PREDICTING FIELD PERFORMANCE ON THE

NCAT PAVEMENT TEST TRACK

Except where reference is made to the work of others, the work described in this dissertation is my own or was done in collaboration with my advisory committee. This dissertation does not include proprietary or classified information.

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PREDICTING FIELD PERFORMANCE ON THE NCAT PAVEMENT TEST TRACK

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PREDICTING FIELD PERFORMANCE ON THE

NCAT PAVEMENT TEST TRACK

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VITA

Raymond "Buzz" Powell, son of Bill Powell and Frances Bradshaw, was born on October 10, 1964 in Montgomery, Alabama. While employed with the Alabama Department of Transportation (ALDOT) in the Pavement Management Division he concurrently attended college, graduating with the degree of Bachelor of Science in Civil Engineering, with High Honors, from Auburn University in March, 1990. After completing his professional internship with ALDOT in site characterization and geotechnical construction, he obtained his license to practice professional engineering in July of 1994. Moving into the field of pavement research, ALDOT granted him the privilege of continuing his education. Subsequently, in December of 1996 he graduated with the degree of Master of Science in Civil Engineering from Auburn University. Following a twelve-year career with ALDOT, he worked for two years as the principal engineer for a consulting firm in Montgomery, Alabama. He has worked since September of 1999 for the National Center for Asphalt Technology (NCAT), serving as Manager of the NCAT Pavement Test Track while pursuing a Doctor of Philosophy Degree in Civil Engineering from Auburn University. Buzz married Kathy Crockett, daughter of Charles and Judy Crockett on January 24, 1986. They are the blessed parents of three wonderful children; Brittany Michelle, Brad Allen and India Blair.

DISSERTATION ABSTRACT

PREDICTING FIELD PERFORMANCE ON THE

NCAT PAVEMENT TEST TRACK

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An experimental facility has been constructed near the campus of Auburn University for the purpose of conducting research to extend the life of flexible pavements. Experimental sections on the 2.8 kilometer Pavement Test Track are cooperatively funded by external sponsors, most commonly state departments of transportation (DOTs), with operation and research managed by the National Center for Asphalt Technology (NCAT). Forty-six different flexible pavements were initially installed at the facility in 2000, each at a length of 61 meters. Materials and methods unique to section sponsors were imported during construction to maximize the applicability of results for the individual sponsors. A design lifetime of truck traffic was

applied in an accelerated manner over a two-year period of time, with field performance documented weekly.

Sponsors typically planned to compare the performance of two or more sections constructed with different materials and/or methods to obtain information that could be used to build pavements with a better life cycle to cost ratio. In addition to assessing alternatives for sponsors, NCAT was responsible for guiding the overall effort in a direction that would address policy issues for the highway industry as a whole.

Specifically, laboratory methods that have the potential to predict rutting when used during construction are compared herein to a detailed record of field performance as a function of traffic over a range of temperatures for every experimental mix. This new method of characterizing traffic is referred to as "load-temperature spectra." Regression methods are used to generate a distinct prediction model for each laboratory test based on weekly changes in rutting, and statistical methods are utilized to evaluate the success of each method. The suitability of testing during gyratory compaction, simulative testing, and fundamental testing in assessing the rutting performance of research mixes is evaluated with recommendations made for future practice.

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CHAPTER 1 – INTRODUCTION

1.1 BACKGROUND

An experimental facility has been constructed near the campus of Auburn University that is being used by governmental agencies throughout the United States to conduct research designed to extend the life of flexible pavements. Managed by the National Center for Asphalt Technology (NCAT), the Pavement Test Track provides an opportunity for sponsors to answer specific questions related to flexible pavement performance in a full scale, accelerated manner where results do not require laboratory scale extrapolations or lifelong field observations.

Experimental sections on the 2.8 kilometer Pavement Test Track are cooperatively funded by external sponsors, most commonly state departments of transportation (DOTs), with subsequent operation and research managed by NCAT. Forty-six different flexible pavements were originally installed at the facility, each at a length of 61 meters. A uniform perpetual foundation ensured that differences in surface performance were the result of the influence of the top 100 mm of pavement structure, which was different for each section. Materials and mix designs unique to section sponsors were imported during construction to maximize the applicability of results. A design lifetime of truck traffic (10 million equivalent single axle loadings, or ESALs) was applied in an accelerated manner over a two-year period of time, with subsequent pavement performance documented weekly.

Unlike conventional efforts on public roadways, research at the NCAT Pavement Test Track is conducted in a closed-loop facility where axle loadings are precisely monitored and environmental effects are the same for every mix. An array of surface parameters (smoothness, rutting, cracking, etc.) is monitored weekly as truck traffic accumulates to facilitate objective performance analyses. State DOTs typically have to wait ten to fifteen years to obtain less reliable results (at risk of injury to testing personnel) in full-scale field studies on public roadways.

Sponsors typically fund track research on two or more sections so they can compare life cycle costs of common paving alternatives. In this manner, they can rationally manage the public's investment in flexible pavements by choosing mixes that cost less over the life of the structure. For example, it is unwise to spend less on construction if the cheaper construction alternative results in a substantially higher life cycle cost.

It is difficult to conduct reliable field performance comparisons on open roadways for several reasons. Since user costs and reconstruction expenditures are usually very high when test pavements fail, there can actually be a disincentive to experiment with new methods and materials. When field comparisons are attempted, the quality of construction sometimes varies greatly from region to region. As a consequence of the longer testing period, it is not uncommon for post-construction personnel reassignments to make it difficult to maintain continuity of effort. After construction, there is typically much uncertainty in the number and weight of axle loadings. Lastly, environmental effects can vary significantly from region to region within a single state's jurisdiction.

The Pavement Test Track (referred to hereafter as the Track) is designed such that experimental sections are not subject to these same limitations. Rehabilitation of potential failures is built into the cost of sponsorship. Construction and testing of all sections is the responsibility of NCAT, with oversight provided by section sponsors. The number and weight of axle loadings is monitored with accuracy, and all sections must perform in an identical climate. The facility was even constructed so that the tangents lie perpendicular to due north to minimize the effect of shadows that would otherwise be cast from surrounding trees.

1.2 OBJECTIVE

The objectives of the work reported herein were to document the accelerated performance testing program at the 2000 NCAT Pavement Test Track, then develop models to predict observed differences in rutting performance using laboratory test results.

1.3 SCOPE

Stockpile materials from eight different states were imported to Opelika, Alabama in support of experimental sections encompassing traditional mixes (e.g., Hveem), SUPERPAVE mixes, stone matrix asphalt (SMA) mixes, and open-graded friction course (OGFC) mixes using natural, processed and synthetic aggregates. Four different grades of asphalt binder were used in the production of fifty-four unique mixes placed in various combinations of binder and surface lifts in the top 100 mm of pavement structure.

In order to apply the traffic loading in an accelerated manner, four triple-trailer rigs of near identical (thus, limited) load spectra were utilized. Pavement temperatures were recorded at the time of each axle pass. Rutting was documented weekly using both contact and non-contact methods that were later correlated to wire line measurements made at the end of the two-year traffic cycle. Testing during gyratory compaction, simulative testing and fundamental testing was conducted on hot-mix material that was sampled from trucks en route to the Track at the time of construction. Due to the uncertainty of wheelpath location in the curves, only data from tangent sections is presented herein and utilized in the development of predictive modeling.

CHAPTER 2 - LITERATURE REVIEW

2.1 GENERAL OVERVIEW

The type of research conducted at the NCAT Pavement Test Track is known as full-scale accelerated performance testing (APT). It is classified as full-scale because actual vehicles are used to apply the traffic loadings, and is referred to as accelerated performance because the design traffic that would normally be applied over many years on an open roadway is compressed and applied in only two years.

In order to assess the need and significance of the NCAT study, literature was reviewed relating to the history of APT, options available for laboratory testing and correlating results, and methodologies available for modeling rutting performance.

2.2 ACCELERATED PERFORMANCE TESTING

APT is a type of research that has been utilized in numerous programs around the world to generate comparative results intended to reveal differences in long-term performance potential within a relatively short period of time. The term has been comprehensively defined in the following way:

"… APT is defined as the controlled application of a prototype wheel loading, at or above the appropriate legal load limit to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated accumulation of damage in a compressed time period. The acceleration of damage is achieved by means of increased repetitions, modified loading conditions, imposed climatic conditions, the use of thinner pavements with a decreased structural capacity and thus shorter design lives or a combination of these factors. Full-scale construction by conventional plant and processes is necessary so that real world conditions are modeled (Metcalf, 1996)."

Because of the broad spectrum of research conducted globally on pavement performance that will be described in detail in the following paragraphs, a more liberal definition of APT is more applicable. The following verbiage has recently been provided to comprehensively describe APT:

"… the controlled application of wheel loading to pavement structures for the purpose of simulating the effects of long-term in-service loading conditions in a compressed time period (Hugo and Martin, 2004)."

Accelerated testing is useful for many reasons. If all performance testing was conducted on open roadways, it would be impossible to fully consider the effect of increasing traffic demands on future performance. For example, the total tonnage that will be shipped by trucks in the United States is projected to increase 49 percent by 2020 (TRIP, 2004). This is a tremendous increase, and would mean that current open roadway performance testing at today's traffic level would only reveal how pavements performed in the past rather than in the future. Additionally, the impact of illegal axle loadings would be beyond the scope of available data. Lastly, safety considerations would make it impossible to obtain continuous performance data over time.

For this reason, there has been much national and international interest in supporting APT efforts in recent decades. In 1986, the planning document for the Strategic Highway Research Program (SHRP) listed APT as a potential study type in Long-Term Pavement Performance (LTPP) research (TRB-SHRP, 1986); however, no mention is made in subsequent LTPP documents. When asked to reconsider supplementing the LTPP database with APT data in 2002, the SHRP planning committee responded positively but cited concerns of limited resources. Current efforts focus on using APT to generate accelerated data for select Specific Pavement Studies (SPS) sections, which would enhance LTPP while at the same time provide data to relate APT to actual field performance. If these efforts are successful, APT would be used only on off-site locations due to safety concerns to the motoring public (Jones, 2003).

2.2.1 Full Scale Test Tracks

Full scale testing has made straightforward contributions to the paving industry over the last century because it does not require significant scale, time, or environmental extrapolation. Of course, the utilization of test tracks has been hindered by cost, which can be prohibitive depending on research funding mechanisms. Throughout the history of the industry, pavement professionals have invested in test tracks when significant changes in either vehicle or pavement technology has rendered the current state of practice inadequate or lacking.

2.2.1.1 History

The first asphalt pavement in the US was placed in Newark, NJ in 1870 using asphalt binder and rock asphalt imported from Europe. In 1871, a pavement was placed in Washington, DC using crushed stone and domestic coal tar (Harman et al., 2002). The first significant asphalt project in the US was placed in 1876 on Pennsylvania Avenue in Washington, DC. It covered 45,000 square meters and was produced with what was considered at the time the ideal material – Trinidad asphalt (Gillespie, 1992).

A 30,000 square meter project on the same street that was produced with North American rock asphalt provided an opportunity for the first domestic performance comparison test. A finding of equal performance set the stage for greater utilization of non-Trinidad material in the future. More projects followed, and builders' understanding of the engineering principles of asphalt construction deepened. By 1910, refined or petroleum asphalt had permanently overtaken both imported Trinidad and domestic rock asphalt (Gillespie, 1992). Hot-mix asphalt pavements became more prevalent in the United States when the developing refinery and terminal infrastructure made the distribution of liquid asphalt cost effective.

In the early 1900s, highway projects were boosted by the hope of automobile-centered prosperity through a state and national road building movement that promoted the ideal of transcontinental highways (Schauer, 2003). Traffic volumes increased and governments rushed to provide roads that were reliably passable by both automobiles and trucks. Asphalt construction that was used to pave the growing infrastructure with available materials was largely in the hands of private companies, who produced mixes in secrecy under the protection of patents. Fortunately, many of the patents that prevented growth and understanding in the industry began to expire around 1920 (Gillespie, 1992).

The first documented road test in the United States was initiated in 1918 at the Bureau of Public Road's Experimental Farm in Arlington, Virginia. The purpose of this experiment, which was built on the site that would ultimately become Reagan National Airport, was to measure impact forces of different types of wheel loads (Weingroff, 1996).

One of the earliest efforts in the United States to conduct accelerated performance testing to enhance the general understanding of the road building process was the Bates Experimental Road. In 1917, voters in Illinois overwhelmingly approved a $60 million bond issue to finance new highway construction. In order to utilize the most effective designs, the State's Bureau of Public Roads commissioned what would become know as the Bates Experimental Road. Sixty-eight test sections were built near Springfield, Illinois and trafficked in 1922 and 1923 with axle loads that varied between 11 and 58 kN. Trucks equipped with solid rubber tires ran on pavements with brick, asphalt concrete, and Portland cement concrete wearing surfaces to determine which designs lasted the longest under different loading conditions (Mahoney, 2000).

During the 1920s, the automobile and truck manufacturing industries switched from solid rubber to pneumatic tires and from single tires to dual assemblies. These advances led to greater loads and much larger vehicles. By 1932, the American Association of State Highway Officials (AASHO) recommended a legal load of 71 kN for single axles. This limit was raised in 1942 to 80 kN to accommodate larger wartime shipments. In 1946, AASHO recommended a 142 kN tandem axle limit. Between the 1920s and the 1950s, the consistent trend was for more trucks with larger loads (Mahoney, 2000).

Many road tests were conducted in the years before loads changed dramatically as a result of World War II. The Hybla Valley Test was conducted in Virginia by the Bureau of Public Roads, the Highway Research Board (HRB), and the Asphalt Institute. The Pittsburg, California Road Test was conducted using surplus army trucks with solid tires (Pasko, 1998). In a test conducted by the US Army Corps of Engineers on a track in Stockton, California it was first observed that necessary pavement thickness increased directly with the logarithm of axle load repetitions (Hveem, 1948).

The Corps of Engineers has conducted accelerated performance testing (such as the Lockbourne Air Force Base test) of various types for over fifty years (Mahoney, 2000). Their primary focus has been consistent with the mission of the Department of Defense, which is force projection and sustainment; however, their work also at times addressed the needs of the Federal Aviation Administration and the Federal Highway Administration (Lynch et al., 1999). Numerous mission-oriented Flexible Pavement Laboratory tests were conducted by the Corps at their Vicksburg, Mississippi Waterways Experiment Station (WES) in order to evaluate hot weather performance, and complementary tests were conducted at their Hanover, New Hampshire Cold Regions

Research and Engineering Laboratory (CCREL) in order to consider the effect of cold weather (Mahoney, 2000).

Among other advances, these programs produced California Bearing Ratio (CBR) design procedures. Both accelerated and full-scale tests were used to extend California Highway Department design curves to aircraft loads and environmental extremes. Accelerated full-scale pavement testing was later used to incorporate the Marshall stability method for expedient airfields. In the late 1940s, WES experiments concluded that lower loads with higher pressures were more detrimental than higher loads at lower pressures. All research conducted by the Corps prior to the 1970s was conducted in the field (Lynch et al., 1999).

As a consequence of new postwar load limits, a flurry of activity occurred after the end of World War II. The Maryland Road Test was conducted just south of Washington, DC near La Plata, Maryland in 1950 and 1951 by the Bureau of Public Roads (now the Federal Highway Administration, otherwise known as FHWA) in conjunction with the Highway Research Board (now the Transportation Research Board), several states, truck manufacturers, and other highway-related industries. In this rigid pavement experiment, an existing 1.8 km two-lane highway was carefully inventoried, instrumented, and traversed by 1,000 trucks per day (Pasko, 1998). The test sections for this study are reported to have been built in 1941, likely interrupted by US participation in World War II (Berthelot, 2004).

The WASHO Road Test was conducted in Malad, Idaho from 1952 to 1954 with HRB again directing the project. The focus of the work was flexible pavements with thicknesses of either 50 or 100 mm as part of from 150 to 550 mm of total structural depth. The total cost of this effort was reported to be $650 thousand, with two thirds of the cost provided by state DOTs. This 1950s figure equates to $84 thousand per section in 1999 dollars, encompassing forty-six sections (without replicates) on two loops with four lanes. One unique load was applied per test lane (80 or 100 kN on single axles, 142 or 178 kN on tandem axles), with no traffic allowed in the winter season and limited traffic in the spring. All totaled, 240,000 load applications were applied via 120,000 passes (Mahoney, 2000).

The WASHO Road Test was a collaboration by twelve state DOTs, vehicle manufacturers, petroleum companies, three Universities, the US Army, and the trucking industry (Mahoney, 2000). The study provided valuable experience upon which subsequent testing would be based, but the results could not be used to describe pavement performance in quantifiable terms (Berthelot, 2004).

The AASHO Road Test (shown in Figure 2.1) has had the greatest impact on the practice of pavement engineering through the landmark finding of the well-known "4^{th} power rule" that described the relative damaging effect of axle loads (National Research Council, 1962). It was conducted from 1956 to 1961 on both rigid and flexible pavements off I-80 in Ottawa, Illinois at a total cost of $27 million. Of this total, $12 million was provided by state DOTs and $7 million was provided by the BPR (the balance came from many other smaller sources). In 1999 dollars, the cost per section has been reported to be $172 thousand (Mahoney, 2000).

A total of 468 flexible sections with three surface thicknesses, three base thicknesses, and three subbase thicknesses were installed and tested at the facility, compared with 368 rigid sections, on six test loops that comprised a full factorial experiment. The length of each flexible test section was 30 meters, while the length of each rigid test section was 73 meters. The project was delayed for five years so it could be modified in consideration of the WASHO test (Mahoney, 2000), and each sponsoring state DOT sent materials for inclusion in the testing program (Berthelot, 2004).

