investigation was to determine the amount of field rutting the test section experienced after one year of service. Field rut depth data were collected with the Wyoming version of the South Dakota Road Profiler. Rut depth measurements were taken every 0.61 m (2 ft) and averaged over 0.161 km (0.1 mi) for recording. These values were averaged for the length of the entire reconstruction. The average field rut depth was compared to the average GLWT rut depth using the two-sided t-test. The results from this analysis are summarized in Table 15. These results indicate that the sample mean rut depths are equal at an α-level = 0.05.

Table 15. Summary of Rut Depth Data

GLWT Rut Depth (cm)	Field Rut Depth (cm)	t-statistic (α = 0.05)	t*	Conclusion
0.203	0.241	3.182	0.855	$\mu_1 = \mu_2$

The results obtained from the GLWT are a good representation of the rut depths that are naturally occurring on this project. Since the highway project has been in service one complete year, the majority of the rutting has most likely occurred. Therefore, it can be concluded that the GLWT can be used as a useful tool in predicting field performance of bituminous pavements prior to construction.

Chapter Summary

In this chapter, a statistical analysis was performed on rut depth measurements collected on laboratory compacted and field cores. Regression models were developed for predicting rut depths of asphalt mixes based on laboratory testing in the GLWT at 46.1°C (115°F). In addition, one primary project was selected for special evaluation in this research project. Loose HMA was collected from behind the paver on this project for testing with the GLWT. The GLWT successfully predicted the rutting that developed on the primary highway project.

Chapter 6

EVALUATING SOMAT AND SMA MIXES

Introduction

Pavement engineers are continuously looking for methods to improve the performance of asphalt mixtures. Improvements can be accomplished by modifying the quality and characteristics of aggregate or asphalt cement.

Shale Oil Modified Asphalt Treatment (SOMAT) has been used to modify asphalt cements for nearly a decade. According to Paraho, the manufacturer, Shale Oil Modified (SOM) asphalt cements are more resistant to stripping, thermal cracking, fatigue cracking, rutting, and aging than conventional asphalt cement [8].

Stone Matrix Asphalt (SMA) mix was developed in Europe more than 20 years ago. It is a gap-graded hot mixture that maximizes the binder and coarse aggregate content [3]. SMA has been shown to have a high resistance to rutting. Currently, several highway agencies in the United States are experimenting with SMA mixes. The Wyoming DOT built several pavement test sections with SMA and SOMAT modified mixes. This chapter evaluates the rut resistance of these new asphalt mixes with the GLWT and other testing instruments.

Evaluation of SOMAT

In the summer of 1994, WYDOT constructed two recycled asphalt test sections on Interstate 90 near Moorcroft. One of the test sections had a Shale Oil Modified (SOM) asphalt cement while the other section had a non-modified asphalt cement. Both sections were 60 percent virgin material and 40 percent recycled material. The gradation for the recycled asphalt pavement is shown in Table 16.

Table 16. Recycled Asphalt Pavement Gradation

Sieve Size	Percent Passing	
	Recycled	Target
25.4 mm (1 in)	100	100
19.0 mm (3/4 in)	96	90 - 100
12.5 mm (½ in)	78	60 - 85
4.75 mm (No. 4)	43	46 - 60
2.36 mm (No. 8)	29	25 - 45
600 µm (No. 30)	16	10 - 30
75 µm (No. 200)	5.6	2 - 7

Each of the test sections had a design asphalt content of 4.5 percent of which 2 percent was provided by the recycled asphalt pavement. The modified mix had 2.5 percent SOM which consisted of 10 percent SOMAT and 90 percent virgin Sinclair AC-20. The non-modified mix had 2.5 percent virgin Sinclair AC-20 to account for its remaining asphalt cement. The virgin aggregates for the test sections were taken from the Fisher Sand and Gravel Pit in Sundance. The aggregate gradation is shown in Table 17.

Loose HMA samples were collected from behind the paver on both test sections. These samples were delivered to the University of Wyoming for testing in the GLWT. Each sample was initially heated to a temperature that would allow the samples to be separated into smaller portions. The appropriate amount of material to compact four 7.62 cm (3 in) high, 15.2 cm (6 in) diameter cores was obtained from each field sample. This material was then broken into four separate samples and heated to approximately 132°C (270°F) prior to compaction. Once heated, the asphalt mixtures were compacted with both the California Kneading Compactor and a static compaction device as described in Chapter 4 of this report.

These samples were allowed to cool overnight before they were extracted from the molds. Upon extraction, the bulk specific gravity was determined by AASHTO T166. All samples were then tested in the GLWT at 46.1°C (115°F). Rut depth measurements were taken after 1,000; 4,000; and 8,000 cycles. These results are shown in Table 18.

Table 17. Virgin Aggregate Gradation for SOMAT Test Sections

Sieve Size	Percent Passing
25.4 mm (1 in)	100
19.0 mm (3/4 in)	95
12.5 mm (1/2 in)	79
9.5 mm (3/8 in)	65
4.75 mm (N. 4)	42
2.36 mm (No. 8)	29
1.18 mm (No. 16)	22
600 μm (No. 30)	18
300 μm (No. 50)	13
150 μm (No. 100)	10
75 μm (No. 200)	8

Testing at the University of Wyoming with the GLWT shows that the SOMAT mixture provides only minimal improvements in resisting rutting. The average rut depth for the (SOMAT) modified samples after 8,000 cycles was 0.424 cm (0.167 in) while the average rut depth for the non-modified mix was 0.460 cm (0.181 in). The standard deviations were 0.0967 cm (0.0381 in) for the modified mixture and 0.0570 cm (0.0224 in) for the non-modified mixture. Using a standard t-statistic, a difference between the mixes is only noticed with α-level > 0.57. The results from the t-test are shown in Table 19.

Table 18. **GLWT Results on SOMAT and Non-SOMAT Mixtures**

	Rut Depth (cm)		
Cycles	**1000**	**4000**	**8000**
SOMAT Mix	0.256	0.354	0.424
S.D.	0.079	0.088	0.097
Non-SOMAT Mix	0.287	0.395	0.460
S.D.	0.066	0.061	0.057

Table 19. Statistical Results from the SOMAT Study

α-level	t*	t(1-α/2, n-1)	**Conclusion**
0.10	0.642	1.943	Ho: $\mu_1 = \mu_2$
0.57	0.642	0.642	Ho: $\mu_1 = \mu_2$
0.60	0.642	0.553	Ha: $\mu_1 \neq \mu_2$

Cost Evaluation of SOMAT Mixes

Pavement engineers must select appropriate material types for particular projects. These selections are heavily influenced by cost. In many cases, the additional costs associated with expensive additives are not justified because the benefits are minimal. In other cases, money invested initially in additives will lead to an increased pavement service life.

Costs for Shale Oil Modified and non-modified asphalt cements were obtained from the 1994 Wyoming Construction Weighted Average Bid Prices. The costs of these two binders are shown in

Table 20. The costs shown in Table 20 are misleading and indicate that SOMAT mixes are more expensive. However, due to the excellent adhesive properties of the SOM binder, no lime is necessary in the mix design. The absence of the lime saves material, labor, and equipment costs, which consequently makes the SOMAT mix cost nearly identical to the cost of standard dense graded asphalt mixes. Monitoring the long term performance of the test sections will provide better information about the cost effectiveness of SOMAT.

Table 20. Cost Comparison of SOMAT and Dense Graded Asphalt

Item	Unit of Measurement	Cost Per Unit
AC-20	Ton	$121.64
Shale Oil Modified AC-20	Ton	$190.00

Evaluation of SMA Mixes at the University of Wyoming

In the summer of 1994, the Wyoming DOT constructed a Stone Matrix Asphalt (SMA) test strip on Interstate 80. The test strip included six different sections. Although each section had a different mix design, the same aggregate source and gradation were used in all mixes. The aggregate gradation used in this experiment is shown in Table 21. The layout for the test strip and the components of the experimental mixes are shown in Figure 15 and Table 22, respectively. The main objective of the test strip was to evaluate the rut resistance provided by SMA mixes and to determine the effects of cellulose fibers, SBS polymer modified AC-20, and asphalt content on pavement performance. Some of the testing performed on the test sections was done on field cores while other testing was done on laboratory compacted samples.