The general practice of pavement engineering in the United States would for decades be determined by results from the full factorial regression analysis conducted on performance data from the AASHO Road Test; however, regional interest in the value of accelerated performance testing persisted. For example, the Penn State Test Track was built at the Pennsylvania Transportation Institute and began testing pavements in 1971 for the purpose of refining pavement design methodology in Pennsylvania and surrounding states (Hugo and Martin, 2004). This project is reported to have contributed significantly to pavement design in the northeastern United States (Mitchell, 1996); however, the facility has not been active in pavements research since 1983 (Hugo and Martin, 2004). Since that time, innovative vehicle research (e.g., the study of hybrid vehicle electrical components) has become a more prominent area of study (Pennsylvania Transportation Institute, 2005).

2.2.1.2 State-of-the-Art

With the rise in popularity of APT load simulators (explained in subsequent paragraphs), interest in full-scale test tracks subsided over the next two decades. As experience with load simulation devices revealed their limitations, researchers gradually realized that test tracks and simulators each provided distinct complementary information to engineers on the long-term performance potential of pavements. This movement was encouraged by the great changes that were occurring in the industry as a result of the Strategic Highway Research Program (SHRP) and SUPERPAVE implementation.

Although not an accelerated loading operation, the first facility to emerge from this renewed interest in full-scale test tracks was the Minnesota Road Research Project (Mn/Road). Constructed adjacent to I-94 northwest of Minneapolis, Mn/Road was opened to traffic in 1994. Twenty-three mainline test sections (nine concrete and fourteen asphalt) were placed into service, along with seventeen with lower traffic volumes. The total cost of construction is reported to have been $25 million. The facility is supported by natural silty clay subgrades that are susceptible to frost action, so four sections were underlain with imported sand in order to vary performance. The two primary objectives of the ten-year project were to facilitate the development, validation and implementation of mechanistic-empirical design procedures, as well as to determine when truck load restrictions should be in place in order to prevent excessive damage during spring thaws (Newcomb et al., 1999).

Also of great recent interest is the now completed Westrack project (shown in Figure 2.2). Located about 100 km southeast of Reno at the Nevada Automotive Test

Center, Westrack was a $14.5 million project in which robotic tractors were utilized to pull triple trailer trains around a 3 km test oval in the process of applying 4.8 million ESALs. The facility was comprised of twenty-six individual test sections (each at a length of 61 meters), eight of which had to be replaced after 2.8 million ESALs (thus, performance data was available for thirty-four total sections). The primary objectives of the experiment were to develop performance related specifications and validate SUPERPAVE, which meant that both structure and materials were variables in the experiment (Epps et al., 1999). Traffic began on the desert site in 1997, which was selected from various proposals based on the anticipated mild winters and 100 mm of annual rainfall (Mitchell, 1996).

The federally controlled WesTrack project generated an increased interest within state departments of transportation in accelerated performance pavement testing. Where WesTrack was intended to answer broad questions of national significance, many states wanted the opportunity to direct research that would answer local questions important to their individual jurisdictions. In response to this increased interest, in the mid-1990s NCAT began the planning process to develop a cooperatively funded accelerated loading test facility in eastern Alabama. After several years of planning with representatives from regional departments of transportation, the concept of the cooperatively funded NCAT Pavement Test Track would become a reality.

2.2.2 Load Simulation Devices

The development and proliferation of a great variety of load simulation devices after the completion of the AASHO road test would lead many to believe that such technologies are a recent phenomenon, but this is not the case. At the same time that engineers in the United States were focusing on vehicle-pavement interaction through various full-scale tests (e.g., Arlington, Bates, etc.), engineers in the United Kingdom (UK) were working to develop and refine clever and cost-effective load simulation technologies.

2.2.2.1 History

APT research was first conducted in the UK at the Transport Research Laboratory in 1912 with the introduction of the National Physical Laboratory's "Road Machine." The device (shown in Figure 2.3), was later transferred to the Road Research Laboratory (RRL). It ran like a carousel, spinning a 13 kN load at a speed of 14 kph around a 10 m diameter test bed. The device simulated vehicle wander and operated inside a housing with environmental control. Initial work with the "Road Machine" examined the use of asphalt mixes over concrete (Brown and Brodrick, 1999).

In 1933, the original device was replaced by the "Road Machine 2," which propelled a 22.5 kN load at a speed of 48 kph around a 34 m diameter test bed. The next generation "Road Machine 3" was implemented in 1963, which increased the capability of the simulations to 68.5 kN at a maximum speed of 25 kph. Work with the #3 unit focused primarily on investigating elastic responses in layered asphalt systems, concluding that elastic theory worked well for cool, stiff pavements but that the viscous effect at higher temperatures introduced important uncertainties. “Road Machine 3” was effectively replaced by the Pilot Scale Facility, when the laboratory moved in the late 1960s, in which a manually driven truck trafficked 28 m by 7 m by 2 m deep test pits (Brown and Brodrick, 1999).

At about this same time, another APT was constructed at Washington State University in the United States. This facility was a full-scale circular test track that became operational in 1967. It is reported that the Washington circular track was the first US facility designed for modern accelerated performance testing (Hugo and Martin, 2004).

Other APT technologies were evolving simultaneously in the late 1960s. The first prototype unit that would eventually become known as the Heavy Vehicle Simulator (HVS) was under development in South Africa by the Council for Scientific and Industrial Research (CSIR) at this time. Although a production unit would not be completely refined until the late 1970s, HVS research provided the dominant influence in the development of South African pavement engineering capability. It is considered a fundamental tool in developing appropriate pavement structural design and analysis methods, and has been a catalyst for the close cooperation between South African road authorities and researchers ever since. Modern versions of the machine have been built to provide dynamic load simulation as well as an option for super heavy aircraft-type loadings (Kekwick et al., 1999).

Meanwhile, APT technology was progressively evolving in Europe. The Nottingham Pavement Test Facility was commissioned in 1973 in the UK as a half-scale laboratory-housed machine to serve as a theoretical test bed. It was a separate and distinct operation from the Pavement Test Facility at the Road Research Laboratory (RRL), which would not be built until 1985. The Nottingham device loads isolated test pits that are 4.8 m by 2.4 m by 1.4 m deep; however, there was no capability to vary water content in the sealed pits during a test. It is reported that the device is still used constantly for comparative testing. Loading can be varied from 0 to 15 kN via a 150 mm wide tire that travels at a speed of up to 16 kph. Test temperature can be controlled between 15 and 30°C and up to 600 mm of wander can be imposed (Brown et al., 1999).

The Danish Road Testing Machine (RTM) became operational at about this same time (1973) under the ownership of the Danish Road Institute and the Danish Technical University in Lyngby. It is an indoor, full-scale facility that is 27 m long and 2.5 m wide. The actual test section is 9 m long and 2 m deep. The RTM operates within a 4 m wide by 3.8 m high climate chamber that uses heating and cooling equipment to maintain test temperatures between –20 and 40°C. Groundwater can be automatically raised or lowered as desired. Loads can be applied by either single or dual wheels at loads of up to 65 kN. The maximum velocity of the load assembly is 30 kph, which accommodates up to 10,000 load repetitions in a workday. The lateral position of the wheel can be automatically changed during testing to simulate wander (Larsen and Ullidtz, 1998).

Although it is reported that one linear APT and two circular APTs were placed into service in Europe between 1975 and 1980 (Mateos, 2002), global implementation of the technology was not embraced until several years later, at a time when many countries faced growing transportation challenges. For example, only one third of Australia's 800,000 km network of roads has paved surfaces. The loading limits of unbound granular pavements were not well defined by this time, but the continent was in the midst of a twenty-year, fourfold increase in overland freight shipments. For this reason, the Accelerated Loading Facility (ALF) was developed and implemented at an initial cost of $1 million as a successor to their Economics of Road Vehicle Limits study (a quarter-scale linear test track). Between the year the program was implemented (1983) and 1999, over 120 different pavement types had been studied. As continued evidence of the practical nature of their work, recent studies in the Australian ALF have investigated alternative materials and maintenance intervention options (Sharp, 1999).

The French Nantes facility was also commissioned in the following year (1984). Reported to have been inspired by the AASHO Road Test, this unique design utilizes a carousel that travels at a maximum speed of 100 kph via a robust central power source. Adjustable load axle assemblies are mounted at the ends of 20 m long adjustable travel arms, on which between 90 to 150 kN single or multiple axles may be mounted. Although originally built with only one unit, the facility now utilizes three carousels designed for unattended night and weekend operation. In the first fifteen years of operation, 50 million loadings were applied in 180 experiments at a reported total cost of between $500 thousand and $700 thousand per test. Experiments have ranged from product evaluations and model development to load configuration experimentation and examination of equivalency laws (Gramsammer et al., 1999). Each of the three annular test tracks have an outside diameter of 40 meters and an inside diameter of 28 meters. The research pavements slope 2 percent to the interior with a circumference of 110 meters. The mean radius of each track, which may have up to four different pavement structures, is 17.5 meters. Only one of the three existing tracks is built inside an isolating concrete pit with moisture control capabilities (the other two are built on natural soils) (Turtschy and Sweere, 1999).

Another Pavement Test Facility was constructed in 1985 at the UK's Road Research Laboratory to allow the existing Nottingham Pavement Test Facility to continue to serve as a theoretical test bed. The RRL device, which supported more practical APT research, applied wheel loads of up to 98 kN at speeds of up to 20 kph on pavements built in 28 m by 10 m by 3 m deep test pits. The length of each test section is 7 m, and different wheel configurations can be studied in either bi- or unidirectional loading with or without wander (Brown and Brodrick, 1999).

Australian ALF technology was first imported to the United States in 1986 and placed into service at FHWA's Pavement Test Facility (PTF), which is located just outside of Washington, DC at the Turner-Fairbank Highway Research Center in McLean, Virginia. The primary objective of the PTF (shown in Figure 2.4) is to develop and verify new specifications, designs, and test procedures using two separate ALF machines that run tests on alternative pavement designs (structures or materials) or on identical pavement designs but in alternative loading configurations (e.g., by varying tire pressure or axle loading). Each device is capable of applying 35,000 axle passes per week at a load ranging from 44 to 100 kN traveling at a speed of 17.5 kph on a test bed that accommodates twenty-four full-scale test sections, each 14 m by 4 m, with environmental control (Mitchell, 2005). As an example of the type of research conducted at the PTF, a SUPERPAVE validation experiment was initiated in 1994 in which rutting and fatigue were measured in forty-eight test sites over twelve lanes in a effort to study binder properties and performance prediction testing (Sherwood et al., 1999).

Another unique European design has been utilized for Spain's APT, operational in Madrid since 1988. The CEDEX test oval has two 75 m straight sections connected by curves, which produces a total length of 304 m. Each tangent can support up to three 20 m test sections (since only 67 m of the tangents lie within curve transitions). Concrete pits that are 2.6 m deep by 8 m wide (with the capability of being flooded) completely isolate test sections, with sprinklers and covers provided to simulate different performance environments. The CEDEX load bogie runs around a concrete rail on the interior of the oval. Electric power for the guidance apparatus and load assembly is provided via guide rail. A typical dual tire load is 63.5 kN, and at least two bogies can run simultaneously. The device has the capability of simulating wander, with 400 mm of transverse movement control across a 1.3 m wide wheelpath (Turtschy and Sweere, 1999). The initial cost for the CEDEX track is reported to have been $2 million. So far, curve research has been limited to surface courses and roadway paints because of the higher shearing forces (Ruiz, 1999).

An example of a highly functional completely indoor design was built in Christchurch, New Zealand in 1989. The Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) is housed in a hexagon shaped building 26 m wide and 6 m high. An annular tank that is 4 m wide and 1.5 m deep confines the bottom and sides of test pavements that are built in the tank using small dozers, pavers, etc. The resulting track has a median diameter of 18.5 m and a circumference of 58.1 m. Loading is applied by either spring- or airbag-suspended Simulated Loading and Vehicle Emulators (SLAVEs) with loading arm radii that can be set differently allowing for the testing of multiple wheelpaths on a single track. The CAPTIF facility has been used in the DIVINE program (Dynamic Interaction Vehicle Infrastructure Experiment), which was intended to quantify performance differences due to initial material and construction variability and performance differences due to variability in wheel loadings induced by varying tire-suspension dynamics (Kenis and Wang, 1999).

In recent decades, APT technology has also been deployed in Asia. Stabilized bases under asphalt pavements are widely used in China, but most of the early research was with light vehicles at low traffic levels. China's Research Institute of Highways (RIOH) bought an ALF unit from Australia in 1989 to facilitate work on stabilized bases under heavy loads and high traffic levels. Their initial testing program was completed in 1991, but work continues with stabilized bases due to the economy of this method of construction and its continued use in their developing infrastructure. In its current configuration, their ALF can apply loads varying from 50 to 80 kN at speeds of up to 20 kph over 12 m long test lengths. The device can apply 400 load cycles per hour in a unidirectional manner, typically with a dual wheel across a normally distributed wheelpath. There are no means for environmental control (Shutao et al., 1999).

Although the growth of APT facilities in Europe stalled in the early 1990s, work continued on developing programs. Construction of the LINTRACK facility located in Delft, Netherlands began in 1987, with proof testing completed in 1991. This device can apply loads of from 15 to 100 kN via single or dual tires with up to 2 m of wander. At a speed of 20 kph, approximately 1000 bi-directional wheel passes can be applied in a testing hour. The device has provisions for environmental control of experimental pavements that can be built with normal paving equipment. The propulsion system for the load application mechanism is a steel cable wrapped around a powered drum, and the entire assembly operates in a 23 m by 6 m by 5 m shed. Testing is conducted on the middle 3.5 m of test pavements, with 4 m on either end for acceleration and deceleration of the load wheel (thus, 11.5 m total trafficked length) (Houben and Dommelen, 2004). It is reported that LINTRACK is mobile and can be used to test existing pavements (Mateos, 2002).

At the same time that European efforts were leveling off, the continued utilization of ALF technology at the PTF site in Virginia was encouraging more American interest in APT. An original linear device was subsequently built in 1991 at Indiana's Purdue University. The Purdue APT is an indoor facility consisting of an environmental building over an isolated concrete test pit. Heating is applied via plumbing in the pit through which hot water is cycled. A load carriage rolls back and forth down a movable reaction beam at an average speed of 8.3 kph (with or without up to 250 mm of wander) to apply traffic to dual test pavements. Testing can either be conducted unidirectionally (in which the wheel is lifted for the return trip down the beam) or in a bi-directional manner (without lifting). Loads of up to 89 kN can be applied via single or dual wheels, although 40 kN is the typical load. Each paving lane is placed at a width of 3 m, where two tests are conducted on each lane (thus, each test lane is 1.5 m wide). Separate 1.5 m wide concrete slabs lie at the bottom of the test pit in order to simulate the asphalt over concrete roadways common in Indiana (Galal and White, 1999).

Australian ALF technology was next utilized in the United States at the Louisiana Transportation Research Center (LTRC) Pavement Research Facility (PRF) located in Port Allen, Louisiana (across the Mississippi River from Baton Rouge). The first PRF-ALF experiment in 1992 compared historically prevalent flexible crushed stone and in-place soil cement stabilized base construction to alternative base construction methods suitable for semi-tropical climates. In this study, 6 million ESALs were applied to nine test lanes via the second ALF of its type in the United States (the first was at the Virginia PTF). The Louisiana PRF device is equipped such that 43 to 85 kN loads can be applied unidirectionally at 17 kph, which produces up to 8100 passes daily (Metcalf et al., 1999).

There was also interest in the development of APT technology in South America at this time. For example, a traffic simulator was designed and built between 1992 and 1994 at the Rio Grande do Sul Federal University in Brazil. The resulting structure is 15 m long by 4.3 m high and 2.5 m wide and holds a half axle of double wheels. The running speed for the Brazilian device is 6 kph over a 7 m travel path, and testing can be conducted with up to 400 mm of imposed wander (Merighi et al., 2001).

The age and condition of the US transportation infrastructure has influenced the proliferation of domestic APT programs. In 1993, CSIR worked with Dynatest and

Caltrans to validate the use of HVS technology in providing reconstruction guidance for California's aging system. As a result of this successful effort, two South African HVS systems were placed in service at Berkeley in 1994 (Pavement Research Center, 2005). The objective of their ongoing research is to design subsequent pavements for a thirty-year service life with minimum thickness, maximum constructability, and minimum maintenance. Their strategy consists of HVS testing, lab tests, and mechanistic-empirical design. At the time when their program was initiated, approximately one third of their total 24,000 centerline km pavement infrastructure required corrective maintenance or rehabilitation (Harvey et al., 1999).

In 1995, a completely different approach to APT was introduced via the first operational tests of the Mobile Load Simulator (MLS) in Victoria, Texas. In the MLS design, six full standard tandem axles travel around an elliptical path as they traffic a section of pavement, lift off, pass back over (inverted) to the starting position, and traffic the section again (thus, traffic is applied in a unidirectional manner). This action occurs inside of a 31 m by 4.5 m by 6 m tall environmental enclosure that is transportable. The cost to develop the original prototype over a period of five years is reported to have been $3.4 million, although it is estimated that any future units would only cost $2 million to produce. The six tandem axle assemblies are loaded with up to 150 kN each and travel at a speed of 18 kph, which produces 6,000 axle repetitions per hour. Research efforts thus far have been intended to investigate new materials, determine load damage equivalency, and investigate truck component-pavement interaction (Hugo et al., 1999).