Table 21. SMA Aggregate Gradation for the Test Strip

Sieve Size	Percent Passing
19.0 mm (3/4 in)	100
12.5 mm (1/2 in)	81
9.5 mm (3/8 in)	63
4.75 mm (No. 4)	35
2.36 mm (No. 8)	28
600 µm (No. 30)	21
300 µm (No. 50)	17
75 µm (No. 200)	10

Table 22. Characteristics of the Test Strip Mixes

Test Section	Milepost		Asphalt (%)	Fiber	Polymer
	From	To			
1	140.45	140.96	4.4	NO	YES
3	138.74	139.25	5.6	YES	YES
4	138.24	138.74	5.6	YES	NO
5	140.19	140.45	5.9	YES	YES
6	139.93	140.19	5.8	NO	YES

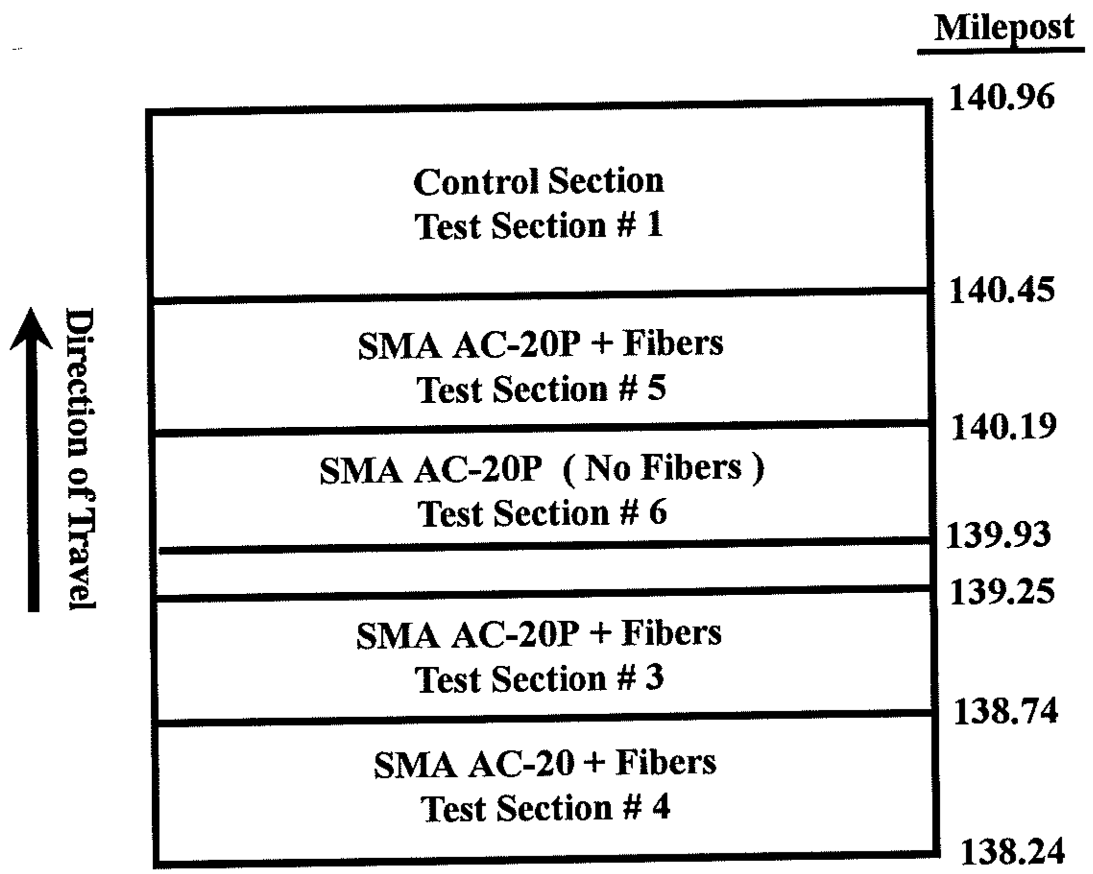


Figure 15. Wyoming I-80 SMA Test Strip

Results from Testing Laboratory Compacted SMA Cores in the GLWT

Loose HMA samples were collected from behind the paver on test sections 1 and 3. The HMA was reheated, compacted, and then tested in the GLWT. The compaction procedure discussed in Chapter 4 was followed in the preparation of all samples.

Three cores were compacted from each of the samples to represent the two HMA test sections. The densities of the cores were measured according to AASHTO T166. Prior to testing, the cores were heated to 46.1°C (115°F) for a minimum of 12 hours. The GLWT rut depth results and densities for the laboratory compacted cores are presented in Table 23.

Table 23. Results from Laboratory Compacted SMA Cores Tested in the GLWT after 8000 Cycles

	Rut Depths (cm)	
Sample	**Control Mixture (Section 1)**	**AC-20P +Fibers (Section 3)**
1	0.279	0.201
2	0.145	0.198
3	0.196	0.267
Average Rut Depth	0.207	0.222
S.D.	0.0676	0.0390
Average Density (kg/m^3)	2308	2366

Based on results from the GLWT, both asphalt mixes have comparable rut resistance. The SMA test section 3 exhibited slightly higher rutting than the Control Mixture. The asphalt content of this SMA mix was 1.2 percent higher than the asphalt content of the Control Mixture which gives the SMA mix more resistant to raveling, stripping, and oxidation. Therefore, based on factors other than rutting, the SMA mixtures are generally more durable than the Control Mixture.

Paired statistical analyses were performed on the two mixtures to determine if their mean rut resistances were statistically different. The standard t-test was used to evaluate the mixes. The critical t-value was t (0.95;4) = 2.132 from the table of the t-distributions [27]. The t-statistic was calculated to be

0.333. Since the t-statistic was less than the required t-value the null was accepted. Therefore, the rut resistances of the mixtures were not significantly different at $\alpha = 0.10$.

Results from Testing SMA Field Cores in the GLWT

In addition to testing SMA laboratory compacted samples, 25.4 cm (10 in) cores were extracted from all test sections. In the laboratory, these 25.4 cm cores were cored again to obtain 15.2 cm (6 in) cores which could be tested in the GLWT. Each core was heated for a minimum of 12 hours prior to testing in the GLWT. The average rut depth results after 8,000 cycles and the densities of the cores are shown in Table 24.

Table 24. Results from SMA Field Cores Tested in the GLWT after 8,000 Cycles

Test Section	Description	Average GLWT Rut Depth (cm)	Average Density (kg/m^3)
1	Control - AC-20P	0.210	2327
3	SMA - AC-20P + Fibers	0.304	2356
4	SMA - AC-20 + Fibers	0.596	2354
5	SMA - AC-20P + Fibers	0.394	2393
6	SMA - AC-20P	0.325	2360

Comparison of Field and Laboratory Compacted Cores

It is clear from Tables 23 and 24 that the densities of the field and laboratory compacted cores are very similar. The maximum observed difference in density between field and laboratory compacted samples is 19 kg/m^3 (1.2 pcf).

The average rut depths obtained by the GLWT at 46.1°C (115°F) for the laboratory compacted and field control section cores were 0.207cm (0.082 in) and 0.210 cm (0.083 in). These rut depths are considered equal. The average rut depths for the laboratory compacted and field cores from test section 3 were 0.222 cm (0.087 in) and 0.304 cm (0.120 in). Although the control mix showed slightly higher rut resistance than the SMA mix, both mixes passed the GLWT test.

Evaluating the Effects of SBS Polymer Modified AC-20 on SMA Mixes

SBS Polymer Modified AC-20 was used in test sections 1, 3, 5, and 6. The only test section that contained unmodified AC-20 was section 4. The increased rut resistance provided from the modified binder can be determined by comparing sections 3 and 4. These two sections have the same percentages of asphalt and their densities are almost identical. The average rut depth for section 4 was 0.596 cm (0.235 in). One of the cores in section 4 experienced a rut depth of 0.706 cm (0.278 in), which is on the threshold of failure. The average rut depth for section 3 was 0.304 cm (0.120 in). The rut depths that developed in section 3 are only 51 percent of those in section 4. Therefore, it is apparent that the SBS Polymer Modified AC-20 is superior to the virgin AC-20 in controlling permanent deformations in SMA mixes.