A new type of linear tester was constructed in Lancaster, Ohio in 1997 as a joint venture between Ohio State University and Ohio University. The Accelerated Pavement Loading Facility is used to evaluate both asphalt and concrete test pavement installed in a test pit that is 13.7 m by 11.6 m by 2.4 m deep. The device applies up to a 133 kN load via either single or dual tire assemblies. The entire assembly is enclosed in a room in which the environment can be controlled between –10 and 55°C, but large doors on both ends of the building are provided to facilitate conventional construction. Recent testing on the performance of asphalt pavements was followed with testing on ultra-thin whitetopping placed on rutted asphalt from a prior study (Edwards and Sargand, 1999).

Another new linear tester was installed at Kanas State University in 1997 (Hugo and Martin, 2004). The "K-APT" was installed inside a 651 square meter building, and consists of two 1.8 m deep pits of varying width at a length of 6.1 m. Loading can be applied in either a uni- or bi-directional manner with a movable load frame. In the K-APT design, an air suspension tandem axle with pneumatic loading moves back and forth via a drive belt assisted by spring reversal on either end of the travel path. It is reported to take 15 seconds for a complete down and back loading cycle, which produces a wheel speed of 8 kph over a 4.3 m research length. The typical load configuration in Kansas testing is 150 kN on dual wheels (Vijayanath et al., 1999). Temperature can be controlled (from above via infrared heaters and from below via subsurface heating system) between –23C and 66°C (K-State Engineering, 2002). It is also possible to control the water table during testing. Research is funded through a group known as the

Midwest States Accelerated Testing Pooled Funds Program, that includes DOTs in Iowa, Kansas, Missouri and Nebraska (Hugo and Martin, 2004).

Even though many European programs were fully developed by this time, several new facilities were placed into service in the late 1990s. The Danish Asphalt Rut Tester (DART) was designed and installed at the Danish Road Institute in 1997 for the purpose of testing wearing courses. Denmark has 870 km of motorways, with about 300 km overlaid with hot-mix asphalt. DART was developed to test slabs since small laboratory rut testing devices only evaluate single mixes and Denmark was interested in overlay performance (thus, composite structures). The device is capable of applying up to a 65 kN load at a travel speed of up to 5 kph using normally distributed wander. Test slabs are typically removed from a roadway that is in need of an overlay in a manner in which all bound layers are removed and transported to the laboratory. The size of the resulting test pavement typically runs about 120 cm by 150 cm by 5 to 25 cm (depending on the thickness of bound layers in the existing structure). Slabs are cut such that the DART will traffic the overlaid pavement in the same direction that traffic will be applied in the field. Tests are run at temperatures that simulate the anticipated performance environment between 25 and 60°C. DART is equipped with a built in laser profilometer. The performance of the overlaid pavement in the field is predicted based on results from laboratory-overlaid slabs (Nielsen, 1999).

It has become more common for agencies to avoid the necessity of inventing complex machinery by investing in production testing equipment when initiating an APT program. The HVS and the ALF are examples of commercially available technologies;

however, the development of HVS user groups in recent years have arguably made it a more popular choice. For example, Finland and Sweden have operated a joint HVS-Nordic research program since 1997 that is shared and transported between countries. It is reported that $17 million was spent on this program between 1994 and 2001. The device utilizes either dual or single wheels to apply loads that vary between 20 and 110 kN at a speed of up to 15 kph. It is fully mobile with environmental control (Hugo and Martin, 2004).

Likewise, the Corps of Engineers turned to the HVS when they decided to standardize their accelerated performance testing in the late 1990s. Until the 1970s, all of their APT testing had been conducted in the field. In the 1980s, these tests were moved indoors to accommodate greater environmental control and to facilitate mechanistic measurements. As part of this overall strategy, the US Army Engineer Research and Development Center (ERDC) chose two HVS units for deployment at their CRREL facility in New Hampshire in 1997 as well as at their WES facility in Mississippi in 1998 (Lynch et al., 1999).

Although the Corps had HVS units produced to suit their specific needs, it is not possible to build a simple linear beam loading unit that will simulate extreme loading scenarios. Because of new, heavy-duty aircraft, the FAA custom built a 270 m by 18 m enclosed facility at the William J. Hughes Technical Center that began testing in 1998. It is a rail-based vehicle that drives two landing gear trucks, each containing as many as 6 wheels that are adjustable up to 340 kN. Operating speed can be varied between 5 and 15 mph (Merighi et al., 2001).

Many countries continue to develop proprietary technology to address their own specific loading situations. The CEFET research center of Sao Paulo, Brazil has developed one such APT device. The CEFET-SP system uses a dual wheel truck axle in a simulator that can either be used in the lab or taken to remote field projects. Their unit applies a load of 120 kN at a speed of 2.5 kph over a travel path that is only 1 m long. The simplicity of the device means that size has been minimized, where the entire length of the current machine is only 4 m. It is reported that 50,000 loadings can be applied in a 24-hour operating cycle, and because it is relatively simple it can be assembled at remote field locations in about 2 hours (Merighi et al., 2001).

Another newly developed proprietary technology is located at the University of Minnesota and is known as the Minnesota Accelerated Loading Facility (Minne-ALF). Proof testing for the Minne-ALF was completed in 1999 on concrete pavement. The unit has a peak speed of 65 kph, which produces an average speed of 44 kph over the 2.75 m travel path. In the current design, a 40 kN load is applied to a 3.7 m by 4.6 m test slab; however, it is possible to increase the load up to 100 kN if a slower speed is utilized. The rocker beam design applies the full load in one direction and a partial load in the other direction in order to produce the benefit of unidirectional loading while avoiding the dynamics of lost contact. This option is said to be important when testing load transfer in concrete pavement, which was the subject of initial proof testing. It is reported that 172,000 load cycles are currently possible in the laboratory-based device in a test day (Snyder and Embacher, 1999).

Even more recently, the Florida Department of Transportation initiated an APT program in October of 2000. Their Mark IV model HVS is capable of applying loads of between 31 and 200 kN at a speed of 13 kph, and is equipped with an automated laser profiler for measuring rut depths while the system is operating. Wander is adjustable from 0 to 750 mm, and the system can apply up to 14,000 unidirectional loadings in a day. Florida took advantage of the previously stated benefits of purchasing a commercially available APT, and the time between their initial investment and the production of useful data was very short. In order to have greater confidence in their HVS results, Florida is sponsoring research on the 2003 NCAT Pavement Test Track in which they hope to validate past HVS research (FDOT, 2005).

Also in 2000, the Accelerated Transportation Loading System (ATLaS) was developed by the Advanced Transportation Research and Engineering Laboratory with funding from the state of Illinois. ATLaS' wheel carriage can be outfitted with single or dual wheels from either trucks or aircraft. The device is positioned by gantry crane, and test loads (which can be either bi- or unidirectional) are transmitted to the wheels via hydraulic ram. Initial testing is focused on continuously reinforced concrete pavements (Hugo and Martin, 2004).

A completely different type of APT was placed in service at the German Federal Highway Research Institute in 2001. Testing is based on the concept that the passage of a single tire can be approximated by progressive load applications that are induced by a hydraulic actuator-based pulse mechanism. A load of up to 56 kN is applied for 0.025 seconds via a 300 mm diameter load plate. This is intended to represent a maximum legal load traveling at 60 kph. A 2.1 m by 1.8 m section of test pavement is automatically loaded in a progressive distribution that is similar to an in-service pavement subjected to moving traffic. Six million impulses can be applied to a test pavement in 30 days via a frequency of 145 impulses per minute (Golkowski, 2002)

There are reported to be several other APT facilities that have been active in Europe for an unknown period of time. For example, the Zurich IVT-ETH test track consists of an annular concrete pit that is 2 m wide by 2 m deep (thus, ground water can be controlled) with a median diameter of 32 m. The loading system consists of 3 arms, each with dual wheels, that are individually driven by electric motors. Speed in the Zurich APT can be varied up to a maximum of 80 kph (Turtschy and Sweere, 1999).

The Shell Laboratory Test Track (LTT) is located at the Shell Research and Technology Centre in Amsterdam. It is a circular track with an outside diameter of 3.25 m and a width of 0.7 m. Wheel loadings are 20 kN at a velocity of 16 kph. It is possible to control the temperature of testing up to a maximum of 60°C (Lijzenga, 1999).

In Asia, there is another unique and noteworthy APT. The Chinese linear track is loaded with an automated vehicle that travels back and forth via electrical power. Test sections are 60 m in total length, but 7.5 m on either end is neglected due to acceleration and deceleration (leaving 45 m in the middle for pavement research). Ramp areas on either end are sloped at 5 percent in order to utilize gravity to slow the vehicle down and limit wear and tear on braking and acceleration components. The load vehicle runs on ten tires placed on three axles at a maximum speed of 30 kph. The average travel cycle takes one minute to move down to the opposite end and return. The facility is capable of running continuously for seven days without service, and the load vehicle is built primarily with parts provided by Nissan to avoid proprietary engineering and maximize reliability. It is reported to have been used in the study of recycled pavements (Mitsui, 2002).

Although it is unclear how to best categorize the project (test track or load simulator), the Public Works Research Institute of China operates a small test track (628 m by 7 m) that is tied into a larger test track (870 m by 7 m) through turnouts. Buried cables run along the sides of each facility for guidance purposes. Three automated vehicles simulate wander as they use the magnetic signal from the cables to navigate either or both track(s). Test vehicles are loaded to 44.1 kN on steer axles and 58.8 kN on drive axles, where the load on drive axles can be incremented using 19.6 kN metal weights up to a maximum capacity of 156.8 kN. The design speed of the facility is 40 kph, which made it possible to apply 750,000 loadings between 1997 and 2000 (Sakamoto, 2001).

2.2.2.2 State-of-the-Art

It is reported that twenty-eight APT facilities are currently operational, with fifteen located in the United States. These figures are almost certainly an underestimate, as they are based on the results of a survey completed by known facilities. Most are tests at fixed sites; however, there are some that focus on field studies in the belief that there is improved vehicle-pavement-environment interaction. Benefit-cost ratios varying from 1:1 to greater than 20:1 have been reported (Hugo and Martin, 2004).

The majority of European APT facilities (known in Europe as ALT) were commissioned in the period between 1975 and 1985. As of 2002 (at the time the referenced article was written), Europe was reported to have ten full-scale facilities (where full-scale pavement sections were tested by full-scale rolling wheels), two hydraulic actuator-based pulse load facilities (in Bergisch Gladbach, Germany and Dresden, Germany), and several small-scale facilities. Several countries host circular track facilities not well-reported in English literature, including Romania and Slovakia. The reported timeline lists one in 1965, two in 1970, three in 1975, six in 1980, eight in 1985, ten in 1990, ten in 1995, and eleven in 2000 (Mateos, 2002).

The European Cooperation in the field of Scientific and Technical Research (COST) effort was founded in 1971 to encourage coordinated cooperation between members of the European Union in research and development. The Brussels-based organization provides a framework in which research facilities, universities and companies cooperate in a broad range of activities primarily in areas of basic research and pre-competitive research. COST is officially composed of fifteen members from the European Union; however, since 1989 non-COST countries have been able to participate in individual programs (COST, 2004). The COST 347 Group is responsible for coordinating APT efforts throughout Europe.

The coordinating organization in the United States is TRB Committee AFD40: Full-Scale and Accelerated Pavement Testing (formerly A2B09), which was transitionally formed in 2000 from Task Force A2B52. The scope of AFD40 states that the committee is concerned with APT via conventional or accelerated traffic conducted in either the lab or the field with mobile or fixed equipment. The objectives of the group are to assimilate significant worldwide accomplishments from the past and the present, recommend approaches for future practice, use the Internet as a clearinghouse for timely issues, and support the formal transfer of information through reports and presentations. There are approximately twenty-two official members of AFD40, including representatives from government, industry and academia. (TRB A2B09, 2003).

2.3 LABORATORY PERFORMANCE TESTING

Ideally, pavement engineers would prefer to conduct performance experiments without the cost and effort associated with full-scale testing or load simulation devices. Efforts to develop or identify laboratory methods to predict field performance would be useful for approving mix designs and verifying mix quality during production, but have proven elusive. Three basic classes of laboratory devices currently exist with the potential to predict full-scale field rutting.

2.3.1 Testing During Gyratory Compaction

The process of performance verification would be greatly simplified if the candidate test could be conducted at the same time volumetric samples are being compacted. This approach is known as testing during gyratory compaction, and will be described further with respect to the method of analysis.

2.3.1.1 Rate of Compaction

An original goal of the SUPERPAVE mix design methodology was to utilize results from the compaction process to assess the performance potential of prepared mixes. By judging the rate of change in the measured height of the specimen throughout the compaction process, it was thought that meaningful construction and performance predictions could be obtained. Researchers have since found that some parameters measured during gyratory compaction, such as compaction slope, appear to be somewhat related to aggregate characteristics; however, they have also been shown to be insensitive to binder characteristics such as asphalt content and grade (Anderson et al., 2003). It has been proposed that the solution to this problem is to combine gyratory compaction parameters with results from other laboratory tests

2.3.1.2 Gyratory Shear

The SUPERPAVE gyratory compactor (SGC) is the central component of most modern design methodologies in the United States. This device replaced the Marshall hammer because it was felt that gyratory compaction better simulated compaction in the field. Commercially available gyratory compactors (shown schematically in Figure 2.5) function in two distinctly different ways, and can be referenced in terms of the method employed to apply the angle to the ends of the compacted specimen during preparation.

2.3.1.2.1 Wobble Angle

The gyratory testing machine (GTM) has been used for decades by the Corps of Engineers (COE) as an engineering tool for the design and construction of both bound and unbound paving materials. The device was developed and refined in the 1960s as a mechanization of the original Texas gyratory compactor, and is intended to serve as a combination compaction and shear testing machine.

In the late 1940s, WES experiments concluded that lower loads and higher pressures were more detrimental to the HMA mix than higher loads at lower pressures using research conducted with Marshall mix designs. As tire pressures and number of loadings continued to increase after the end of World War II, simply increasing the design blows for Marshall compaction no longer produced stable pavements; consequently, the Corps gyratory method was useful for the construction of stable military airfields (Lynch et al., 1999).

Based on this successful history, the GTM was originally considered to serve as the standard laboratory compaction device for the new SUPERPAVE mix design methodology that was developed as part of the SHRP effort. In the summer of 1990, FHWA abruptly changed the direction of the program and decided the GTM was impractical to use in a mobile laboratory as a consequence of its size and cost. FHWA further decided that the Texas gyratory shear test machine (the GTM's predecessor) was also deficient because its higher angle of compaction produced too few gyrations to facilitate useful laboratory comparisons (Harman et al., 2002).

The GTM has remained a popular tool for researchers who wish to obtain various measures of the stability and shear strength of hot-mix at the same time samples are being compacted. Specimens are subjected to compactive effort using a rotating head (loaded to approximate the contact stress between a tire and the pavement surface) that must maintain a minimum angle via two adjustable rollers positioned on opposite sides of the cylindrical test specimen. As the load head is gyrated, material placed inside the heated mold causes the head to wobble back and forth as the material collapses and dilates. This wobbling motion is monitored with each gyration via automated instrumentation, and is sent to a computer in conjunction with the pressure used to maintain the minimum angle between the two rollers and the changing height of the specimen.

Numerous sophisticated stress-strain type analyses are recommended using the resulting graphical test record, and each can be compared to long-term field performance and evaluated in turn. Additionally, a special GTM testing and analysis procedure was developed specifically for testing Track mixes in the hopes of creating a more suitable method of analyzing modern mixes with modified binder.

2.3.1.2.2 Fixed Angle

Following the 1990 decision to change the direction of the SHRP program away from the GTM, FHWA proposed a hybrid gyratory compactor with an angle of 1.25 degrees. This value was much less than the Texas machine that operated with a 6 degree angle (believed to compact mixes too aggressively), and slightly greater than a French machine that operated with a 1 degree angle (believed to not compact mixes aggressively enough). It has been reported that the slight increase to 1.25 degrees was necessary to ensure that mixes could be designed with 4 percent air voids (Harman et al., 2002).

The final design for the SGC differs from the (floating angle) GTM in that the angle is permanently set to 1.25 (external) degrees and is maintained via three fixed points rather than two. A logical consequence of fixing the gyratory angle in this manner is that the mix may not be able to dilate and collapse beyond what is allowed early in the compaction process before a significant change in density has occurred.

The force required to maintain the fixed angle changes as a sample densifies; consequently, an SGC manufacturer has outfitted a unit to measure this change in load versus number of gyrations. The manufacturer recommends normalizing shear measurements with respect to the compactor's ram pressure because there is said to be a linear relationship between the two (Dalton, 1999). This method produces a gyratory shear ratio (S_r) that changes with each revolution of the compactor.

Theoretically, more rut resistant mixes would produce a S_r record that does not decrease significantly over time, while mixes that deform easily would be expected to produce a S_r record that cannot remain stable. In other words, rut resistant mixes are expected to maintain shear strength as they consolidate under traffic. Analysis of these data has also led researchers to relate the number of gyrations that produces the maximum gyratory shear ratio to design gyrations as a potential indicator test that could warn of a need to investigate a mix further. For example, if the S_r peaks and begins to decrease before the design level of gyrations is achieved, the mix may not perform well under traffic (Harrigan, 2002).