Evaluating the Effects of Asphalt Content on SMA Mixes

Increasing the asphalt content of bituminous mixes beyond the optimum amount is the leading cause of permanent deformations. Additional binder added to an asphalt mix leads to thick films of asphalt cement coating the aggregates. As pavement temperatures increase, the viscosity of the asphalt cement is reduced. Thick films of softened asphalt do not possess enough strength to support heavy vehicular loadings. For this reason, asphalt cement is forced from between the aggregates and causes the mixes to consolidate or develop permanent deformations. Problems also develop when too little binder is added to asphalt mixes. Mixes that lack the proper amount of asphalt cement to thoroughly coat the aggregate are susceptible to raveling as well as oxidation. Some mix designs allow asphalt percentages to fluctuate while other mix designs are very sensitive to deviations from the optimum percent.

In the SMA field experiment, sections 3 and 5 were used to analyze the effects of varying the asphalt content. These sections are both SMA mixes with fibers and SBS Polymer Modified AC-20. Table 25 shows the design and actual field asphalt contents for all of the test sections.

Table 25. Asphalt Contents for the SMA Test Strip

Test Section	Design AC %	Field AC% (Based on Extractions)
1	4.25	4.40
3	6.00	5.60
4	5.75	5.60
5	6.00	5.90
6	5.75	5.80

The design asphalt content for sections 3 and 5 was 6 percent. However, section 3 had an asphalt content of 5.6 percent while section 5 had an asphalt content of 5.9 percent. As shown in Table 24, the average rut depth which developed in section 5 after 8,000 cycles was 0.394 cm (0.155 in). Section 3 experienced rut depths of 0.304 cm (0.120 in) at the same level of loadings. These results indicate that field compacted SMA mixes may be sensitive to fluctuations in asphalt content and mixes with higher asphalt contents experience higher rut depths.

Evaluating the Effects of Cellulose Fibers in SMA Mixes

SMA mixes are gap graded with higher percentages of large aggregate and filler material than standard dense graded mixes. The large aggregates form a "stone skeleton" in the asphalt mix that is the underlying benefit of SMA mixes. The voids between the large aggregates are filled with the filler material and asphalt cement. When the temperature of the pavement increases the asphalt cement flows between the large aggregates and out of the mix. The fibers behave like webbing throughout the SMA mixes to hold the asphalt cement in the mix.

Test sections 5 and 6 were used to determine the effects of cellulose fibers on rut resistance of SMA mixes. Test section 5 had fiber while test section 6 did not. The average rut depth of test section 5 after 8,000 cycles was 0.394 cm (0.155 in) while the rut depth of test section 6 was 0.325 cm (0.128 in) under similar testing conditions. These results indicate that the fibers do not increase the resistance to rutting. The benefit of the fibers is more important in confining the asphalt cement in the mixture.

Performance of the Control Mixture

Testing at the University of Wyoming with the GLWT concluded that the Control mix had higher resistance to permanent deformations than the SMA mixes. This was concluded in both the laboratory compacted HMA study and the extracted field core investigation. The reason for the high rut resistance was the low percentage (4.4 percent) of binder compared to 5.6-5.9 percent with the SMA mixes. Typically, dense graded asphalt mixes have asphalt contents of approximately 5 to 5.5 percent. Even though the control mix is superior in resisting rutting to SMA, problems are expected to occur in the control mix after aging. This mix will be prone to raveling, stripping, and oxidation which are common among mixes with low percentages of asphalt cement. Therefore, the service life of the control mixture will certainly be shorter than those of the SMA mixtures.

SMA Testing Performed at the Colorado Department of Transportation

As part of the SMA investigation, 18 five-gallon buckets representing the five asphalt mixes at the Wyoming SMA test strip were shipped to the Colorado Department of Transportation (CDOT) for testing in the French Rutting Tester, the Hamburg Wheel-Tracking Device, and the Thermal-Stress Restrained-Specimen Test (TSRST). The following sections summarize the findings of these tests.

Testing in the Hamburg Wheel-Tracking Device

At CDOT, the SMA mixes were reheated and compacted with the Linear Kneading Compactor. Separate samples were tested in the Hamburg Wheel-Tracking device at 45°C, 50°C, and 55°C (113°F, 122°F, and 131°F). The samples were tested at all three temperatures to determine the temperature susceptibility of the individual mixes. At 55°C, samples which are prone to stripping will strip in the Hamburg Tester. The results from the Hamburg Wheel-Tracking Device are shown in Table 26.

Table 26. Results from the Hamburg Wheel-Tracking Device

	Rut Depths (mm)		
Test Section	**45°C (113°F)**	**50°C (122°F)**	**55°C (131°F)**
1	1.2	1.3	3.0
3	1.8	2.4	6.9
4	4.9	10.6	>20
5	1.9	3.6	4.9
6	1.5	2.2	>20

Rut depths exceeding 10 mm (0.394 in) after 20,000 passes in the Hamburg Wheel-Tracking Device at 50°C (122°F) are considered failing in Colorado. As shown in Table 26, test section 4 is the only section which fails in this device. In addition to evaluating the rut susceptibility of asphalt mixes, the Hamburg device can also alert designers of mixes which are susceptible to stripping. Testing at 55°C (131°F) shows that test sections 4 and 6 developed excessive rutting due to stripping. These sections were more susceptible to stripping than the other mixes. These results illustrate that test section 4 is an extremely poor mix design and could be a high maintenance mix design. Observation of this test section in the future will determine if this section is actually a high maintenance mix.

Testing in the French Rutting Tester

At CDOT, the SMA mixes were reheated and compacted in the Laboratoire Central des Pons et Chaussees (LCPC) Plate Compactor. The samples were then tested in the French Rutting Tester. Rut depth measurements in the French device are measured as a percentage of the slab thickness. Rut depths exceeding 10 percent of the slab thickness after 30,000 cycles at 60°C (115°F) are considered failing. The results from this study are shown in Table 27. According to Table 27, none of the mixes failed in the French Rutting Tester.

Table 27. Results from the French Rutting Tester

Test Section	Rut Depth (% Slab Thickness)
1	3.3
3	3.4
4	7.9
5	5.2
6	4.6

Summary of the Results from Testing Performed at CDOT & UW

The results from testing the SMA mixes at CDOT are summarized in Table 28. Also included in Table 28 are the results from the SMA field cores tested in the GLWT at the University of Wyoming. The numbers in parenthesis next to the rutting results are the order in which each mix performed, from best to worst, in each testing device.

Table 28. Summary of the Results from Testing at CDOT & UW

		Rut Depths			
		UW	CDOT		
Test Section	Description	GLWT (mm)	Hamburg (mm)	French (%)	TSRST (kPa)
1	Control - AC-20P	2.1 (1)[a]	1.3 (1)	3.3 (1)	1975
3	SMA - AC-20P + Fibers	3.04 (2)	2.4 (3)	3.4 (2)	2794
4	SMA - AC-20 + Fibers	5.96 (5)	10.6 (5)	7.9 (5)	2341
5	SMA - AC-20P + Fibers	3.94 (4)	3.6 (4)	5.2 (4)	3011
6	SMA - AC-20	3.25 (3)	2.2 (2)	4.6 (3)	3243

[a]Ranking of rut resistance
(1) = Most favorable
(5) = Least favorable

The results obtained after ranking the samples are very interesting. There is a 100 percent correlation in the ranking of rut depths obtained with the GLWT and the French Rutting Tester. Using the Spearman's Coefficient of Rank Correlation it was calculated that there was 100 percent correlation. There also is an excellent correlation between the GLWT and the Hamburg Wheel-Tracking device. Using the Spearman's Coefficient of Rank Correlation it was calculated that there was 99.1 percent correlation. It is noticed that the Control mixture has the best performance in all of the rut testing devices. However, when the Control mixture was tested in the TSRST it was the most susceptible to low-temperature and fatigue cracking. This can be attributed directly to the low asphalt content of this mix. These results verify the conclusions mentioned earlier in this chapter that the Control mixture would develop distresses due to the low percentage of asphalt cement. The SMA mixes, which had significantly higher percentages of asphalt than the Control mixture, had better performance in the TSRST. Test section 4, which was the SMA mixture with non-modified AC-20 and fibers, had the worst performance in all of the rut testing devices.

Cost Evaluation of the SMA Mixes

Most of the SMA mixes performed well in the GLWT, however, the Control mixture performed the best. On the other hand, results from the TSRST indicate that the Control mixture may be susceptible to low-temperature cracking and fatigue cracking. In terms of service life, the SMA mixes will last longer and have better overall performance. For this reason, a cost evaluation must be performed to determine if the additional service life out weighs the costs associated with the SMA mixes. The costs for the SMA and standard Wyoming type I dense graded asphalt are shown in Table 29.

Table 29. Costs of SMA and Wyoming Type I Asphalts

Item	Unit of Measurement	Cost Per Unit
Type 1 Hot Plant Mix Pavement	Ton	$17.39
Stone Matrix Asphalt	Ton	$41.00

The cost of the SMA mixes are 235 percent more than a standard HMA. The price of the SMA mixes is exceptionally high for a couple of reasons. First, all of the SMA pavements constructed in

Wyoming were test sections. These test sections are highly variable and short in length compared to normal paving procedures. Therefore, the contractor increased the unit cost to account for any unexpected mishaps. If SMA mixes became a common mix their cost would decrease substantially. The second reason for the higher costs of SMA mixes is due to the aggregate gradation. The gap graded mix design increases the percentages of large aggregates and fine material. This gradation produces much more wasted aggregate than standard HMAs. The cost associated with this waste must be absorbed by the contractor.

Chapter Summary

This chapter summarized the results from two studies. The first study investigated the reduction in rut depth associated with using SOMAT. The second study concentrated on evaluating SMA mixes. Field and laboratory compacted cores were both tested in the laboratory. Finally, rut depth results from the GLWT on several SMA mixes were compared to results obtained with other testing devices.

Rut depth results from the GLWT correlated well with the results obtained with the French Rutting Tester and the Hamburg Wheel-Tracking Device. This result is important considering that both of the European Testing Devices cost a minimum of $100,000 including the compaction device while the GLWT costs $11,000.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

This research project was performed in two phases. In the first phase, the original GLWT was modified to test 15.2 cm (6 in) cores. In addition, a compaction procedure was developed for preparing cores. Next, a laboratory study was performed to determine the repeatability of the modified GLWT. In the second phase of this research project, GLWT results and field rut depths for several selected sites throughout Wyoming were correlated. In addition, the GLWT was used to determine the effectiveness of SOMAT in improving the rut resistance of asphalt concrete mixes. Finally, SMA mixes were tested in the GLWT and other more expensive European testing devices. Correlation studies were later performed on all test results.

Conclusions from Phase I

The following conclusions can be drawn from the first phase of this research:

1. The Georgia Loaded-Wheel Tester can be used for testing 15.2 cm (6 in) pavement cores instead of beams. Obtaining cores from field sections is much easier than obtaining undisturbed beams. Therefore, the modified GLWT can be used more effectively in the future.
2. Concrete spacers are beneficial for confining cores in the modified GLWT because they do not develop deformations and they are reusable. Slight fluctuations in core heights can be accommodated by casting several spacers of varying height prior to testing.
3. The laboratory compaction process developed at the University of Wyoming for preparing cores produces densities similar to those obtained in the field.

4. Testing conducted at 48.9°C (120°F) in the GLWT was found to be too severe for most asphalt mixes. Therefore, testing at the University of Wyoming was conducted at 46.1°C (115°F).

Conclusions from Phase II

The following conclusions can be drawn from the second phase of this research:

1. Rut depth measurements from the GLWT at 40.6°C (105°F) did not correlate well with actual field measurements on test sections.
2. Rut depth measurements from the GLWT at 46.1°C (115°F) correlated with actual field rut depths after considering factors such as elevation and pavement surface type.
3. The pavement test section with unacceptable field rut depth failed in the GLWT, which confirms the capabilities of the GLWT in predicting poor asphalt mixes.
4. SBS Polymer Modified AC-20 is superior to virgin AC-20 in reducing the amount of rutting in SMA mixes. SMA mixes also are sensitive to fluctuations in asphalt contents. Mixes with higher percentages of asphalt cement tend to experience more rutting.
5. Cellulose fibers added to SMA mixes are useful confining the asphalt cement in the mixture. However, they should not be used as a rut inhibitor.
6. Ranking results from the GLWT on SMA mixes correlated 100 percent with the results from the French Rutting Tester. There also were excellent rank correlations between the GLWT and the Hamburg Wheel-Tracking Device.
7. The GLWT is an inexpensive testing device that can produce quick results comparable to results obtained by more expensive instruments.

Recommendations

1. Because some of the field cores tested in the GLWT had surface treatments, which caused irregular rut depth measurements, future studies should investigate removing these surface treatments prior to testing.
2. An investigation should be conducted to determine the feasibility of incorporating the GLWT in asphalt mix design procedures.
3. Future studies should investigate the effects of tire pressure on rutting in hot mix asphalts.

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APPENDICES

APPENDIX A

GLWT Data Sheets

APPENDIX A1

Repeatability Study Data Sheets

Core #: 20

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 3-23-94

Compaction Procedure:
100 @ 350 psi (kneading)
10,000 lb static levelling load (1 min)

Gmb Calculations
A = 3209.4 g
B = 3235.4 g
C = 1813.8 g
Gmb = A/(B-C) = 2.26

Density = Gmb * 62.4pcf = 140.9 pcf

Testing Date: 4-5-94
Testing Temperature: 115 F
Height: 3 1/8 in.

Comments:
st. dev.=0.0166

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.527	0.547	0.503	0	0	0
1000	0.451	0.456	0.42	0.076	0.091	0.083
4000	0.416	0.408	0.385	0.111	0.139	0.118
8000	0.382	0.382	0.371	0.145	0.165	0.132

Core #: 21

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 3-24-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3210.4 g

B = 3247.4 g

C = 1820.8 g

Gmb = A/(B-C) = 2.25

Density = Gmb * 62.4pcf = 140.4 pcf

Testing Date: 4-6-94

Testing Temperature: 115 F

Height: 3 1/8 in

Comments:

st. dev.=0.0295

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.497	0.525	0.501	0	0	0
1000	0.441	0.456	0.412	0.056	0.069	0.089
4000	0.42	0.423	0.369	0.077	0.102	0.132
8000	0.407	0.403	0.352	0.09	0.122	0.149

Core #: 22

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 3-24-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3212.4 g

B = 3234.4 g

C = 1828.8 g

Gmb = A/(B-C) = 2.29

Density = Gmb * 62.4pcf = 142.6 pcf

Testing Date: 4-19-94

Testing Temperature: 115 F

Height: 3 1/8 in

Comments:

st. dev.=0.00721

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.497	0.506	0.481	0	0	0
1000	0.424	0.389	0.36	0.073	0.117	0.121
4000	0.341	0.345	0.325	0.156	0.161	0.156
8000	0.328	0.323	0.302	0.169	0.183	0.179

Core #: 36

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 5-10-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3201.4 g

B = 3230.4 g

C = 1830.8 g

Gmb = A/(B-C) = 2.29

Density = Gmb * 62.4pcf = 142.7 pcf

Testing Date: 6-14-94

Testing Temperature: 115 F

Height: 3 1/16 in

Comments:

st. dev. = 0.0084

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.547	0.577	0.562	0	0	0
1000	0.469	0.521	0.497	0.078	0.056	0.065
4000	0.462	0.504	0.486	0.085	0.073	0.076
8000	0.45	0.494	0.48	0.097	0.083	0.082

Core #: 39

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Project: N/A

Compaction Date: 5-11-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3212.9 g

B = 3223.9 g

C = 1823.8 g

Gmb = A/(B-C) = 2.29

Density = Gmb * 62.4pcf = 143.2 pcf

Testing Date: 6-14-94

Testing Temperature: 115 F

Height: 3 1/8 in

Comments:

st. dev. = 0.0211

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.672	0.683	0.634	0	0	0
1000	0.54	0.533	0.486	0.132	0.15	0.148
4000	0.515	0.496	0.468	0.157	0.187	0.166
8000	0.495	0.469	0.456	0.177	0.214	0.178