2.3.1.3 Lateral Pressure

Additionally, a relatively new approach to empirical mix testing involves measuring the side forces exhibited by a mix on the mold while it is being compacted. The lateral pressure indicator (LPI) test was recently developed at Worcester Polytechnic Institute (WPI) based upon the concept of lateral earth pressure in soils. As samples are subjected to vertical loading during conventional SGC compaction, twin load cells mounted on opposite sides of the compaction mold measure the pressure between the sides of the specimen and the mold itself. Small windows cut in the sides of the mold are fitted with hinged flaps, and load cells are utilized to measure the force applied to the flaps by the mix under compaction. Higher quality mixes would intuitively be expected to exhibit lower side forces during compaction as the sample itself would support more of the load.

The basic premise of the LPI is that heated hot-mix needs confinement to generate a strength response to the shearing action of the gyratory compactor. If a mix needs a high amount of lateral confinement pressure (i.e., greater than 140 kPa) to develop an interlocking structure that is resistant to shear, it may not perform well in the field over the level of air voids represented by the test. Mixes that exhibit inconsistent changes in lateral pressure may have unstable aggregate structures that will rut (Mallick, 2003).

2.3.2 Simulative Testing

Simulative testing (shown schematically in Figure 2.6) is a type of torture testing in which a small-scale mechanism applies repetitive loadings to a compacted specimen.

Resulting data have been used to screen laboratory designs and rank mixes in an empirical manner based on local experience (Shami et al., 1997). For this reason, simulative testing is sometimes referred to as index testing. Typically, simulative tests are relatively easy to run and results are interpreted in a straightforward manner. Technicians can complete them with little or no expert guidance, and laboratory testing is easily adapted to field use. Primarily because of the low level of technical skill required to run the test and analyze results, this relatively simple class of mix characterization testing has grown in popularity in both European and American practice.

2.3.2.1 European Methods

A common type of simulative device popular with researchers and offered commercially in the United States (US) is known as the Hamburg loaded wheel tester. Although the test method and accompanying pass/fail criterion were popularized in Hamburg, Germany in the 1970's, the mechanical design of the reciprocating wheel system likely originated in the UK (TFHRC, 2001). While the German version of the machine sported a steel wheel, the original UK device utilized a rubber tire (Romero and Stuart, 1998). The device ultimately included in Track research applied a 710 Newton vertical force through a 50 mm wide steel wheel with a 12.5 mm thick rubber contact surface. It had a dual wheel assembly that accommodated testing two SGC specimens simultaneously.

The device has a computer interface that allows the user to plot rut depth versus time via displacement instrumentation on each loaded wheel. SGC samples are placed inside wooden sample holders and mounted on a reciprocating platform that translates a horizontal distance of 230 mm. The rate of loading is 26 cycles per minute, which corresponds to 52 wheel passes per minute. Since the height of acceptable SGC specimens varies by ±5 mm, plaster of Paris is used to fill the small void below each specimen and provide a uniform base for the wooden molds after the test specimens have been installed. Loading is performed inside a heat-regulated cabinet that is temperature controlled with input from thermocouples mounted in holes drilled in the tops of test specimens.

Results from Hamburg testing have been shown to correlate with results from repeated load testing, as well as with results from other types of simulative tests (Bhasin et al., 2005). The Texas Department of Transportation has enough confidence in wet Hamburg testing to utilize it as a mechanism to screen HMA mix designs for rutting susceptibility (Zhou and Scullion, 2004).

The original UK predecessor to the Hamburg machine was known as the Wessex Wheel Tracker. Unlike the steel wheel in the evolved Hamburg device, the load wheel in the Wessex machine was equipped with a rubber contact surface. The load applied in the course of Wessex testing is 530 N at a speed of 21 cycles/minute (Diviani, 2000). Like the Hamburg, Wessex testing can be conducted in either a dry or wet state. In order to avoid the confounding effect of stripping in the diverse types of aggregates used to produce Track mixes, all performance testing reported herein will refer to testing in the dry condition. Since the only substantive difference between Hamburg and Wessex testing is the magnitude and rate of the wheel load, a single device was used for Track testing in which both of these parameters could be adjusted accordingly.

2.3.2.2 American Methods

The Asphalt Pavement Analyzer (APA) is the commercial version of the American research device formerly know as the Georgia Loaded Wheel Tester (GLWT). The APA employs a reciprocating load application mechanism modeled after the old Benedict Slurry Seal tester that applies repetitive loadings on laboratory samples through a pressure regulated rubber tube. Pressure in the tube is automatically maintained at a level that is intended to simulate common truck tires. A heated chamber allows the technician to maintain a constant test temperature that is selected based on anticipated field conditions. A computerized loading system maintains a constant load throughout the testing process. Six samples are tested simultaneously, and electronic instrumentation logs the depth of sample rut depths resulting from the back-and-forth application of load cycles. Results from the APA have been shown to correlate well with rutting measured under accelerated loading conditions, producing a coefficient of determination of 0.95 (Martin and Park, 2003).

A second American device included in Track research is known as the rotary loaded wheel tester (RLWT). This device was developed in the US in the late 1990's in an effort to simplify the load simulation process for use in quality control (QC) laboratories. In this approach, a 125 N force is applied to the surface of single SGC specimens through concentrically mounted, free-spinning load wheels. The load application mechanism is allowed to move vertically, but the free-spinning load wheels accommodate the load mechanism being fixed in the horizontal direction. An integrated temperature controller heats the specimen via an insulated heating jacket based upon input that is received from an infrared sensor. The load mechanism is weighted to approximate a contact pressure of 690 kPa as it applies 140 load applications per minute. The arm that supports the loaded wheel is instrumented so that rut depth over time can be recorded and sent to a computer for real time plotting. The RLWT has been found to correlate with the APA with a coefficient of determination of 0.76 (Edgar, 2002).

2.3.3 Fundamental Testing

The major drawback to simulative testing is that mechanistic material properties are not directly measured. Engineers intuitively search for test methods that generate fundamental data because it can still be useful when field conditions change. In contrast, simulative testing produces results that may only be useful in ranking mixes given the exact conditions under which the test was performed.

By utilizing fundamental test methods (shown schematically in Figure 2.7), an effort is being made to characterize Track mixes in terms of stress and strain related properties. As is the case with empirical (simulative) testing included in the study, results from fundamental analysis methods are ultimately compared to actual rutting observed on Track sections.

2.3.3.1 Shear Testing

The Simple Shear Tester (SST) is a fundamental properties test that was originally proposed as a mix performance evaluation tool in the SHRP design methodology; however, the complexity and cost of the equipment as well as the excessive variability in testing (Romero and Mogawer, 1998) have relegated it to use as a comparative research tool. The simplified procedure for running SST testing published as the American Association of State Highway and Transportation Officials (AASHTO) Interim Provisional Procedure TP 7-01 was utilized in Track research to test specimens prepared during construction for rutting susceptibility.

In this procedure, SST testing consists of applying a repeated haversine shear stress of 68 kPa to a 50 mm thick slice of a 150 mm diameter SGC specimen. Platens are attached to the flat ends of test slices in order to accommodate testing. As a shear stress is induced, just enough axial stress is applied to maintain a constant height. A load cycle consists of a 0.1 second load followed by a 0.6 second rest. Testing is completed when 5 percent permanent strain is measured or 5000 load cycles have been applied (whichever comes first). Testing is performed in a highly specialized, computer controlled load frame and specimens are typically compacted to at 3 percent air voids (VTM) (Zhang et al., 2002). Research has shown that maximum permanent strain is related to rutting in the field (Sousa, 1994).

2.3.3.2 Triaxial Testing

Recently, there has been growing interest in triaxial testing of bituminous specimens using various methods. The repeated load confined creep (RLCC) test is a destructive, controlled stress test (shown schematically as "Trixial" in Figure 2.7) in which a pulsing axial load is applied under constant confining pressure. Typically, confining stress is set at a level that simulates pavements in the field and the axial load is set at a level that simulates a loaded tire. Haversine axial loads are applied at a frequency of 1 Hz, where each load cycle consists of a 0.1 second load followed by 0.9 seconds of dwell time (Zhang et al, 2002). Sample deformation is automatically recorded as a function of number of load cycles. In addition to permanent strain, the number of repetitions required to induce tertiary flow has also been shown to relate to field rutting with a coefficient of determination of 0.75 (Kaloush and Witczak, 2002).

The dynamic modulus of a material is a nondestructive, visco-elastic test response developed under sinusoidal loading conditions. It is typically run on prepared samples right before destructive triaxial testing, and it is the absolute value of dividing the peak-to-peak stress by the peak-to-peak strain for a material subjected to sinusoidal loading. Phase angle (which is computed as the lag in degrees between peak stress and peak strain) is also an output from this test, where a value of 0 is indicative of a purely elastic response and a value of 90 is indicative of a purely viscous response (Harman, 2001). Testing can be completed in as little as 10 minutes and has been deemed by many as suitable for quality control purposes (Christensen et al., 2003).

2.4 PERFORMANCE CORRELATIONS

2.4.1 Laboratory Methods

Although no test has currently been standardized as the official test for validating modern hot-mix asphalt designs, lab tests are routinely used to rank mixes and evaluate the effect of changing methods and/or materials on rutting performance. Primarily due to the high cost of field performance testing, laboratory results have been used extensively in the past to perform factorial experiments in order to differentiate the effect of design properties on mix rutting performance.

In an abbreviated study intended to provide interim guidance to the paving industry until final recommendations could be made (which is expected to take years), simulative testers were the only technologies that could be recommended for immediate adoption and use. The basis of this recommendation was a small study of four mixes in which gravel and stone were tested at both optimum and elevated binder contents. Testing during gyratory compaction, simulative testing and fundamental testing of various types were all used to evaluate study mixes in the laboratory. The Suitability of each technology was assessed based on the assumption that stone mixes should perform better than gravel mixes and mixes at optimum binder contents should outperform mixes at elevated binder contents. Ultimately, the recommendations for permanent deformation testing were the APA, Hamburg and French devices (in order from highest to lowest rank) (Brown et al., 2001).

A more in-depth study was reported in 2002 that compared simulative testing in the APA to fundamental testing in both repetitive triaxial testing and the SST. Mixes blended with two different coarse aggregates, seven different fine aggregates, and one PG64-22 asphalt binder were compacted at three different levels of design gyrations. A total of forty-one unique mix designs were included in the study, with aggregates selected to provide for variety in angularity and surface texture (Zhang et al., 2002).

APA testing was conducted on 64°C samples compacted to 6 percent air voids using a 690 kPa hose pressure and a load of 445 N. Repeated shear constant height (RSCH) testing was conducted in the SST on 50°C samples compacted to 3 percent air voids loaded over 5000 cycles. Repeated load confined creep testing was conducted in a dynamic triaxial device on 60°C samples compacted to 4 percent air voids that were subjected to a 138 kPa confining stress. Samples were loaded 3600 times with a 690 kPa deviator stress and allowed to rebound for 15 minutes before final strain measurements were recorded (Zhang et al., 2002).

Even though test samples were compacted to different levels of air voids (in accordance with recommended practice at the time), researchers reported good correlations between methods. The coefficient of determination between APA and RSCH results was 0.827, while the coefficient of determination between APA and RLCC results was 0.725 (Zhang et al., 2002).

It has also been reported that measured dynamic moduli are sensitive to changes in mixture composition. For example, significant effects have been observed in factorial experiments where changes in coarse aggregate content, mineral filler content, and (to a lesser degree) asphalt binder content caused substantive changes in measure moduli (Christensen et al., 2002). This finding could be important in a practical sense because any laboratory performance test that is used during mix production should be capable of revealing such changes in plant blending percentages.

2.4.2 Laboratory to Field

Of course, the ultimate objective of all laboratory procedures mentioned herein is to successfully identify real differences in field performance. This can be accomplished by either comparing results to accelerated testing or actual field performance.

2.4.2.1 Accelerated Performance Testing

In a study published in 2003, rutting rate in the APA (rather than total rut depth at the end of the test) seemed to provide slightly better correlations with rutting performance on Westrack replacement sections. In this study, samples were compacted to field air voids and the comparison was limited to only five of the Westrack replacement sections due to limitations in materials and difficulty in matching the necessary level of compaction. Also, only one wheelpath from Westrack was used in the analysis (Martin and Park, 2003).

It has also been widely reported that good correlations were observed between results from core testing with laboratory loaded wheel testers (LWT) and the performance of coarse-graded mixes on the Westrack project. A coefficient of determination of 0.76 was reported for the Hamburg LWT, and a coefficient of determination of 0.80 was reported for the APA. In both cases, final rut depth in the laboratory was related to final rut depth on WesTrack without consideration to seasonal effects or rate of loading.

Testing was conducted on core samples that were removed from the surface of the track after 0.6 million ESALs had been applied to tangent sections and 3.3 million ESALs had been applied to curve sections (Westrack Forensic Team, 1998).

It may also be possible to utilize simulative tests to produce results from which useful fundamental properties may be derived. For example, it was recently reported that a comprehensive constitutive model has been developed that uses an algorithm to extract modeling parameters from results generated by a conventional Georgia Loaded Wheel Tester. A related finite element computer program then analyzes the visco-elastic and visco-plastic behavior of hot-mix asphalt based on these parametric inputs (Liang et al., 2003). The existence of such relationships might explain why other researchers have identified simple relationships between fundamental and simulative testing (Zhang et al., 2002).

Of course, inferring value in this finding assumes that fundamental testing is more useful for predicting field performance. Other researchers have reported identifying good correlations between repeated load flow numbers (cycles required to induce tertiary flow) and flow time from static creep testing (time required to induce tertiary flow) of samples compacted to field voids from Mn/Road, (FHWA's) PTF and Westrack; however, both deviator and confining stresses were adjusted on a situational basis. It is not clear if a rationale for adjusting these parameters in unknown future testing is a realistic expectation. In this same effort, it is reported that correlations for other parameters were not so good (Kaloush and Witczak, 2002).

In consideration of the speed in which results can be obtained, it would be ideal if dynamic modulus testing could provide meaningful performance parameters. Recent research recommended compressive dynamic modulus for both rutting and fatigue correlations, with the best fit produced by measuring dynamic modulus at 5 Hz using samples compacted to field voids (Pellinen and Witczak, 2002).

2.4.2.2 Actual Open Roadways

Ultimately, any test that is identified as suitable for predicting rutting under accelerated traffic must also successfully predict performance in the field under actual traffic and in full consideration of the effect of age hardening and environmental variability.

In a recent study, 64°C SGC samples were compacted to 4 percent air voids and tested in the APA outfitted with a 25 mm (outside) diameter hose. Hose pressure was controlled at 827 kPa under a target load of 534 N. It was reported that results correlated well with field rutting values, but it was not possible to make accurate field rutting predictions based on laboratory results. This was because total rut depth in the laboratory was simply related to total rut depth in the field normalized by the square root of the number of ESALs. It was not possible to include a consideration for seasonal effects. The best correlations were still only able to provide a coefficient of determination of about 0.4 using data from the Mn/Road and WesTrack projects. The equation encompassing mixes from both projects took the following form (Kandhal and Cooley, 2003):

$$\frac{Field\ Rut\ Depth}{\sqrt{ESALs}} = 0.7575 \cdot \left(Laboratory\ Rut\ Depth\right)^{-1.7533} \qquad \text{Equation 2.1}$$

In consideration of the inherent difficulty in making field predictions with the APA, Georgia has utilized it for years as a mix design screening test by specifying a maximum 8000 cycle rut of 5 mm at the end of tests run at 50°C (Shami et al., 1997). It is logical that performance test temperature should not be the same for every jurisdiction; consequently, NCAT recommends a maximum APA rut of 8.2 mm after testing at the appropriate high performance grade (PG) temperature for the installed pavement (Zhang et al., 2002). This revised maximum is intended to essentially adjust the 5 mm cutoff up to a comparable average PG value for the State of Georgia, which can then be related (either down or up) to PG values that correspond to other locations.

Research has also been conducted to relate fundamental laboratory results to field performance. Regarding triaxial testing, it has been found that permanent strain is a better overall predictor than either total unrecoverable strain or time dependent creep. This comprehensive study involved 42 different pavements (Brown et al., 2001). It has also been reported that repeated loading produces better correlations than static creep (Barksdale, 1972), which one would intuitively expect since repeated loading more closely simulates the action of passing traffic on the surface of pavements in the field.

Likewise, it has also been found that RSCH testing in the SST corresponds well with rutting in LTPP sections; however, results from the SST are more problematic due to high variability (Anderson et al., 2000). In consideration of this observation, it has been recommended that five samples be tested (rather than the conventional three). Values used for performance correlations would then represent the average of the three closest results (Romero and Anderson, 2001).

Simply designing a mix to perform within a certain level at the design PG temperature will not necessarily ensure the lowest life cycle cost (with respect to rutting) for the installation. Because the relationship between stiffness and temperature in hot-mix asphalt is nonlinear, savings resulting from higher stiffnesses at lower temperatures do not offset the reduction in pavement life as a result of lower stiffnesses at higher temperatures. Thus, temperature averaging over any period of time inevitably results in an overestimation of pavement life in terms of permanent deformation. This principle was illustrated well by researchers who recently studied hot-mix asphalt pavements in Tennessee to support this intuitively correct assertion (Zuo et al., 2003). As a result of this effect, it may be necessary to either correct laboratory results to changing field temperatures or vice versa in order to accurately predict rutting from laboratory results.