Core #: 40

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 5-11-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3219.4 g

B = 3253.4 g

C = 1824.8 g

Gmb = A/(B-C) = 2.25

Density = Gmb * 62.4pcf = 140.6 pcf

Testing Date: 6-15-94

Testing Temperature: 115 F

Height: 3 3/16 in

Comments:

st. dev. = 0.0163

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.604	0.632	0.626	0	0	0
1000	0.544	0.566	0.571	0.06	0.066	0.055
4000	0.506	0.536	0.556	0.098	0.096	0.07
8000	0.502	0.519	0.545	0.102	0.113	0.081

Core #: 41

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 5-11-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3220.4 g

B = 3257.4 g

C = 1819.8 g

Gmb = A/(B-C) = 2.24

Density = Gmb * 62.4pcf = 139.8 pcf

Testing Date: 6-16-94

Testing Temperature: 115 F

Height: 3 3/16 in

Comments:

st. dev.= 0.0155

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.652	0.674	0.613	0	0	0
1000	0.574	0.579	0.525	0.078	0.095	0.088
4000	0.534	0.534	0.491	0.118	0.14	0.122
8000	0.494	0.5	0.47	0.158	0.174	0.143

Core #: 50

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 6-27-94

Compaction Procedure:

100 @ 350 psi

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3201.9 g

B = 3225.4 g

C = 1825.8 g

Gmb = A/(B-C) = 2.29

Density = Gmb * 62.4pcf = 142.8 pcf

Testing Date: 7-5-94

Testing Temperature: 115 F

Height: 3 1/8 in

Comments:

st. dev. = 0.0216

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.53	0.562	0.537	0	0	0
1000	0.448	0.457	0.43	0.082	0.105	0.107
4000	0.405	0.407	0.4	0.125	0.155	0.137
8000	0.382	0.371	0.37	0.148	0.191	0.167

Core #: 51

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 6-27-94

Compaction Procedure:
100 @ 350 psi (kneading)
10,000 lb static levelling load (1 min)

Gmb Calculations
A = 3221.4 g
B = 3245.4 g
C = 1831.8 g
Gmb = A/(B-C) = 2.28

Density = Gmb * 62.4pcf = 142.2 pcf

Testing Date: 7-6-94
Testing Temperature: 115 F
Height: 3 1/8 in

Comments:
st. dev. = 0.0525

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.586	0.623	0.569	0	0	0
1000	0.495	0.53	0.489	0.091	0.093	0.08
4000	0.458	0.481	0.471	0.128	0.142	0.098
8000	0.434	0.406	0.456	0.152	0.217	0.113

Core #: 52

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 6-28-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3203.4 g

B = 3214.4 g

C = 1832.8 g

Gmb = A/(B-C) = 2.32

Density = Gmb * 62.4pcf = 144.7 pcf

Testing Date: 7-7-94

Testing Temperature: 115 F

Height: 3 1/16 in

Comments:

st. dev. = 0.0353

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.521	0.526	0.513	0	0	0
1000	0.414	0.425	0.444	0.107	0.101	0.069
4000	0.368	0.384	0.411	0.153	0.142	0.102
8000	0.334	0.343	0.389	0.187	0.183	0.124

Core #: 53

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 6-28-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3208.4 g

B = 3230.4 g

C = 1825.8 g

Gmb = A/(B-C) = 2.28

Density = Gmb * 62.4pcf = 142.5 pcf

Testing Date: 7-8-94

Testing Temperature: 115 F

Height: 3 1/8 in

Comments:

st. dev. = 0.0078

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.58	0.603	0.573	0	0	0
1000	0.462	0.507	0.464	0.118	0.096	0.109
4000	0.418	0.465	0.418	0.162	0.138	0.155
8000	0.403	0.437	0.392	0.177	0.166	0.181

Core #: 54

Project: N/A

Sieve	% Retained	Weight (g)
3/4"	0	0
5/8"	1	30.6
3/8"	31	948.4
# 4	27	826
# 8	16	489.5
# 30	13	397.7
#200	8	244.8
Pan	4	122.4
Lime		30.6
AC - 20		154.1
Total		3244.1

Compaction Date: 6-28-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3217.4 g

B = 3234.9 g

C = 1834.8 g

Gmb = A/(B-C) = 2.30

Density = Gmb * 62.4pcf = 143.4 pcf

Testing Date: 7-10-94

Testing Temperature: 115 F

Height: 3 1/8 in

Comments:

st. dev.=0.0210

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.563	0.585	0.565	0	0	0
1000	0.475	0.503	0.481	0.088	0.082	0.084
4000	0.421	0.47	0.449	0.142	0.115	0.116
8000	0.387	0.44	0.429	0.176	0.145	0.136

APPENDIX A2

Field Cores Data Sheets

Core #: 23

Project: P-25-04

Compaction Date: N/A

Compaction Procedure: N/A

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Gmb Calculations

A = 3376.0 g

B = 3381.0 g

C = 1933.8 g

Gmb = A/(B-C) = 2.33

Density = Gmb * 62.4pcf = 145.6 pcf

Testing Date: 5-10-94

Testing Temperature: 115 F

Height: 3 1/4 in

Comments: No Chip Seal.

st. dev. = 0.0457

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.673	0.677	0.637	0	0	0
1000	0.605	0.583	0.546	0.068	0.094	0.091
4000	0.563	0.511	0.502	0.11	0.166	0.135
8000	0.513	0.441	0.483	0.16	0.236	0.154

Core #: 24

Project: P-34-10

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 2959.9 g

B = 2963.4 g

C = 1692.8 g

Gmb = A/(B-C) = 2.33

Density = Gmb * 62.4pcf = 145.4 pcf

Testing Date: 4-19-94

Testing Temperature: 115 F

Height: 2 3/4 in

Comments: Rough Chip Seal

st. dev. = 0.0199

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.223	0.262	0.257	0	0	0
1000	0.163	0.173	0.163	0.06	0.089	0.094
4000	0.11	0.114	0.123	0.113	0.148	0.134
8000	0.064	0.065	0.089	0.159	0.197	0.168

Core #: 25

Project: P-23-03

Sieve	% Retained	Weight
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3098.4 g

B = 3102.4 g

C = 1733.8 g

Gmb = A/(B-C) = 2.26

Density = Gmb * 62.4pcf = 141.3 pcf

Testing Date: 4-25-94

Testing Temperature: 115 F

Height: 3 .0 in

Comments: PMWC

st. dev. = 0.0342

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.451	0.546	0.527	0	0	0
1000	0.364	0.401	0.406	0.087	0.145	0.121
4000	0.329	0.368	0.381	0.122	0.178	0.146
8000	0.303	0.331	0.357	0.148	0.215	0.17

Core #: 26

Project: P-12-27

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3334.0 g

B = 3337.5 g

C = 1881.8 g

Gmb = A/(B-C) = 2.29

Density = Gmb * 62.4pcf = 142.9 pcf

Testing Date: 4-21-94

Testing Temperature: 115 F

Height: 3 1/4 in

Comments: Medium Rough Chip Seal

st. dev. = 0.0111

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.679	0.632	0.573	0	0	0
1000	0.651	0.59	0.542	0.028	0.042	0.031
4000	0.632	0.569	0.529	0.047	0.063	0.044
8000	0.609	0.553	0.516	0.07	0.079	0.057

Core #: 27

Project: P-34-09

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3393.5 g

B = 3397.5 g

C = 1937.8 g

Gmb = A/(B-C) = 2.32

Density = Gmb * 62.4pcf = 145.1pcf

Testing Date: 4-21-94

Testing Temperature: 115 F

Height: 3 3/16 in

Comments: No Chip Seal

st. dev. = 0.0061

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.642	0.601	0.544	0	0	0
1000	0.594	0.564	0.492	0.048	0.037	0.052
4000	0.566	0.536	0.479	0.076	0.065	0.065
8000	0.549	0.519	0.461	0.093	0.082	0.083