2.5 RUTTING PERFORMANCE MODELS

2.5.1 Before the AASHO Road Test

In 1888, Joseph Boussinesq derived isotropic linear elastic field equations to calculate the stresses, strains and deflections that occur in a homogeneous axisymmetric semi-infinite half space under a point load (Berthelot, 2003). Elastic response under loaded areas (such as under loaded tires) could also be computed by mathematically superimposing multiple point loads. This advancement in computational physics would later serve as the foundation upon which mechanistic pavement analysis could develop.

A simple form of mechanistic analysis was arguably first applied to pavements by O. J. Porter in 1938 with the development of the CBR design methodology (Porter, 1938). Known as the “California Method” of pavement design, pavement thickness on top of a given CBR subgrade was selected to limit rutting and fatigue to a tolerable amount for the duration of the design period. Although it was not possible to predict performance, rutting and fatigue in the overlying pavement could be controlled (Sousa et al., 1991).

Porter’s methodology was put to the test in 1940 with the construction of the Brighton Test Track. Constructed by the California Department of Highways, the Brighton study was a full-scale test track consisting of eight different bases ranging in depth from 75 to 450 mm. All test sections were built on top of a uniform silty clay subgrade soil that exhibited a CBR of 3. Experimental pavements were subjected to traffic with a loaded truck. Although it was found that HMA thickness could not be related to base CBR, an orderly and consistent trend with both HMA and base tensile strengths was observed (Hveem and Carmany, 1948).

After Boussinesq’s elastic layer theory was developed to describe single layer structures of infinite width and depth, for over half a century it was not possible to utilize his equations for multi-layered pavement analysis. In 1943, Donald M. Burmister used superposition to expand Boussinesq’s formulations to derive elastic response for two-layered systems. In 1945, Burmister developed three-layer formulations; however, the solutions were so rigorous that he created and published graphical solutions. In doing so, it was then possible to treat pavements as layered systems in order to compute responses at layer interfaces under representative loading conditions (Berthelot, 2003). This development was a milestone for pavement analysis and design.

Another important step in the evolution of pavement analysis was F. N. Hveem's introduction of the concept of equivalent 22.2 kN wheel loads in 1948 (Hveem and Carmany, 1948). By this time, Hveem and his team had come to view HMA as a highly cohesive soil, with a focus on the stability of the underlying subgrade. His design methodology selected a pavement thickness that would prevent plastic deformation and moisture expansion of the subgrade while at the same time preventing fatigue failure of the base and surface layers.

California's CBR design procedure was chosen by the Corps of Engineers to design airfields during WWII. The Corps modified the CBR procedure as well as the California thickness design curve, which had been developed for highway loading, using elastic layer theory in order to encompass a range of aircraft wheels. APT was used analytically to validate and modify derived thicknesses for different aircraft load and gear configurations (ASCE, 1950).

The military's application of elastic layer theory to pavements was not limited to the Corps of Engineers. The United States Navy used two-layered solutions in their design methodologies for airfield pavements in the 1950s to limit surface deflections for specified naval aircraft (US Navy, 1953). Plate bearing tests were used to determine resilient moduli of subgrade soils.

Up until this time, characterization of HMA surfaces in elastic layer analyses had been done without regard to the thermoplastic nature of the asphalt binder. In the early 1950s, C. Van der Poel introduced the concept that the stiffness characteristics of HMA are dependent upon the time of loading and temperature of the mix (Van der Poel, 1954).

Prior to the time of the 1952 WASHO Road Test, Hveem had studied pavement deflections and found a strong relationship with fatigue (Hveem, 1955). This was also an important advancement for mechanistic analysis since up until that time engineers had focused on limiting plastic rather than elastic deformation to control pavement fatigue. A. C. Bekelman subsequently introduced his rolling deflection beam at the WASHO Road Test, which allowed pavement deflections to be measured under slow moving loads (Highway Research Board, 1955). This development facilitated rapid measurement of pavement response and comparison to predicted values in order to estimate future performance, which encouraged the application of elastic response analysis.

The WASHO Road Test also provided an excellent opportunity to compare design standards with observed performance differences in a relatively controlled manner. It was reported that results from the WASHO Road Test seemed to be in agreement with California's CBR design procedure, but not enough data was available to confirm its suitability over a broad range of considerations. A parameter known as the stabilometer resistance value could either be estimated from CBR data or calculated directly based upon the ratio of horizontal to vertical pressure in the laboratory. Equation 2.2 was used to calculate required pavement thickness as a function of traffic requirements and material quality (Hveem and Sherman, 1963):

$$h_{required} = 0.104 \cdot \sqrt{\frac{W}{5}} \cdot n^{0.12} \cdot (90 - R), \textit{ where} \qquad \text{Equation 2.2}$$

$h_{required}$ = *Required Thickness*

W = *Wheel Load in kips*

n = *Load Repetitions*

R = *Stabilometer Resistance Value*

2.5.2 After the AASHO Road Test

Prior to the time of the AASHO Road Test, the focus of most research had been geared toward developing preventive design methodologies rather than actual performance modeling. This changed with the completion of the Road Test because it provided a database of useful information that would allow predictions to be compared with measured performance literally for decades. An abundance of information on construction, material characterization, traffic and the environment would finally allow researchers to scrutinize the effect of each on pavement performance. Very little work had been done on rutting prediction prior to 1960, but the availability of the AASHO data would change the research landscape forever (Sousa et al., 1991). As a validation of the methodology available at the time the test was run, actual thicknesses from the AASHO Road Test were found to relate to computed thicknesses from the California CBR method with a coefficient of determination of 0.87 (Hveem and Sherman, 1963).

The First International Conference on the Structural Design of Asphalt Pavements was held in 1962 to provide a technical venue for discussion of AASHO findings

(Monismith, 2004). At this time, the Shell Oil Company presented the first pavement design approach which explicitly considered both fatigue and rutting as mechanisms of distress (Sousa et al., 1991). Research by the Road Research Laboratory (Whiffin and Lister, 1963) and the Asphalt Institute (Skok and Finn, 1963) demonstrated how Burmister's solutions to multi-layered elastic systems could be used to identify some correlations with pavement deterioration. Work presented from the United States was based on data from the WASHO and AASHO Road Tests (Skok and Finn, 1963), while work from the United Kingdom was based on observed failures of heavily trafficked roadways (Whiffin and Lister, 1963). These approaches were all based on fundamental material properties and accounted in different ways for the effects of variations in loading and environmental conditions on the amount of predicted rutting (Sousa et al., 1991). The model based on the American test roads is presented as Equation 2.3.

$$\varepsilon_p = \begin{pmatrix} -3.661 + 0.096 \cdot h_{surface} + 0.128 \cdot h_{base} + 0.132 \cdot h_{subbase} + 0.391 \cdot \sigma_{subgrade} \\ +0.531 \cdot WP \end{pmatrix} \cdot \sqrt{N_{18}}$$

where Equation 2.3

ε_p = *Plastic Strain*

N_{18} = *Number of 18k Single Axle Loads*

h_i = *Thickness of Various Layers*

$\sigma_{subgrade}$ = *Vertical Compressive Stress on Top of Subgrade*

WP = 1 for Outside Wheelpath, Otherwise Zero

In the UK, the Road Research Laboratory installed test sections on major highways in order to study long-term performance. Five sections were built between the years 1949 and 1960, with test locations selected for their relatively high traffic. Excluding private cars and light delivery vans, commercial traffic on these sections was reported to range from 2,000 to 3,700 vehicles per day. Five different base materials laid in three thicknesses were surfaced with a range of HMA thicknesses. The subgrade CBR for these sections was approximately 20 percent. It was generally observed that thicker bases and/or thicker HMA layers improved performance; however, the type of surfacing and base material seemed to be more important in determining performance than overall thickness. In fact, it was felt the consistency of these relationships was an indication that differences in base layers were not sufficiently varied to prevent confounding of results. Beyond the necessary amount, there did not appear to be an increase in rutting performance with extra HMA thickness (Lee and Crony, 1963).

H. F. Hveem introduced the concept of layer equivalency in 1963. In his study of data from the AASHO Road Test in conjunction with forensic laboratory testing, he quantified the effect of changing temperature on HMA and related thickness of surface layers of various types to equivalent thicknesses of other layer types (Hveem and Sherman, 1963). This advancement made it possible to quantify expected performance differences as a function of layer type in composite structures, which was important to the evolution of mechanistic analysis.

The Second International Conference on the Structural Design of Asphalt Pavements was held in 1967. A number of general solutions for determination of stresses and deformations in multi-layered elastic solids were presented at the 1962 and 1967 conferences, which interacted with rapidly advancing computer technology to lead to the creation of numerous elastic and visco-elastic computer programs. In the late 1960s, finite-element analyses were introduced by a number of researchers, which was useful in considering the nonlinear response of pavement materials (Monismith, 2004).

At the Third International Conference on the Design of Asphalt Pavements the layer-strain methodology for predicting rutting in HMA pavements was presented. One method used nonlinear layered theory, the plastic stress-strain response of component materials, and a hyperbolic plastic stress-strain law to estimate rut depth as a function of load cycles; however, no field verifications were available from which a coefficient of determination could be computed. This proposed methodology, shown as Equation 2.4, assumed it would be suitable to treat HMA in a manner similar to a cohesive soil (Barksdale, 1972).

$$\varepsilon_p = \frac{(\sigma_v - \sigma_h)/(K \cdot \sigma_h^n)}{1 - \dfrac{(\sigma_v - \sigma_h) \cdot R \cdot (1 - \sin\theta)}{2 \cdot (c \cdot \cos\theta + \sigma_h \cdot \sin\theta)}}, \textit{ where} \qquad \text{Equation 2.4}$$

σ_v = *Vertical Compressive Stress*

σ_h = *Horizontal Compressive Stress*

c, θ = *Cohesion and Angle of Internal Friction from Triaxial Testing*

K, n, R = Constants that Define Initial Tangent Modulus and Relate Stress to Strength

Another method was even more abstract and left it up to the individual researcher to input a suitable deformation law for each layer in the pavement structure. Rather than speculate on the significance of predicted values, published work on this method states "It is hardly possible to reckon the accuracy of the calculated results, owing to the number of possible sources of inaccuracy and the poor knowledge one has of the respective possible deviations." The author concludes that "The final results will mostly be worth what the data are worth; they may be valid to within 30 or 50 percent if the physical properties are acceptably well known, but the uncertainty factor will increase notably if the knowledge of these properties leaves to be desired" (Romain, 1972).

Although rutting in pavements is typically some combination of densification and shear deformation, test track studies were presented in which shear deformation rather than densification was the primary rutting mechanism (Hofstra and Klomp, 1972). This finding was in agreement with earlier trenching studies from the AASHO Road Test (Highway Research Board, 1962). It was also shown on the Shell test track that deformation in the HMA was greatest near the loaded surface and gradually decreased at lower levels (Hofstra and Klomp, 1972).

A period of increased interest in rutting performance prediction followed the 1972 conference. In 1974, it was reported that deformation within an HMA layer stopped increasing with increasing layer thickness past a certain threshold value (Uge and Van de Loo, 1974). This finding was consistent with research from the AASHO Road Test, where it was found that surface rut depth reached a limiting value at a certain thickness (HRB, 1962).

A 1976 symposium on rutting in HMA pavements was sponsored by the Transportation Research Board. It included several papers that emphasized rutting prediction and tests necessary for model inputs (Sousa et al., 1991). Of particular interest was a study comparing visco-elastic, elasto-plastic and linear-elastic analysis methodologies for rutting prediction on thirty-two sections from the AASHO Road Test. It was recommended that linear-elastic procedures offer the best long-term possibilities for rutting prediction (Saraf et al., 1976).

Another paper demonstrated how a mix design procedure based on creep testing could be used as a subsystem in an overall pavement management process. The proposed "Shell Method" facilitated the calculation of the decline in ride quality on a pavement as a function of time resulting from permanent deformation (Van de Loo, 1976). Other researchers also utilized results from creep testing, as well as repeated load triaxial testing for rutting prediction (Brown, 1976) (Kenis and Sharma, 1976).

The Fourth International Conference on the Design of Asphalt Pavements was held in 1977. Here, rutting prediction methodologies could be classified into the following five types: 1) statistical regression; 2) permissible strain; 3) elastic analysis; 4) linear visco-elastic analysis; and 5) nonlinear finite element analysis (Sousa et al., 1991).

A statistical regression approach was provided by researchers as a result of an effort to develop a comprehensive distress prediction computer program. Since they did not feel it was practical to utilize a rutting model that was based on mechanistic principles, researchers relied upon a statistical analysis of rutting data from the AASHO Road Test to produce a regression model that was a function of seasonal rutting rate, vertical compressive stress in the asphalt concrete, surface deflection and ESALs. Separate models (shown together as Equation 2.5) were developed for thin and thick pavements, with coefficients of determination of 0.98 and 0.96, respectively (Finn et al., 1977).

$$\begin{aligned}\varepsilon_p(\leq 150\,mm) &= -5.617 + 4.343 \cdot (\log d) - 0.167 \cdot (\log N_{18}) - 1.118 \cdot (\log \sigma_v) \\ \varepsilon_p(> 150\,mm) &= -1.173 + 0.717 \cdot (\log d) - 0.658 \cdot (\log N_{18}) + 0.666 \cdot (\log \sigma_v)\end{aligned}\text{, } \textit{where}$$

Equation 2.5

d = Surface Deflection

An example of a limiting strain procedure was found in the evolving Shell methodology. Shown as Equation 2.6, permissible levels of asphalt strain were determined from extensive laboratory measurements for many different types of mixes over a range of stiffness moduli. Permissible levels of subgrade strain were derived from analysis of AASHO Road Test sections using CBR design methodology. Although no coefficients of determination are provided in referenced literature, researchers report “excellent” agreement between predicted rut depths and results from a laboratory test track as well as a wheel tracking machine. It is noted that transversely distributed loads introduce more uncertainty into the predictive process, which would cause concern in applying this method to in-service roadways with normally distributed wheelpaths (Claessen et al., 1977).

$$\varepsilon_p = C_M \cdot h_{HMA} \cdot \frac{\sigma_v}{S_{mix}}, \textit{ where} \qquad \text{Equation 2.6}$$

C_M = *Correction Factor to Relate Static Creep and Dynamic Rutting*

S_{mix} = *Graphically Derived as Function of Binder Viscosity*

A linear visco-elastic analysis approach was presented in a modeling effort by an FHWA research team. The analytic techniques that serve as the basis of the VESYS computer program were assembled from the application of fundamental principles and evaluations of results from laboratory and field studies by several different prominent researchers. In this program, closed form probabilistic models are used for the prediction of cracking, rutting and roughness. The rutting model, shown as Equation 2.7, applies a repeated loading deformation law with a nonlinear exponential decay factor (thus, visco-elastic) in order to determine the accumulation of permanent strain in each layer in the pavement structure (Kenis, 1977).

$$\varepsilon_p = \varepsilon_e \cdot \left(\mu \cdot N^{-\alpha}\right), \textit{ where} \qquad \text{Equation 2.7}$$

ε_e = *Elastic Strain*

N = *Number of Axle Loadings*

μ, α = *Permanent Deformation Material Regression Constants*

A nonlinear finite element layer-strain approach was presented by a Dublin research group. In this method, the stress and strain field was determined with an iterative finite element elastic analysis procedure. With the stress field modeled, the process reverted to an empirical method to estimate permanent strain from laboratory dynamic creep testing. The resulting model, shown as Equation 2.8, was verified using data from the Shell test track described by Hofstra and Klomp in 1972 as well as data from the University of Nottingham test facility. In the Shell data, a promising correlation was only found in thinner pavements. In the Nottingham data, rutting only compared within a factor of two. The proposed model was also used to estimate load equivalency factors (Kirwan et al., 1977).

$$\varepsilon_p = \left(0.00015 \cdot (\log N)^{1.9} \cdot \sigma_v\right)^{1.75} \qquad \text{Equation 2.8}$$

The methodology described in the Shell Pavement Design Manual utilized a rutting stiffness parameter (shown in Equation 2.9 as $S_{mix,\upsilon}$) that was computed as a function of asphalt viscosity at the average operational pavement temperature, anticipated wheel loading time, and contact stress (Shell, 1978). It was later recommended that it was more appropriate to estimate stiffness from mix properties such as asphalt and air content (Sabha et al., 1995).

$$\varepsilon_p = k \cdot h_{HMA} \cdot \left(\frac{\sigma_v}{S_{mix,\upsilon}}\right), \textit{where} \qquad \text{Equation 2.9}$$

k = *Experimental Coefficient*

$\log S_{mix,\upsilon} = \log a + b \cdot \log S_{bit,\upsilon}$

$S_{bit,\upsilon}$ = *Viscous Component of Binder Stiffness*

a, b = *Mix Specific Parameters*

The Fifth International Conference on the Design of Asphalt Pavements was held in 1982. It has been asserted that the only new information presented on rutting prediction related to limiting vertical compressive strain on the subgrade (Sousa et al., 1991). Later that same year, another empirical rutting prediction methodology was reported (Uzan and Lytton, 1982). This method was a statistical fit that was simply an updated version of the serviceability model developed using data from the AASHO Road Test. In this approach, a mechanistic subsystem was used to indirectly predict rutting as a function of serviceability.