Core #: 28

Project: P-20-17

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3117.9 g

B = 3121.4 g

C = 1782.3 g

Gmb = A/(B-C) = 2.33

Density = Gmb * 62.4pcf = 145.3 pcf

Testing Date: 4-24-94

Testing Temperature: 115 F

Height: 2 7/8 in

Comments: No Chip Seal

st. dev. = 0.0441

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.396	0.414	0.399	0	0	0
1000	0.297	0.274	0.293	0.099	0.14	0.106
4000	0.261	0.227	0.205	0.135	0.187	0.194
8000	0.237	0.17	0.177	0.159	0.244	0.222

Core #: 29

Project: P-12-26

Compaction Date: N/A

Compaction Procedure: N/A

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Gmb Calculations

A = 3056.9 g

B = 3064.4 g

C = 1732.3 g

Gmb = A/(B-C) = 2.29

Density = Gmb * 62.4pcf = 143.2 pcf

Testing Date: 5-2-94

Testing Temperature: 115 F

Height: 2 7/8 in

Comments: Medium Rough Chip Seal

st. dev. = 0.0192

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.447	0.486	0.451	0	0	0
1000	0.339	0.402	0.385	0.108	0.084	0.066
4000	0.308	0.374	0.354	0.139	0.112	0.097
8000	0.297	0.36	0.339	0.15	0.126	0.112

Core #: 30

Project: P-20-15

Compaction Date: N/A

Compaction Procedure: N/A

Sieve	% Retained	Weight
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Gmb Calculations

A = 2973.4 g

B = 2978.4 g

C = 1707.3 g

Gmb = A/(B-C) = 2.34

Density = Gmb * 62.4pcf = 146.0 pcf

Testing Date: 5-3-94

Testing Temperature: 115 F

Height: 2 3/4 in

Comments: PMWC

st. dev. = 0.0289

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.291	0.277	0.226	0	0	0
1000	0.207	0.164	0.123	0.084	0.113	0.103
4000	0.156	0.102	0.085	0.135	0.175	0.141
8000	0.124	0.062	0.063	0.167	0.215	0.163

Core #: 31

Project: P-40-13

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3239.0 g

B = 3243.0 g

C = 1857.8 g

Gmb = A/(B-C) = 2.34

Density = Gmb * 62.4pcf = 145.9 pcf

Testing Date:

Testing Temperature: 115 F

Height: 3.0 in

Comments: No Chip Seal, Apparent Bleeding.

st. dev. = 0.0615

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.469	0.504	0.511	0	0	0
1000	0.37	0.349	0.377	0.099	0.155	0.134
4000	0.338	0.278	0.328	0.131	0.226	0.183
8000	0.323	0.235	0.304	0.146	0.269	0.207

Core #: 32

Project: P-30-18

Compaction Date: N/A

Compaction Procedure: N/A

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Gmb Calculations

A = 3382.0 g

B = 3388.5 g

C = 1915.8 g

Gmb = A/(B-C) = 2.30

Density = Gmb * 62.4pcf = 143.3 pcf

Testing Date:

Testing Temperature: 115 F

Height: 3 1/4 in

Comments: Rough Chip Seal.

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.763	0.807	0.852	0	0	0
1000	0.651	0.59	0.676	0.112	0.217	0.176
4000	0.597	0.509	0.652	0.166	0.298	0.2
4100	0.594	0.506	0.647	0.169	0.301	0.205
8000						

Core #: 33

Project: P-12-21

Sieve	% Retained	Weight(g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3156.9 g

B = 3160.4 g

C = 1774.8 g

Gmb = A/(B-C) = 2.28

Density = Gmb * 62.4pcf = 142.2 pcf

Testing Date: 5-10-94

Testing Temperature: 115 F

Height: 3.0 in

Comments: Light Chip Seal.

st. dev. = 0.0331

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.497	0.501	0.504	0	0	0
1000	0.421	0.442	0.473	0.076	0.059	0.031
4000	0.398	0.435	0.464	0.099	0.066	0.04
8000	0.382	0.429	0.454	0.115	0.072	0.05

Core #: 35

Project: P-12-20

Sieve	% Retained	Weight (g)
3/4"	N/A	N/A
5/8"	N/A	N/A
3/8"	N/A	N/A
# 4	N/A	N/A
# 8	N/A	N/A
# 30	N/A	N/A
#200	N/A	N/A
Pan	N/A	N/A
Lime	N/A	N/A
AC - 20	N/A	N/A
Total	N/A	N/A

Compaction Date: N/A

Compaction Procedure: N/A

Gmb Calculations

A = 3609.5 g

B = 3617.0 g

C = 2027.6 g

Gmb = A/(B-C) = 2.27

Density = Gmb * 62.4pcf = 141.7 pcf

Testing Date:

Testing Temperature: 115 F

Height: 3.5 in

Comments: Medium Chip Seal.

st. dev. = 0.0326

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.987	0.984	0.969	0	0	0
1000	0.901	0.894	0.907	0.086	0.09	0.062
4000	0.868	0.862	0.892	0.119	0.122	0.077
8000	0.852	0.848	0.89	0.135	0.136	0.079

APPENDIX A3

Primary Highway Project Data Sheets

Core #: 59

Project: STPP-013-2(57)

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	77	
3/8"	65	
# 4	48	
# 8	36	
# 30	17	
#200	3	
Pan	0	
Lime		
AC - 20		
Total		3219

Compaction Date: 8-6-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3170.4 g

B = 3174.4 g

C = 1847.3 g

Gmb = A/(B-C) = 2.39

Density = Gmb * 62.4pcf = 149.1 pcf

Testing Date: 8-28-94

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Primary Highway Project

st. dev. = 0.0035

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.344	0.346	0.314	0	0	0
1000	0.305	0.302	0.278	0.039	0.044	0.036
4000	0.287	0.285	0.26	0.057	0.061	0.054
8000	0.281	0.276	0.248	0.063	0.07	0.066

Core #: 60

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	77	
3/8"	65	
# 4	48	
# 8	36	
# 30	17	
#200	3	
Pan	0	
Lime		
AC - 20		
Total		3219

Project: STPP-013-2(57)

Compaction Date: 8-7-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3183.4 g

B = 3188.9 g

C = 1855.8 g

Gmb = A/(B-C) = 2.39

Density = Gmb * 62.4pcf = 149.0 pcf

Testing Date: 8-28-94

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Primary Highway Project

st. dev. = 0.0194

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.354	0.354	0.348	0	0	0
1000	0.309	0.298	0.281	0.045	0.056	0.067
4000	0.288	0.264	0.254	0.066	0.09	0.094
8000	0.278	0.246	0.237	0.076	0.108	0.111

Core #: 61

Project: STPP-013-2(57)

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	77	
3/8"	65	
# 4	48	
# 8	36	
# 30	17	
#200	3	
Pan	0	
Lime		
AC - 20		
Total		3219

Compaction Date: 8-8-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3184.4 g

B = 3191.4 g

C = 1851.8 g

Gmb = A/(B-C) = 2.38

Density = Gmb * 62.4pcf = 148.3 pcf

Testing Date: 8-28-94

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Primary Highway Project

st. dev. = 0.0015

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.394	0.394	0.361	0	0	0
1000	0.329	0.318	0.282	0.065	0.076	0.079
4000	0.279	0.276	0.245	0.115	0.118	0.116
8000	0.262	0.259	0.228	0.132	0.135	0.133

Core #: 62

Project: STPP-013-2(57)

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	77	
3/8"	65	
# 4	48	
# 8	36	
# 30	17	
#200	3	
Pan	0	
Lime		
AC - 20		
Total		3219

Compaction Date: 8-9-94

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3185.4 g

B = 3195.4 g

C = 1852.3 g

Gmb = A/(B-C) = 2.37

Density = Gmb * 62.4pcf = 148.0 pcf

Testing Date: 8-29-94

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Primary Highway Project

st. dev.= 0.0030

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.388	0.382	0.346	0	0	0
1000	0.369	0.373	0.334	0.019	0.009	0.012
4000	0.363	0.368	0.323	0.025	0.014	0.023
8000	0.362	0.362	0.323	0.026	0.02	0.023

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SOMAT Data Sheets

Core #: 66

Project: SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-11-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3205.4 g