The Sixth International Conference on the Design of Asphalt Pavements was held in 1987. Research showed good agreement between layer-strain analysis using dynamic creep testing and actual rutting in two full-scale test pavements built in France in 1978. In this method, dynamic creep results were broken down into primary, secondary and tertiary phases of testing in order to best relate to changing conditions in the field. The resulting nonlinear model (presented as Equation 2.10) was the first equation to explicitly include mix temperature as a model parameter (Eckman, 1987).

$$\varepsilon_p = \left(\frac{10^{a_1 + b_1 \cdot T + c_1 \cdot \sigma_v + d_1 \cdot \left(\frac{1}{\sigma_h + 1} \right)}}{10^{6 \cdot \left(a_2 + b_2 \cdot T + c_2 \cdot \sigma_v + d_2 \cdot \left(\frac{1}{\sigma_h + 1} \right) \right)}} \right) \cdot t^{\left(a_2 + b_2 \cdot T + c_2 \cdot \sigma_v + d_2 \cdot \left(\frac{1}{\sigma_h + 1} \right) \right)}, \textit{where} \qquad \text{Equation 2.10}$$

a_i, b_i, c_i and d_i = Mix Specific Regression Coefficients

T = In-Service Mix Temperature (°C)

t = Loading Time in Seconds

Another effort concluded that rutting in the secondary phase was mainly caused by deformation flow without volume change. In comparison, volume reduction alone was responsible for primary consolidation, and volume increase (dilation) was active in the tertiary phase. In this study, a wheel system was run on representative slabs in the laboratory using different loads and inflation pressures in single and dual tire systems. The predictive equation for this effort is presented as Equation 2.11 (Eisenmann and Hilmer, 1987).

$$RutDepth = a + b \cdot \sqrt{N}, \textit{where} \qquad \text{Equation 2.11}$$

a,b = Regression Coefficients

In 1989, a statistically derived predictive model for permanent strain was published that was a function of temperature, mix and constituent properties, load repetitions, and deviatoric stress. The nonlinear regression model shown as Equation

2.12 was recommended to relate the ratio of cumulative plastic to elastic strain based on test results from over 250 HMA specimens representing various binder types, asphalt contents, stress levels, temperatures and aggregate types. The overall coefficient of determination for this effort was 0.76, but limitations on potential applications were reported. Temperature was shown to be by far the most important variable, and the effect of mix properties was quantified (Leahy, 1989).

$$\varepsilon_p = \varepsilon_e \cdot 10^{-6.631+(0.435\cdot\log N)+(2.767\cdot\log T)+(0.110\cdot\log\sigma)_v+(0.118\cdot\log\eta)+\left(0.930\cdot\log V_{beff}\right)+(0.5011\cdot\log V_a)}, \textit{ where}$$

Equation 2.12

η = Viscosity at 70°F (10^6 Poise)

$V_{\mathbf{b}eff}$ = Effective Volumetric Binder Content

V_a = VTM

The Seventh International Conference on Asphalt Pavements was held in 1992 in Nottingham, England. A general shift in focus occurred at this conference towards the more practical elements of asphalt pavement engineering, which coincided with more research emphasis on implementation. Various teams worked to make the utilization of fundamental properties accessible to practitioners at every level. For example, dynamic modulus is a mix characterization parameter that is intended to represent both elastic stiffness and internal damping. In this manner, it fully describes the stress-strain relationship of visco-elastic materials. Preparation and testing of laboratory samples that yields meaningful dynamic modulus results can be challenging for engineers who are accustomed to traditional index testing. Implementation research suggested that dynamic moduli could be estimated via regression equation (presented as equation 2.13) as a function of asphalt viscosity, load frequency, percent air, effective binder content and several different gradation parameters (Fonseca and Witczak, 1996).

$$\log E = -0.261 + 0.008225 \cdot p_{200} - 0.00000101 \cdot p_{200}^{\;2} + 0.00196 \cdot p_4 - 0.03157 \cdot V_a - 0.415 \cdot \frac{V_{b_{eff}}}{V_{b_{eff}} + V_a} + \left(\frac{1.87 + 0.002808 \cdot p_4 + 0.0000404 \cdot p_{3/8} - 0.0001786 \cdot p_{3/8}^{\;2} + 0.0164 \cdot p_{3/4}}{1 + e^{(-0.716 \cdot \log f - 0.7425 \cdot \log \eta)}} \right)$$

where Equation 2.13

E = Dynamic Modulus in 10^5 *psi*

p_{200} *= Percent Retained on #200 Sieve*

p_4 *= Percent Retained on #4 Sieve*

$p_{3/8}$ *= Percent Retained on 3/8 inch Sieve*

$p_{3/4}$ *= Percent Retained on ¾ inch Sieve*

f = Load Frequency in Hz

The trend in implementation continued with the Eighth Conference, which was held in August of 1997 in Seattle, Washington. This event was quickly followed by the First International Conference on Accelerated Performance Testing in 1999 in Reno, Nevada. The 1999 Reno Conference highlighted major work in the field of rutting performance prediction, with special focus on APT. The inaugural APT conference was intended to be to the WesTrack project what the First International Conference on the

Design of Asphalt Pavements was to the AASHO Road Test – a technical venue for the discussion of road test results.

For example, in a significant research effort it was found that incremental rut depth on WesTrack sections could be predicted as a function of material characterization, environmental conditions and time hardening. Parameters included in the regression model included stiffness, asphalt content, field air voids, cumulative rut depth, previous incremental change in rut depth, percent compaction at initial gyrations, dust proportion, and film thickness. Multiple interactions were also included, resulting in a respectable coefficient of determination of 0.80. It is noteworthy that this method accounted for both environmental conditioning and time hardening, with emphasis on temperature sensitivity in the early life of the mix. It successfully predicted nonlinear WesTrack rutting and was intended to be applicable to other climatic regions; however, the research team cautioned that local calibration and verification would be required, and that it might not be sensitive enough to capture the effect of high asphalt contents on rutting performance (Hand et al., 1999).

Other researchers highlighting WesTrack findings excluded densification and assumed rutting was controlled by shear deformation. Their premise was that permanent shear strain could be modeled via regression as a function of elastic shear response and axle loadings. This method (shown numerically as Equation 2.14) employed an hourly change in elastic shear strain in order to encompass time hardening. Rutting was also predicted via direct regression (via Equation 2.15) as a function of traffic, the environment and mix parameters. Unfortunately, researchers cautioned that both predictive methods were only applicable to WesTrack mixes (Epps et al., 1999).

$$\gamma^{p}_{50\,mm} = a \cdot \exp\left(b \cdot \tau^{e}_{50\,mm}\right) \cdot \gamma^{e}_{50\,mm} \cdot N^{c}, \textit{ where} \qquad \text{Equation 2.14}$$

$\gamma^{p}_{50\,mm}$ *= Plastic Shear Strain at a Depth of 50 mm*

$\tau^{e}_{50\,mm}$ *= Elastic Shear Stress at a Depth of 50 mm*

$\gamma^{e}_{50\,mm}$ *= Elastic Shear Strain at a Depth of 50 mm*

a, b, c = Regression Coefficients

$$\ln ESALs_{10\,mm} = A_0 + A_1 \cdot P_{asp} + A_2 \cdot V_{air} + A_3 \cdot P_{asp}{}^2 + A_4 \cdot V_a{}^2 + A_5 \cdot P_{asp} \cdot V_{air} + A_6 \cdot fines + A_7 \cdot finesplus + A_8 \cdot coarse, \textit{ where}$$

Equation 2.15

$ESALs_{10\,mm}$ *= Traffic Loadings to a Rut Depth of 10 mm*

P_{asp} *= Binder Content by Weight*

A_i *= Regression Constants, i=0,1,…,5*

fines, fineplus and coarse = 1 for specific mix types (otherwise zero)

Another effort simply correlated laboratory results with final rut depths on the WesTrack project. In this work, the research team successfully related RSCH performance to track rutting (using Equation 2.16) with a coefficient of determination of 0.49. Here, a power function was used to characterize each individual laboratory test. As a lesson learned in pursuit of this mediocre correlation, the importance of test temperature and the suitability of laboratory compaction was emphasized (Williams et al., 1999).

$$\varepsilon_p = a \cdot N^b = percent\ permanent\ strain,\ where \qquad \text{Equation 2.16}$$

N = Number of Load Applications

a,b = Modeling Constants

Of course, the inaugural APT conference was also an opportunity for a largely American audience to learn about accelerated performance research being conducted in other countries. For example, it was reported that rutting in a Chinese ALF was confined to the asphalt surface layer. The contribution of deterioration of the asphalt-base interface resulting from water ingress through cracks was also noted. Different temperatures and rainfall patterns during testing made it difficult to directly compare performance between test pavements. The research team used a simple power model to relate number of loadings to permanent strain. Since each section had a unique model, no general coefficient of determination was provided (Shutao et al., 1999).

During this period of significant international technology transfer, the SHRP effort to implement mechanistic pavement design in the United States was moving forward. Leahy's model was scrutinized again in a study that incorporated laboratory test data for AASHTO's Mechanistic-Empirical Pavement Design Guide (MEPDG). It was found that the Leahy model could be simplified to only include terms for load repetitions and mix temperature. The coefficient of determination for Equation 2.17 dropped to 0.72, which researchers still considered very good when modeling permanent deformation (Ayres, 2002).

$$\varepsilon_p = \varepsilon_e \cdot 10^{-4.80661+(2.58155 \cdot \log T)+(0.429561 \cdot \log N)} \qquad \text{Equation 2.17}$$

The Ayres simplification was taken one step further by the NCHRP 9-19 "SUPERPAVE Models" research team. This examination used the original Leahy/Ayres data in combination with 9-19 results. With this larger database, a final model encompassing terms for load repetitions and mix temperature was recommend for the MEPDG. The coefficient of determination for the recommended model (shown as Equation 2.18) was 0.64 without field calibration, and it is expected that local calibration will improve its effectiveness (ARA, 2004). This model is generally considered to represent the state-of-the-art in rutting performance prediction.

$$\varepsilon_p = \varepsilon_e \cdot \left(\beta_{r1} \cdot 10^{-3.15552} \cdot T^{1.734 \cdot \beta_{r2}} \cdot N^{0.39937 \cdot \beta_{r3}}\right), \textit{where} \qquad \text{Equation 2.18}$$

β_{r1}, β_{r2}, β_{r3} = Local Calibration Factors

2.5.3 Current Research Needs

In consideration of this detailed historical perspective, it is proposed herein that modern predictive models for HMA rutting performance do not address several factors that can have a significant impact on real world rutting performance:

1) The majority of past modeling efforts have been tuned to conditions that are project specific; thus, widespread implementation is not likely;
2) It may not be practical to adapt existing methodologies to real world construction in a manner that will accommodate performance prediction for quality control. Theoretical approaches that take days or weeks to complete cannot be plotted on a control chart and used to monitor the changing quality of thousands of tons of HMA while it is being produced;
3) It has been shown that the difference in compaction between laboratory testing and mat placement has a multidimensional confounding effect on predictions of primary, secondary and tertiary consolidation in the field;
4) It is currently impossible to apply laboratory results across the real world temperature spectra without some type of master curve; and
5) Broadly applicable time-hardening methodology is problematic for the practicing engineer.

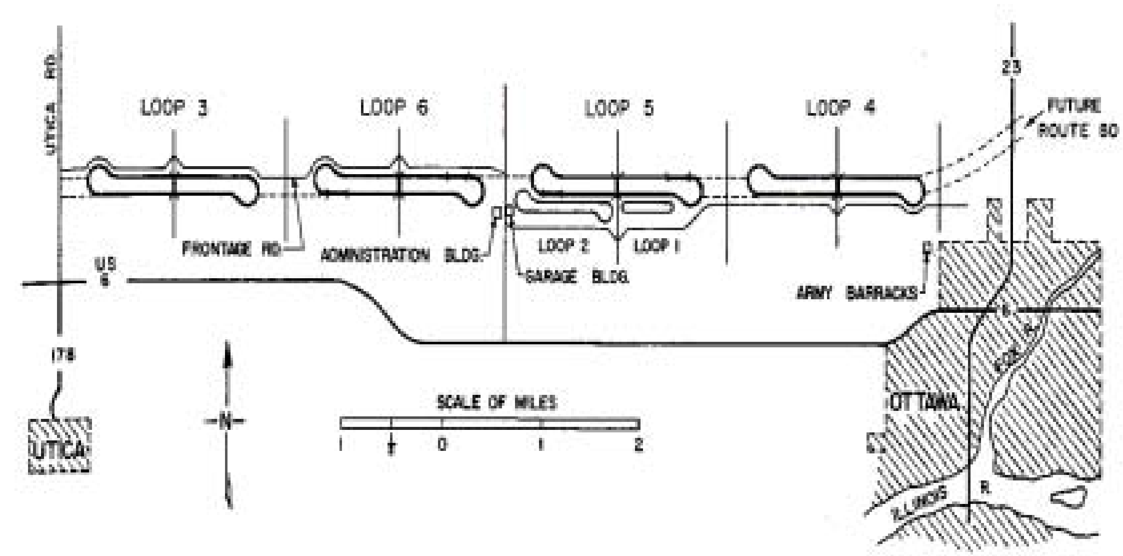

FIGURE 2.1 Layout of Early Track Testing via the AASHO Road Test.

FIGURE 2.2 Later Test Track in Reno, Nevada.

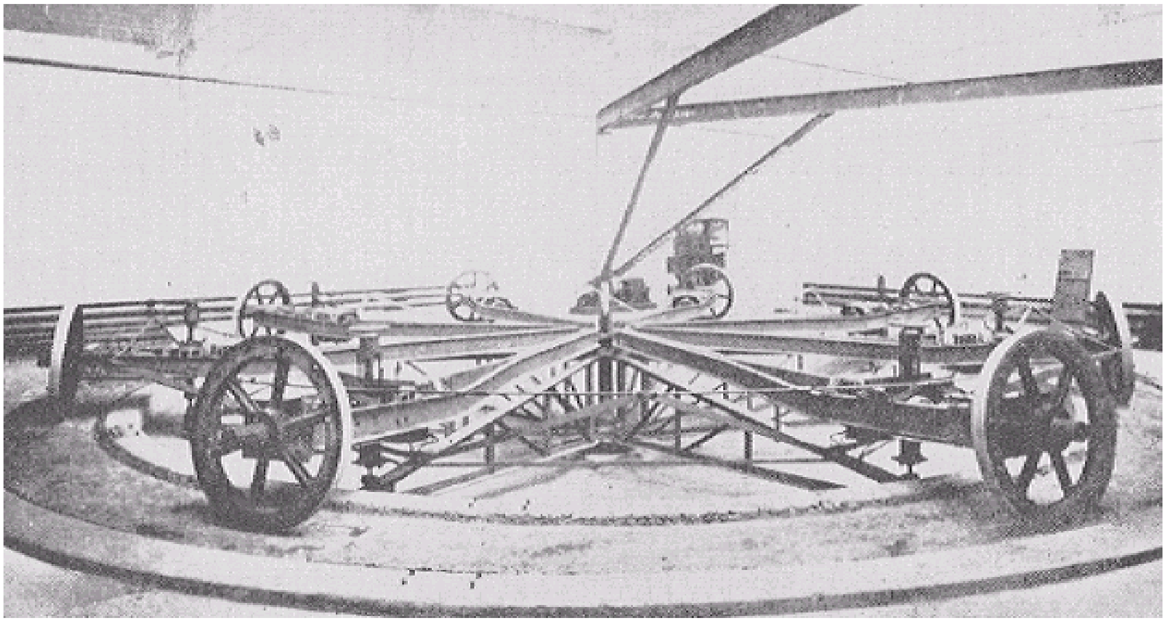

FIGURE 2.3 Early (1912) Load Simulator Testing Asphalt Mixes Over Concrete (Brown, 1999).

FIGURE 2.4 Later Load Simulator Testing via ALF at FHWA's PTF.

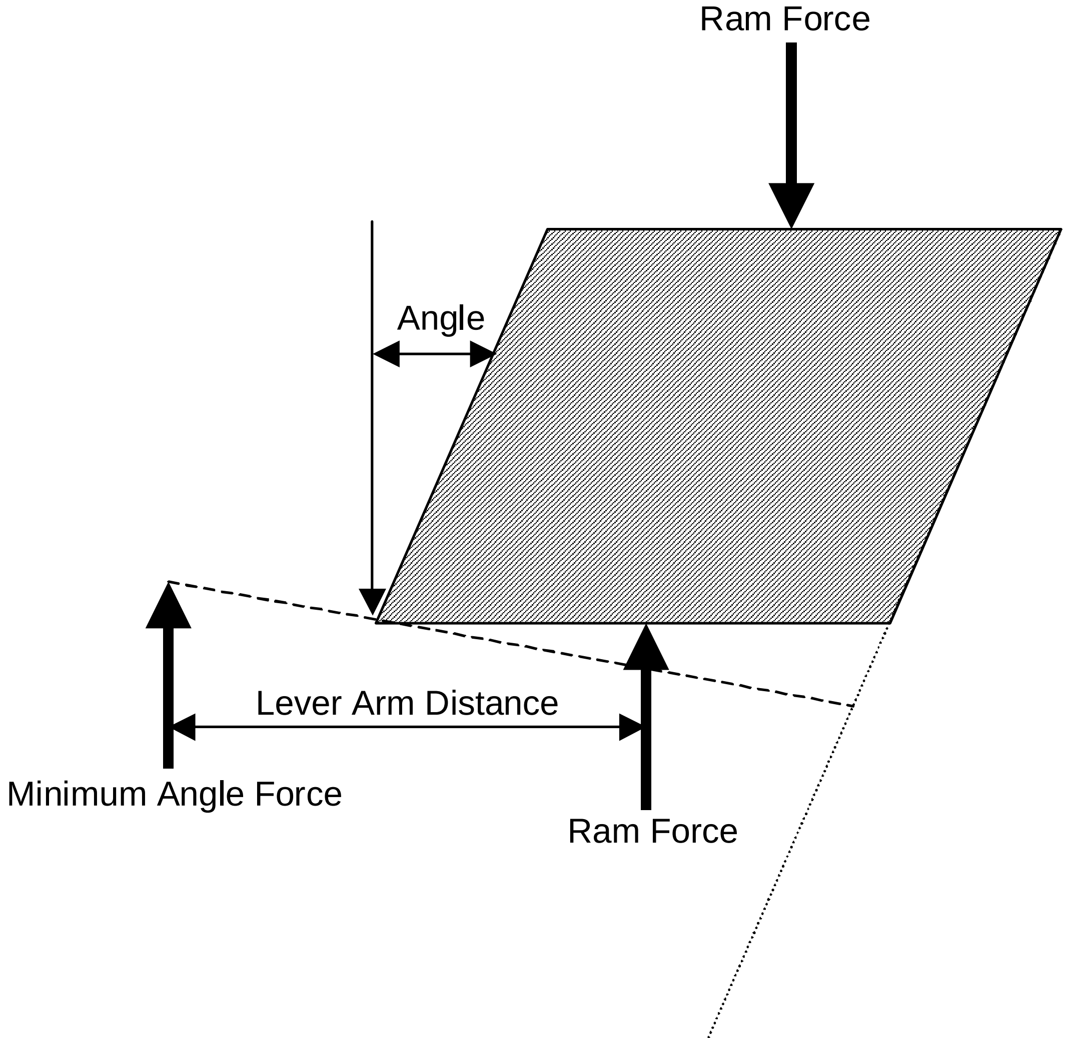

FIGURE 2.5 Schematic of Gyratory Compaction (SGC Angle is Fixed in 2 Dimensions, While GTM Angle is Fixed in Only 1 Dimension).

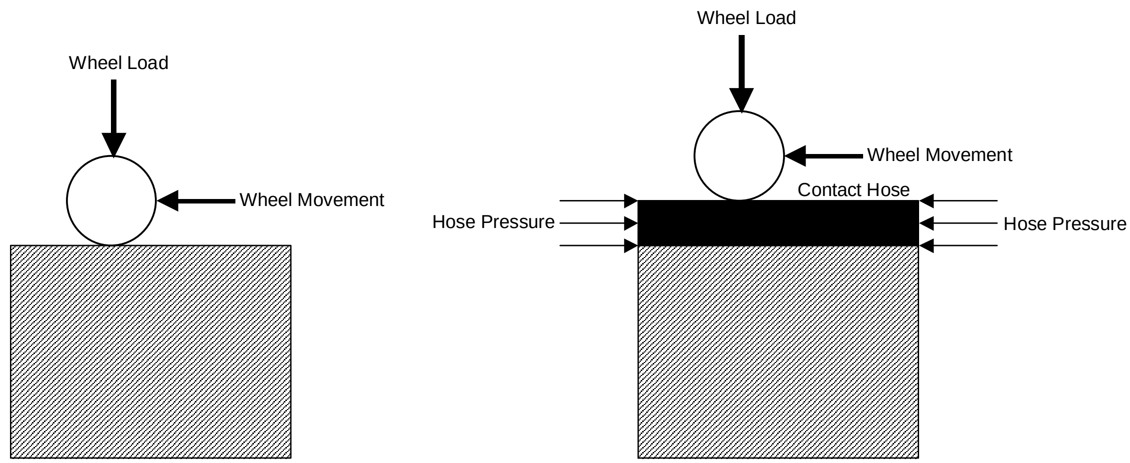

FIGURE 2.6 Schematic of Controlled Load (Left) versus Controlled Pressure (Right) Loaded Wheel Testers.

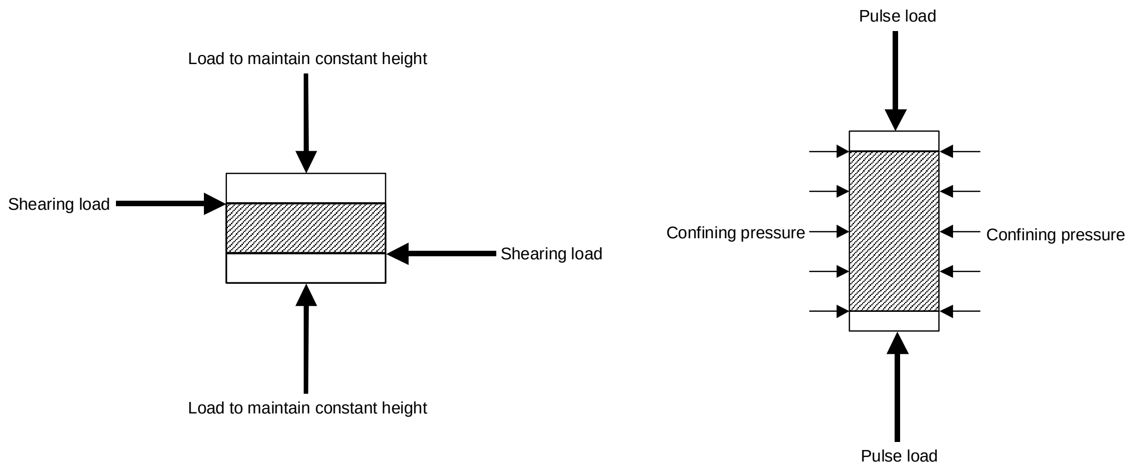

FIGURE 2.7 Schematic of SST (Left) versus Triaxial (Right).

CHAPTER 3 – RESEARCH PROGRAM

3.1 PROJECT DEVELOPMENT

3.1.1 Organization

The interest in accelerated performance pavement testing generated by the Westrack project was the starting point in the development of the Pavement Test Track. The Westrack project had generated enough regional interest that the operation of a southeastern pavement test track appeared viable; however, it would be necessary to develop a business model that provided individual state sponsors with enough oversight and control to make participation worth their while. The primary distress mechanism under investigation in the initial research cycle on the NCAT Track was rutting.

3.1.1.1 Funding

Several DOTs and the Federal High Administration (FHWA) agreed to sponsor the work at the NCAT Track through a pooled fund that would be managed by the Alabama Department of Transportation (ALDOT). Research would be administered by NCAT in a cooperative manner such that each sponsor funded a prorated share of the project's expenses. The cost of participation could then be determined by the length of each experiment as a percentage of the total Track tangent length. A robust structural foundation could support surface mix performance studies such that wearing layers could be periodically milled and replaced to facilitate a perpetual testing program that would survive as long as the research need persisted.

The computation for prorating the cost of each tangent section would have been very high for the first cycle of testing if first round sponsors had to pay for the cost to purchase the land, develop the Track, and build on-site facilities. To facilitate the partnership and control initial costs, land for the operation was located and purchased by Auburn University in a remote location that was not too far from campus. In exchange for four research sections in the first and second cycles of testing, ALDOT developed the initial plans and funded construction of the Track's foundation up to the bottom of the first experimental mixes, as well as an on-site laboratory and truck maintenance building.

This initial investment in the testing program by ALDOT, which was supported by a succession of Transportation Directors, served to limit the cost of sponsorship to a prorated share of surface mix construction and operational expenditures. Auburn University originally paid \$500 thousand for the 309-acre parcel of land on which the Track would be located. The total cost of the construction of the facility (including the placement of experimental mixes) as \$7.5 million. Of this amount, \$2.3 million was spent on the grade and drain phase, \$1.4 million was spent on on-site buildings (including the testing laboratory, truck maintenance building, plant bathroom, and two deep well pump houses), \$3.4 million was spent on the perpetual pavement foundation, and \$0.4 million was spent to replace continuous mix with 20 additional test sections in the curves (Powell, 2000). A total of \$0.7 million in construction costs was repaid by nine section sponsors (Powell, 2001), who each contributed a basic fee of \$495 thousand per pair of tangent sections (to support both construction and operational costs). The average annual cost to operate the first three-year cycle of testing on the NCAT Pavement Test Track was \$1.3 million (including two years of trucking costs, staff salaries, etc.), which places the total cost to complete the first pavement rutting study at \$11.3 million. Although trucking expenses can vary significantly as a function of the global fuel market, the total project cost is expected to be less in future research cycles because the research infrastructure (e.g., Track foundation, on-site facilities, etc.) is already in place.

3.1.1.2 Planning

This funding level represented a tremendous investment in pavement research that required an oversight structure to ensure that everyone's research needs were properly satisfied. An oversight committee consisting of representatives from interested state DOTs was initially formed in which sponsors were encouraged to consider research efforts by other sponsors as they developed comparison studies for their own section pairs.

Through a series of planning meetings that began on August 13, 1998 (Buchanan, 1998) and ran through the Summer of 1999, the Track's "research co-op" approach ultimately facilitated an overall experiment where sponsors could design their own individual studies with consideration given to the efforts of their partners from other states. As a result, the experiment was not formally designed and a great variety of mix types and materials ultimately evolved to best serve the needs of research sponsors.

3.1.1.3 Oversight

After selecting studies that addressed individual research needs, sponsors continued to actively provide project oversight. Many sponsors performed their own mix designs and coordinated the availability of contractor-donated stockpiles. During construction, each sponsor had the option of sending a delegation to Auburn to supervise plant, laboratory and roadway practices (shown in Figure 3.1). In all cases, NCAT personnel were responsible for running tests and compiling results; however, sponsor representatives were each given the opportunity to make final adjustment and/or acceptance decisions. The Track Manager served as the sole project contact for the entire sponsor oversight committee to maintain a clearly defined chain of command through ALDOT contract administration personnel to the contractor.

Sponsor oversight continued after construction was completed. Every six months, the project provided travel expenses to bring a single representative of the sponsor's choosing to Auburn to participate in committee meetings. These regular events were intended to ensure that everyone maintained close communications with project personnel and remained fully aware of the direction in which the project was being managed. A portion of these meetings involved a physical inspection of the Track, while a portion involved classroom presentations on every aspect of the work (construction, trucking, testing, etc.). The project web site (www.pavetrack.com) was used over the entire course of the study to communicate messages and preliminary results to project sponsors.

3.1.2 Experiment Design

David Volkert and Associates was selected by ALDOT to develop the plans for the construction of the Track as well as the on-site buildings. Volkert representatives attended meetings involving ALDOT, NCAT and the sponsor oversight committee as various details of the project were finalized. It was decided that construction would progress in two phases – the first phase involving the completion of the buildings and Track up to the bottom of the subgrade, and the second involving the installation of the foundation and overlying experimental mixes. Conventional specifications were utilized in the first phase, and a custom specification was developed through a joint NCAT-ALDOT effort for the second phase.

3.1.2.1 Overall Experiment

Through a series of preconstruction meetings, the sponsor oversight committee designed the experiment to suite each sponsor's needs. Simultaneously, NCAT attempted to guide the cumulative effort into an overall experiment that would address major policy issues for the industry as a whole. The most common area of interest to 2000 Track sponsors was in identifying the effect of gradation (above the maximum density line versus below the maximum density line) on rutting performance.

When the final experiment had taken shape, five of ten sponsoring entities (nine states plus FHWA) chose to study the effect of gradation on rutting performance. Two states chose to study the effect of mix design methodology on rutting performance, two states chose to compare gravel mixes to stone mixes, and one state chose to compare mixes with different design nominal maximum aggregate sizes. Most sponsors elected to ship in their own unique local aggregates while relying upon a common, representative source for asphalt binders in order to eliminate the effect of binder supply on performance.

Table 3.1 is included to generally summarize the nature of the final consensus experiment, which resulted from numerous meetings of the oversight committee over several months. A schematic of the Track layout is included as Figure 3.2 so that the placement of each section can be determined.

3.1.2.2 Pavement Structure

The underlying pavement structure is robust to avoid fatigue failures and is identical for every section on the Track. The final buildup (shown in Figure 3.3), which was developed by ALDOT under the supervision of the oversight committee, consisted of (from the bottom up) on-site common subgrade (the same under all test sections), 150 mm of dense crushed aggregate base, 125 mm of permeable asphalt treated base, 225 mm of SUPERPAVE lower base mix (produced with unmodified asphalt), 150 mm of SUPERPAVE upper base mix (produced with modified asphalt), then 100 mm of experimental mix (typically placed as a 50 mm binder mix under a 50 mm surface mix). The entire Track is underlain by edgedrains that run along both the inside and outside edges of the pavement structure, fed by water from the permeable asphalt treated base as well as water that leaches in from the shoulders.

The aforementioned buildup was intended to provide a perpetual research platform upon which a virtually unlimited number of mill/inlay cycles of pavement performance studies could be conducted. Based on ALDOT structural coefficients that were applicable at the time the Track was constructed, the composite structural number for the completed Track was calculated as 10.83. Based on the 1993 AASHTO Design Guide, the Track foundation was expected to survive a total projected traffic life of 348,111,873 ESALs, which would represent almost thirty-five cycles of traffic. Clearly, the NCAT Pavement Test Track can be identified as a perpetual pavement.

3.1.2.3 Experimental Mixes

As a benefit of sponsorship, NCAT offered to complete all mix designs that would be used to construct the Track. Most sponsors chose to build sections with mixes that already had a performance history on their roadways; consequently, NCAT was only asked to design twenty-nine of the forty-nine unique mixes used to construct the upper 100 mm of experimental pavement (including mixes on the curves and the tangents). Five sections not included in these figures were built with aggregate blends identical to blends in an adjacent section, but with elevated asphalt contents. Eight of the NCAT-designed mixes were placed on the tangents, and twenty-one were placed in the curves. Methods, materials and specifications representing the standard of practice within the sponsors' respective jurisdictions were used to complete the NCAT mix designs.

3.1.3 Experimental Plan

An experimental plan was developed in order to identify models that could be used to predict field rutting performance from laboratory test data:

1) Test sections containing sponsors' mix designs would be installed on the surface of the Track;
2) During construction, a large number of SGC test specimens would be prepared using actual plant-run material at compaction levels comparable to QC pills;
3) Heavy triple trailers would be used to apply accelerated loading to the surface of experimental pavements;
4) Multi-depth pavement temperatures and relevant environmental conditions at the time of each truck pass would be recorded;
5) Field performance would be measured on a weekly basis in order to characterize the change in rut depth resulting from each week of truck traffic;
6) Samples compacted during construction would be subjected to an array of laboratory testing protocols that have the potential to relate in some way to field performance; and
7) A method would be identified to relate laboratory and field performance as some function of traffic, temperature, lab performance, age, etc.

Proposed models must provide a practical mechanism through which practitioners can predict rutting performance on open roadways with only information that is either known, measured or can be assumed before or during construction.

3.2 TEST SECTION CONSTRUCTION

3.2.1 Constituent Materials

Since the vast majority of hot-mix asphalt is produced with local materials, it was important to provide sponsors with the option of shipping local materials to produce research results having maximum local significance.

3.2.1.1 Source Selection

Sponsors selected their own sources for solid mix components, and the entire sponsor oversight committee selected a single binder source with a broad geographical influence to maximize the applicability of research results. Aggregate stockpiles and asphalt binders were hauled in from eight different states in order for the research to adequately reflect the local interests of the sponsoring entities (nine states and FHWA). Most sponsors sought material donations from local contractors and aggregate suppliers; however, hauling costs represented the bulk of the expenditures and were paid through NCAT by the respective state sponsors.

In order to control variability in field performance comparisons, a single source was used to supply the neat (PG 67-22) and SBS modified (PG 76-22) asphalt binders; however, it was determined that this supplier could not guarantee that the needed SBR modified (PG 76-22) binder would arrive at the on-site plant at the proper grade (when shipped in uncirculated tankers). Consequently, the neat/SBS supplier shipped unmodified material to a second, more local supplier who added their SBR modifier then delivered the final product to the plant site under certification for quality. Near the end of the project, an SB modified (PG 70-28) asphalt binder from another supplier was also utilized for the production of two adjacent sections as specified by the research sponsor.

3.2.1.2 Supply Logistics

Sponsors were required to secure stockpile donations, make them available for timely transport, then provide a funding mechanism that could be used to reimburse the Track for hauling costs. Local hauling companies with nationwide capability were enlisted to accomplish the daunting task of delivering stockpiles just in time before production to allow for a sufficient amount of stockpile testing (gradations, moistures, etc.). NCAT personnel successfully coordinated between aggregate suppliers, trucking dispatchers and the on-site laboratory to accomplish this critical task. Two different local hauling companies were enlisted to complete the work in a timely manner. An elaborate mapping system was developed in order to ensure the identity and use of each separate stockpile. Figure 3.4 is provided to illustrate the complexity of stockpile management during Track construction.

3.2.1.3 Verification Testing

Mix designs were completed using samples from stockpiles and in consideration of gradations from source production records. As loads were delivered and any time a problem was otherwise suspected, multiple gradations were run and averaged to represent material on the yard. Every morning, multiple samples from each stockpile scheduled for the day's production were tested for moisture content. Plant settings were subsequently adjusted to reflect moisture contents measured on each day of production; however, drought conditions in the spring and summer of 2000 provided ideal conditions for producing hot-mix asphalt with little fear of variable stockpile moisture.

3.2.2 Mix Production

The Track was built via a competitively bid construction contract that was let and administered by ALDOT. The selected contractor (APAC-Couch) was required to supply an on-site plant, a material transfer device (MTD), a rubber-tired roller, and a host of other equipment (each meeting a particular specification requirement) that would be useful for paving mixes to the satisfaction of engineers from different parts of the country. The actual specification that governed the production of experimental mixes in 2000 is included as Appendix A.