B = 3210.4 g

C = 1917.8 g

Gmb = A/(B-C) = 2.48

Density = Gmb * 62.4pcf = 154.7 pcf

Testing Date: 1-13-95

Testing Temperature: 115 F

Height: 2 13/16 in

Comments:

st. dev. = 0.0044

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.329	0.323	0.298	0	0	0
1000	0.218	0.201	0.189	0.111	0.122	0.109
4000	0.159	0.156	0.13	0.17	0.167	0.168
8000	0.129	0.131	0.105	0.2	0.192	0.193

Core #: 67

Project: SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-11-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3205.4 g

B = 3210.4 g

C = 1913.8 g

Gmb = A/(B-C) = 2.47

Density = Gmb * 62.4pcf = 154.3 pcf

Testing Date: 1-16-95

Testing Temperature: 115 F

Height: 2 7/8 in

Comments:

st. dev. = 0.0247

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.407	0.384	0.329	0	0	0
1000	0.284	0.233	0.189	0.123	0.151	0.14
4000	0.259	0.208	0.145	0.148	0.176	0.184
8000	0.231	0.163	0.113	0.176	0.221	0.216

Core #: 68

Project: SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-11-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3210.4 g

B = 3213.4 g

C = 1915.8 g

Gmb = A/(B-C) = 2.47

Density = Gmb * 62.4pcf = 154.4 pcf

Testing Date: 1-17-95

Testing Temperature: 115 F

Height: 2 7/8 in

Comments:

st. dev. = 0.0331

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.254	0.252	0.221	0	0	0
1000	0.198	0.163	0.135	0.056	0.089	0.086
4000	0.17	0.125	0.077	0.084	0.127	0.144
8000	0.15	0.097	0.055	0.104	0.155	0.166

Core #: 69

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Project: SOMAT RCPMP

Compaction Date: 1-12-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3210.9 g

B = 3212.4 g

C = 1914.3 g

Gmb = A/(B-C) = 2.47

Density = Gmb * 62.4pcf = 154.3 pcf

Testing Date: 1-18-95

Testing Temperature: 115 F

Height: 2 13/16 in

Comments:

st. dev.= 0.0108

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.249	0.242	0.214	0	0	0
1000	0.187	0.157	0.142	0.062	0.085	0.072
4000	0.159	0.138	0.105	0.09	0.104	0.109
8000	0.134	0.107	0.082	0.115	0.135	0.132

Core #: 70

Project: Non-SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-12-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3209.4 g

B = 3212.4 g

C = 1915.8 g

Gmb = A/(B-C) = 2.48

Density = Gmb * 62.4pcf = 154.5 pcf

Testing Date: 1-19-95

Testing Temperature: 115 F

Height: 2 13/16 in

Comments:

st. dev.= 0.0185

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.26	0.24	0.202	0	0	0
1000	0.194	0.153	0.117	0.066	0.087	0.085
4000	0.153	0.105	0.067	0.107	0.135	0.135
8000	0.128	0.075	0.039	0.132	0.165	0.163

Core #: 71

Project: Non-SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-12-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3211.9 g

B = 3214.4 g

C = 1915.3 g

Gmb = A/(B-C) = 2.47

Density = Gmb * 62.4pcf = 154.3 pcf

Testing Date: 1-30-95

Testing Temperature: 115 F

Height: 2 13/16 in

Comments:

st. dev. = 0.0444

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.263	0.252	0.238	0	0	0
1000	0.192	0.131	0.105	0.071	0.121	0.133
4000	0.155	0.094	0.054	0.108	0.158	0.184
8000	0.137	0.072	0.024	0.126	0.18	0.214

Core #: 72

Project: Non-SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-13-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3209.4 g

B = 3212.4 g

C = 1910.3 g

Gmb = A/(B-C) = 2.46

Density = Gmb * 62.4pcf = 153.8 pcf

Testing Date: 1-31-95

Testing Temperature: 115 F

Height: 2 7/8 in

Comments:

st. dev. = 0.0316

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.336	0.332	0.284	0	0	0
1000	0.209	0.179	0.139	0.127	0.153	0.145
4000	0.181	0.128	0.096	0.155	0.204	0.188
8000	0.169	0.103	0.075	0.167	0.229	0.209

Core #: 73

Project: Non-SOMAT RCPMP

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-13-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3214.4 g

B = 3216.9 g

C = 1912.3 g

Gmb = A/(B-C) = 2.46

Density = Gmb * 62.4pcf = 153.7 pcf

Testing Date: 1-31-95

Testing Temperature: 115 F

Height: 2 7/8 in

Comments:

st. dev. = 0.0232

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.33	0.318	0.317	0	0	0
1000	0.207	0.193	0.2	0.123	0.125	0.117
4000	0.164	0.139	0.168	0.166	0.179	0.149
8000	0.123	0.107	0.148	0.207	0.211	0.169

APPENDIX A5

SMA Data Sheets

Core #: 82

Project: SMA Test Strip

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3089.4 g

B = 3106.4 g

C = 1774.3 g

Gmb = A/(B-C) = 2.32

Density = Gmb * 62.4pcf = 144.7 pcf

Testing Date: 3-11-95

Testing Temperature: 115 F

Height: 3 in

Comments: Test Section 1

st. dev. = 0.0458

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.387	0.371	0.372	0	0	0
1000	0.33	0.31	0.309	0.057	0.061	0.063
4000	0.317	0.297	0.295	0.07	0.074	0.077
8000	0.307	0.285	0.283	0.08	0.086	0.089

Core #: 92

Project: SMA Test Strip

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3059.4 g

B = 3071.9 g

C = 1762.3 g

Gmb = A/(B-C) = 2.34

Density = Gmb * 62.4pcf = 145.8 pcf

Testing Date: 4-6-95

Testing Temperature: 115 F

Height: 2 7/8 in

Comments: Test Section 1

st. dev. = 0.0127

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.36	0.314	0.226	0	0	0
1000	0.307	0.267	0.161	0.053	0.047	0.065
4000	0.295	0.252	0.134	0.065	0.062	0.092
8000	0.285	0.244	0.132	0.075	0.07	0.094

Core #: 83

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Project: SMA Test Strip

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3139.9 g

B = 3148.4 g

C = 1797.3 g

Gmb = A/(B-C) = 2.32

Density = Gmb * 62.4pcf = 145.0 pcf

Testing Date: 3-11-95

Testing Temperature: 115 F

Height: 3 in

Comments: Test Section 3

st. dev. = 0.0269

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.389	0.376	0.374	0	0	0
1000	0.334	0.285	0.301	0.055	0.091	0.073
4000	0.325	0.273	0.286	0.064	0.103	0.088
8000	0.317	0.251	0.268	0.072	0.125	0.106

Core #: 90

Project: SMA Test Strip

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3168.9 g

B = 3173.4 g

C = 1847.3 g

Gmb = A/(B-C) = 2.39

Density = Gmb * 62.4pcf = 149.1 pcf

Testing Date: 4-4-95

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Test Section 3

st. dev. = 0.0327

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.301	0.336	0.293	0	0	0
1000	0.201	0.211	0.205	0.1	0.125	0.088
4000	0.19	0.17	0.191	0.111	0.166	0.102
8000	0.173	0.161	0.181	0.128	0.175	0.112

Core #: 88

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Project: SMA Test Strip

Compaction Date: N/A

Compaction Procedure:
Field Compacted

Gmb Calculations

A = 3240.4 g

B = 3244.9 g

C = 1893.3 g

Gmb = A/(B-C) = 2.40

Density = Gmb * 62.4pcf = 149.6 pcf

Testing Date: 4-4-95

Testing Temperature: 115 F

Height: 3 in

Comments: Test Section 4

st. dev. = 0.0080

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.409	0.412	0.391	0	0	0
1000	0.281	0.311	0.282	0.128	0.101	0.109
4000	0.242	0.257	0.215	0.167	0.155	0.176
8000	0.218	0.229	0.192	0.191	0.183	0.199

Core #: 94

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Project: SMA Test Strip

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3136.4 g

B = 3151.9 g

C = 1795.8 g

Gmb = A/(B-C) = 2.31

Density = Gmb * 62.4pcf = 144.3 pcf

Testing Date: 4-6-95

Testing Temperature: 115 F

Height: 3 in

Comments: Test Section 4

st. dev. = 0.0420

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.431	0.446	0.384	0	0	0
1000	0.266	0.265	0.263	0.165	0.181	0.121
4000	0.185	0.208	0.195	0.246	0.238	0.189
8000	0.138	0.135	0.153	0.293	0.311	0.231

Core #: 87

Project: SMA Test Strip

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3219.4 g

B = 3222.4 g

C = 1882.8 g

Gmb = A/(B-C) = 2.40

Density = Gmb * 62.4pcf = 150.0 pcf

Testing Date: 4-4-95

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Test Section 5

st. dev. = 0.0185

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.382	0.375	0.33	0	0	0
1000	0.276	0.242	0.208	0.106	0.133	0.122
4000	0.252	0.215	0.19	0.13	0.16	0.14
8000	0.247	0.203	0.176	0.135	0.172	0.154

Core #: 89

Project: SMA Test Strip

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3193.4 g

B = 3197.4 g

C = 1857.8 g

Gmb = A/(B-C) = 2.38

Density = Gmb * 62.4pcf = 148.8 pcf

Testing Date: 4-4-95

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Test Section 5

st. dev. = 0.0356

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.323	0.345	0.323	0	0	0
1000	0.24	0.22	0.175	0.083	0.125	0.148
4000	0.207	0.201	0.146	0.116	0.144	0.177
8000	0.201	0.193	0.13	0.122	0.152	0.193

Core #: 91

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Project: SMA Test Strip

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3184.4 g

B = 3189.4 g

C = 1853.8 g

Gmb = A/(B-C) = 2.38

Density = Gmb * 62.4pcf = 148.8 pcf

Testing Date: 4-6-95

Testing Temperature: 115 F

Height: 2 13/16 in

Comments: Test Section 6

st. dev. = 0.0164

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.39	0.362	0.312	0	0	0
1000	0.276	0.277	0.223	0.114	0.085	0.089
4000	0.255	0.264	0.212	0.135	0.098	0.1
8000	0.255	0.252	0.208	0.135	0.11	0.104

Core #: 93

Project: SMA Test Strip

Sieve	% Passing	Weight (g)
3/4"	100	
1/2"	81	
3/8"	63	
# 4	35	
# 8	28	
# 30	21	
#200	17	
Pan	0	
Lime		
AC - 20		
Total		

Compaction Date: N/A

Compaction Procedure:

Field Compacted

Gmb Calculations

A = 3052.9 g

B = 3058.4 g

C = 1750.8 g

Gmb = A/(B-C) = 2.33

Density = Gmb * 62.4pcf = 145.7 pcf

Testing Date: 4-6-95

Testing Temperature: 115 F

Height: 2 15/16 in

Comments: Test Section 6

st. dev. = 0.0165

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.384	0.268	0.222	0	0	0
1000	0.296	0.164	0.116	0.088	0.104	0.106
4000	0.278	0.144	0.087	0.106	0.124	0.135
8000	0.262	0.123	0.068	0.122	0.145	0.154

Core #: 74

Project: SMA Test Strip

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 1-31-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3200.4 g

B = 3206.4 g

C = 1855.3 g

Gmb = A/(B-C) = 2.37

Density = Gmb * 62.4pcf = 147.8 pcf

Testing Date: 2-6-95

Testing Temperature: 115 F

Height: 3 in

Comments:

st. dev. = 0.0358

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.45	0.42	0.358	0	0	0
1000	0.384	0.345	0.299	0.066	0.075	0.059
4000	0.375	0.336	0.293	0.075	0.084	0.065
8000	0.369	0.332	0.29	0.081	0.088	0.068

Core #: 75

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Project: SMA Test Strip

Compaction Date: 1-31-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3190.4 g

B = 3200.4 g

C = 1851.3 g

Gmb = A/(B-C) = 2.36

Density = Gmb * 62.4pcf = 147.6 pcf

Testing Date: 2-6-95

Testing Temperature: 115 F

Height: 3 in

Comments: Test Section 3

st. dev. = 0.0023

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.477	0.494	0.466	0	0	0
1000	0.414	0.434	0.403	0.063	0.06	0.063
4000	0.405	0.424	0.394	0.072	0.07	0.072
8000	0.396	0.417	0.389	0.081	0.077	0.077

Core #: 76

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Project: SMA Test Strip

Compaction Date: 1-31-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3190.4 g

B = 3197.4 g

C = 1848.3 g

Gmb = A/(B-C) = 2.36

Density = Gmb * 62.4pcf = 147.6 pcf

Testing Date: 2-6-95

Testing Temperature: 115 F

Height: 3 in

Comments: Test Section 3

st. dev. = 0.0165

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.484	0.503	0.467	0	0	0
1000	0.398	0.412	0.391	0.086	0.091	0.076
4000	0.389	0.391	0.385	0.095	0.112	0.082
8000	0.379	0.381	0.378	0.105	0.122	0.089

Core #: 84

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total ,		3219

Project: SMA Test Strip

Compaction Date: 2-7-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3201.4 g

B = 3235.4 g

C = 1845.8 g

Gmb = A/(B-C) = 2.30

Density = Gmb * 62.4pcf = 143.8 pcf

Testing Date: 3-12-95

Testing Temperature: 115 F

Height: 3 1/8 in

Comments: Control Test Section

st. dev. = 0.0187

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.49	0.52	0.498	0	0	0
1000	0.412	0.426	0.402	0.078	0.094	0.096
4000	0.407	0.408	0.381	0.083	0.112	0.117
8000	0.401	0.402	0.374	0.089	0.118	0.124

Core #: 85

Project: SMA Test Strip

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 3-10-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3195.4 g

B = 3223.4 g

C = 1840.8 g

Gmb = A/(B-C) = 2.31

Density = Gmb * 62.4pcf = 144.2 pcf

Testing Date: 3-12-95

Testing Temperature: 115 F

Height: 3 1/16 in

Comments: Control Test Section

st. dev. = 0.0277

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.47	0.461	0.471	0	0	0
1000	0.424	0.445	0.427	0.046	0.016	0.044
4000	0.403	0.437	0.405	0.067	0.024	0.066
8000	0.398	0.436	0.397	0.072	0.025	0.074

Core #: 86

Project: SMA Test Strip

Sieve	% Retained	Weight (g)
3/4"		
5/8"		
3/8"		
# 4		
# 8		
# 30		
#200		
Pan		
Lime		
AC - 20		
Total		3219

Compaction Date: 3-10-95

Compaction Procedure:

100 @ 350 psi (kneading)

10,000 lb static levelling load (1 min)

Gmb Calculations

A = 3189.9 g

B = 3218.9 g

C = 1839.8 g

Gmb = A/(B-C) = 2.31

Density = Gmb * 62.4pcf = 144.3 pcf

Testing Date: 3-12-95

Testing Temperature: 115 F

Height: 3 1/16 in

Comments: Control Test Section

st. dev. = 0.0205

Cycles	Dial Indicator Reading (in)			Rut Depths (in)		
	LOC	Center	ROC	LOC	Center	ROC
0	0.485	0.519	0.513	0	0	0
1000	0.421	0.48	0.444	0.064	0.039	0.069
4000	0.408	0.467	0.424	0.077	0.052	0.089
8000	0.407	0.463	0.416	0.078	0.056	0.097

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13. ABSTRACT (Maximum 200 words)

In this research, the feasibility of using the Georgia Loaded-Wheel Tester (GLWT) to predict rutting in the laboratory was investigated. This research was performed in two phases. The first phase consisted of modifying the GLWT to handle 15.2 cm (6 in) cores, developing a laboratory compaction procedure for cores, determining the optimum laboratory testing conditions, and investigating the repeatability of the GLWT. The second phase of this research project included correlating rut depth values obtained with the GLWT to actual field rut depth values, utilizing the GLWT to evaluate the effects of the asphalt additive SOMAT on asphalt concrete mixes, and evaluating the rut resistance of Stone Matrix Asphalt (SMA).

Results from this study show that the GLWT is capable of predicting rutting in asphalt pavements prior to construction. In addition, results from the GLWT correlate well with results from more expensive European Testers.

14. SUBJECT TERMS
asphalt, pavement, GLWT, Georgia Loaded-Wheet Tester, SOMAT, rutting, Stone Matrix Asphalt

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