The first lower lift was placed in the second section of the east curve on March 21, 2000. Work proceeded in a counter-clockwise manner around the Track through the spring and into summer. The east curve was completed, followed by the north tangent, the west curve, and finally the south tangent. The last upper lift was placed on the first section of the east curve on July 14, 2000.

3.2.2.1 Preparation and Calibration

A portable double drum plant (shown in Figure 3.5) was temporarily located on-site to produce mix exclusively for Track construction with minimal haul times. Laboratory job-mix formulas were used as a starting point when each unique mix was trial run through the plant for the first time, except that actual stockpile gradations were used to make subtle adjustments to the bin percentages wherever possible. Any time a stockpile was run through the plant for the first time, the gate opening and belt speed were calibrated so the plant control system could precisely blend the bin materials at the desired percentages. Stockpile moisture contents were measured daily on any mixes that were scheduled for production to minimize the effect on plant operations and resulting final mix proportions.

Plant run trial mix was loaded into haul trucks and sampled just as if it would be placed on the surface of the Track to facilitate meaningful laboratory quality testing. Plant settings were then adjusted based upon laboratory test data and either another trial run was deemed necessary or the final plant-run job-mix formula was established. Whenever practical, trial mix was placed on Lee Road 151 (the local, previously unpaved road leading into the facility) so that sponsor representatives could weigh placement and compaction into the decision making process. Following the determination of the final job-mix formula, production of mix for placement on the Track surface was authorized.

Production and placement information describing mix placed in each lift of the top 100 mm of tangent test section buildups is provided in Appendix B.

3.2.2.2 Mixing and Loading

Construction of the actual test sections was allowed to begin after sponsors were satisfied with their trial mix results. Enough mix was produced in a continuous run to accommodate placement of both the inside and outside lanes of a single lift to minimize the amount of wasted material required to obtain stable production. Since most of the equipment was relatively cool due to the nature of the sporadic production runs, the plant was typically allowed to produce mix at slightly under the high end of the allowable temperature range (which is approximately 350°F to protect the binder from rapid aging).

End-dump trucks were used to haul mix on the majority of experimental sections; however, live-bottom trucks (also known as flow-boys or horizontal discharge trucks) were used to haul mix during the construction of sections located on curves. This difference was intended to avoid the possibility of tipping lifted beds while paving on super-elevation (which transitions to a full 15 percent within the curves). Regardless of the type of haul vehicle used, trucks were consistently loaded in a manner intended to minimize within-load segregation. This procedure involved three separate dumps in the end-dump trucks and four to five separate dumps in the flow-boys.

The on-site double barrel plant supplied by the contractor had a maximum production capacity of about 400 tons per hour; however, mix was produced for all experimental sections at a target rate of 200 tons per hour. The prescribed method of filling trucks by dumping in the front, then the back, and finally the middle portion of the bed was utilized for all loads.

Before production began in all cases, stockpile moisture contents were run in order to correct belt feed rates. Whenever practical, stockpile gradations were run before production to facilitate minor adjustments to bin percentages. Only about 2 tons of uncoated aggregates were run through the plant before liquid asphalt was added. In this manner, wasting of stockpile materials hauled long distances for inclusion in the research was minimized. Subsequently, about 15 tons of coated aggregates with varying quality was dumped directly into a haul truck and wasted in a RAP stockpile.

Before any mix was placed on the surface of the Track, at least one trial mix was run through the plant to verify the mix design under production conditions. In virtually every case, it was necessary to make at least a minor adjustment to the laboratory job-mix formula. In many cases, it was necessary to make multiple trial mix runs to either verify proposed changes or collect additional information so that an informed decision could be made. Whenever possible, trial mix was paved on the facility's county-maintained access road.

If the mix was for trial purposes and not bound for the surface of the Track, the next 20 tons went into the silo and was immediately dumped into a haul truck. The load was subjected to conventional quality control sampling and testing and the remaining material was placed on the local county road that provided access to the facility. Approximately 15 tons of coated material (assumed to have unpredictable quality) was wasted near the end of the plant run, the liquid asphalt flow was turned off, and finally the belts facilitating aggregate flow were stopped. Unused material left in the cold feed bins was removed individually and recycled to the appropriate aggregate stockpile.

If the mix had been successfully verified via trial run(s) and was intended for paving experimental sections, 45 tons was allowed to accumulate in the surge bin before the first truck was loaded out. In this manner, segregation resulting from mix flowing directly through the surge bin was avoided. When the surge bin had collected 45 tons again, the second truck was loaded out. While both of the first two trucks hauled the short distance to the Track, the surge bin was again loaded to 45 tons with the remainder of the mix required to pave the dual lane lift currently under construction. At this point, the plant was then hot-stopped (i.e., the liquid asphalt was abruptly turned off at the same time the plant was stopped with the drum full of coated material).

Both trucks were reloaded with material stored in the surge bin before the stoppage. After haul trucks had emptied their load a second and final time on the roadway, the plant was restarted and approximately 15 tons of material was passed through the surge bin and dumped in the truck as waste (without restarting the flow of liquid asphalt). The aggregate flow was turned off, and unused material left in the cold feed bins was removed individually and recycled to the appropriate aggregate stockpile.

3.2.2.3 Sampling and Testing

A sufficient quantity of both coated and uncoated material was wasted on either end of each production run so that a meaningful sample could be recovered and tested in the on-site laboratory (shown in Figure 3.6). Representative samples were recovered using conventional shovel sampling methods, an automated robotic sampling device, and an automated cold belt sweep sampler. A mechanical hot-mix sample splitting device was used in the on-site laboratory to avoid rapid cooling associated with conventional quartering and its subsequent effect on laboratory sample compaction temperatures.

Nuclear asphalt content measurements were utilized to supplement extractions via biodegradable solvent and ignition furnace. Conventional gradations with washed fines were then run on uncoated aggregate blends. Volumetric samples were prepared using an SGC for the majority of the Track, with a portable Texas gyratory being used for the single Hveem mix and a Marshall hammer used for the SMAs. Both laboratory density and roadway compaction were compared to theoretical maximum values obtained via the method described in AASHTO T 209 to compute percent compaction and air voids. Drybacks were utilized for sections containing absorptive materials.

The standard practice for conventional shovel sampling at the Track consisted of removing the top of each accessible dump in the back of the haul truck to a depth of approximately 300 millimeters. In most instances, this was accomplished by standing on the sampling platform at the plant and leaning out over the side of the truck; however, in many cases it was necessary to actually climb over into the bed of the truck to get into a position that would accommodate recovering representative material. In either case, a 19-liter metal bucket was filled by removing and combining material from near the mid-portion of each accessible mound of hot material.

Shovel sampling was initially considered the primary method of representative sample recovery; however, early experiences with obtaining unexpected laboratory results in consideration of plant adjustments necessitated a field review of alternative sampling methods. To test the suspicion that shovel samples were less representative of the entire production run than robotic samples, a simple experiment was designed in which a truckload of hot production mix was sampled numerous times. Laboratory results from a single robotic sample were compared to results from 3 different shovel samples, all taken with care in the hopes of generating representative test results.

Based upon these data (shown in Table 3.2), it was observed that the robotic sampling device produced results that reflected the change in binder content at the plant. This was not the case with shovel samples. Relying upon this limited information, a change in practice was quickly initiated in which robotic sampling became the primary method of representative sample recovery. Shovel samples would continue to be obtained in a manner that would facilitate comparisons after the completion of construction.

A standardized method for robotic sample recovery was developed to complement the methods already being utilized to load haul trucks. The probe was inserted at approximate third points in each exposed dump for loads contained within end-dump trucks. The two largest mid-load dumps (determined visually) were sampled in this same manner any time flow-boys were in use. Robotic sample depth was completely controlled by the operator; consequently, every effort was made to extract third point material from each dump at the greatest possible depth of penetration. In this manner, the objective was to remove material from the "core" of the dumped mass. Four probes typically produced two 19-liter buckets of sample material that could be taken to the laboratory, combined, split, and tested.

These experiences at the Track illustrate the difficulty in obtaining high quality representative samples from a loaded truck using a shovel. Without scaffolding to provide access to hard-to-reach areas of the bed, in many cases it was necessary to literally climb into the back of the truck to obtain samples.

The advantages of the robotic sampler in the area of safety are obvious by comparison, where samples can be retrieved from the middle of the truck while the operator stands safely on the platform completely outside the bed of hot material. Track technicians generally found the remote control panel easy to learn and operate, and adapted well to its use. The biggest problem encountered was in selecting the location to insert the probe when sampling mix from flow-boy trucks. Care had to be taken to avoid the numerous metal components that run in both directions of the bed. Flow-boy beds were eventually marked so that operators would know what areas to avoid.

3.2.3 Mix Placement

3.2.3.1 Hauling and Unloading

Two 24-ton haul trucks were loaded and driven the short distance to the location of test section placement, with the balance of the plant run being kept in the 65-ton surge bin. Paving was allowed to begin only when both trucks were lined up and ready to discharge into the MTD. Generally, the inside lane was paved first to establish a rolling pattern and was then utilized for destructive coring so that corrected nuclear gauge testing could be done non-destructively in the research (outside) lane. A picture of the typical paving scenario is provided as Figure 3.7.

3.2.3.2 Transfer and Placement

Beginning with the second section in the East curve, paving operations proceeded around the oval in a counterclockwise manner. Enough mix was produced with each plant production run to facilitate placement of both the inside and outside lanes of the lift under construction. Inside lanes were paved first so that roller breakpoints could be identified and avoided in the more critical outside (research) lane. It was found early on that with the inherently tight working area and excessive amount of equipment within the limits of the 61 meter sections, it would not be possible to pave lower and upper lifts of a section within the same workday without damaging the fresh mat; consequently, lower lifts were paved out ahead of upper lifts to enhance overall construction quality.

Pavers were preheated and raised slightly off the surface of the previously placed mat using metal spacer plates of varying thickness. When a steady flow of mix was available from the MTD, the paver pulled off the joint and began its slow but steady movement to the far end of the section. In every case, it was required that placement operations proceed in the direction of traffic (counter-clockwise). At the far end of the joint, the paver overran the distance requirement by 1 to 3 meters and lifted up the screed. This allowed the paver to be driven clear of the immediate construction zone. If the inside lane had just been completed, the paver was backed up and positioned on metal plates in anticipation of pulling the outside (research) lane. Typically, two pavers (conventional and gravity feed) were used to pave a section such that the first unit paved the inside lane and the second unit paved the outside lane.

A backhoe was then used to slice into the mound of material that had been left in place at the end of the run when the screed was lifted. This excess material was pulled back and pushed off the side of the shoulder for later cleanup and removal. With a cleanly defined fresh mat at the far end of the run, the first roller was then allowed to drive onto the uncompacted inside lane. When the roller reached the far end of the mat, it simply ramped down the overplaced mix and reversed direction. Relative increases in density were monitored in the inside lane to identify the breakpoint in the compaction operation, which was used to establish the roller pattern in the outside (research) lane. Vibratory steel-wheeled rollers were used for breakdown rolling, a pneumatic rubber-tired roller was used as necessary for intermediate rolling, and the vibratory steel-wheeled roller was used in static mode for finish rolling.

3.2.3.3 Sampling and Testing

Between paving the inside and outside lanes as the Track was being built, the MTD was advanced slightly and boomed over to accommodate dumping 2 to 3 tons of blended mix into a front-end loader. This material would be utilized for the fabrication of numerous research specimens that would later be used for laboratory performance testing, so it was important to wait until a truck in the middle of the production run had been dumped into the MTD before discharging blended material into the front-end loader. Once filled with material that was representative of the new mat, the front-end loader was driven back to the on-site laboratory where material was sampled via shovel and stored in buckets for staged heating and research sample compaction.

Concurrently, mixed material was sampled at the plant during production and taken directly to the on-site laboratory so that volumetric samples could be prepared and evaluated. Although numerous models of gyratory compactors had been installed at the on-site lab to facilitate the preparation of the large number of research specimens that would be required, a single SGC was identified before construction for use as the specification device to avoid any controversy over potential differences in compactive effort on QC specimens. The average air voids, sample height, and mass of all QC samples were documented and used to calculate target values for research specimens. Average air voids and sample mass were determined via AASHTO T 166, while sample height was measured in the SGC. When these values had been established, compaction of research specimens began using the full array of gyratory compactors that were available.

Sample mass equal to the average of the QC set was placed in each machine, which were then gyrated a varying number of revolutions until the average height of the QC set had been attained in each compacted specimen. Extra specimens were prepared to account for any that might miss the target (QC) air voids outside of the acceptable range (± 1 percent). After VTM for each specimen had been determined via AASHTO T 166, random sorting techniques were used to blindly assign samples to performance test protocols in a manner that achieved approximately equal averages in air voids and standard deviations. The overall research plan with testing protocol assignments for research specimens is provided in Table 3.3.

3.2.4 Mat Compaction

It was necessary to compact test sections in an aggressive manner due to the abbreviated nature of the short production runs. Hot-mix was dumped into a relatively cool surge bin, loaded into cool haul trucks, dumped into a cool MTD and finally boomed into a paver with a cool hopper. Even though the screed had been preheated in anticipation of paving, heat loss in the mix likely occurred more rapidly throughout the production and placement process than on a conventional project in which thousands of tonnes are run through the same relatively hot equipment in a production day.

3.2.4.1 Rolling Patterns

In compacting the typical experimental section, four coverages with the vibratory steel-wheeled roller were accomplished with twelve passes. The first pass was begun just as the paver lifted up and pulled away at the far end of the mat. Since the vast majority of experimental mixes contained modified asphalt binder, the average temperature documented behind the paver prior to initial compaction was 159°C. Compaction temperatures for all experimental pavements are provided in Appendix B. Generally, vibratory rollers were operated at high frequency and low amplitude; however, the mats were monitored closely to avoid crushing mix aggregates. The pneumatic rubber-tired roller was utilized in several instances where a tender zone was encountered in intermediate temperatures, as well as in several cases where the required level of compaction was not achieved by vibratory rolling alone. Finally, steel-wheel rollers were utilized in static mode to accomplish finish rolling, which typically consisted of four coverages via twelve passes with the mat at or just under 80°C (Powell, 2001).

Once the placement and compaction operation for both lanes had been completed, a straightedge was used to identify a distance from the far end of the mat that would most likely accommodate a smooth transition between sections. A chalk line was then popped at this distance and a masonry saw was used to cut a clean vertical face in the new mat. Lastly, a backhoe was used to pull all excess material off the shoulder for later cleanup and removal. As-built lengths for all experimental pavements resulting from this methodology are provided in Table 3.1.

3.2.4.2 Sampling and Testing

Density test locations were identified in a stratified random manner (one location within each 15 meter research increment) in both the inside and outside lanes before placement began. Nondestructive testing using both nuclear and impedance methods was conducted on locations for the inside lane, which was paved first and would not be subjected to truck traffic. After all data had been collected, cores were cut from these same locations so that a mix-specific correlation for the gauges could be computed and applied using the method described in ALDOT-350 (ALDOT, 2000). Armed with corrected nondestructive densities, the outside (traffic) lane could be tested for acceptance in the same stratified random manner without the need for cutting cores. Final density values for all experimental pavements are provided in Appendix B.

Based upon a survey of elevations between the wheelpaths before and after construction, the average thickness of the completed experimental sections was 104 millimeters, with an average thickness standard deviation within each section of 2.5 millimeters. The target thickness for sections included in this evaluation was 102 millimeters. It should be noted that two sections placed at a target thickness of 91 millimeters and two sections placed at a target thickness of 76 millimeters were not included in this analysis.

3.2.4.3 Final Smoothness

A smoothness specification (included in Appendix A) allowing for a maximum deviation of 9.5 mm was utilized to review and accept the quality of joint construction for every section on the Track. Although all joints passed their deviation tolerance using a 4.5-meter straightedge, it was later decided (based upon subjective as well as a more rigorous objective smoothness analysis) that diamond grinding should be utilized to enhance the rideability of eleven of the forty-six total construction joints.

A precision diamond grinder of the type used to plane utility cuts on racetracks was brought in to the NCAT Track on a cool day in November by the Penhall Company (a grinding subcontractor). It was thought that waiting until cooler weather would help to avoid any potential damage to the mat by the hard, heavy steel wheels that support the diamond grinding system. Penhall brought in their recirculating sedimentation tank system and completed the work in just a few hours with a single load of water. Transverse joints leading into sections N6, N8, N9, N10, N12, N13, S1, S4, S6, S7, and S9 were improved by diamond grinding. Final smoothness was verified subjectively via numerous laps in a vehicle with a stiff suspension.

3.3 ACCELERATED PAVEMENT DAMAGE

3.3.1 Trucking Organization

It was decided that to achieve the lowest trucking cost for research sponsors, a competitively bid contract would be utilized to select a trucking contractor to apply the required traffic loading. In this manner, the successful qualified bidder could potentially use their purchase power to secure the use of equipment and appurtenances at reduced cost. NCAT also chose to supply fuel that it purchased at tax-reduced rates, which would in turn accommodate the documentation of operating costs as the surface condition of the Track changed over time. Tires were also supplied by NCAT because they could be purchased at a greatly reduced cost from the government bid list.

3.3.1.1 Outsourcing

Following the completion of Track construction, a contract was competitively awarded to Covenant Transport to apply 10 million ESALs on the experimental sections over a period of approximately two-years. Although the request for bids was written to allow for various tractor-trailer combinations and robotic vehicles, trucking operations began at the Track in the fall of 2000 utilizing conventional tractors driven by live operators pulling triple trailer trains. As stipulated in contract documents, the trucking contractor was allowed to use the on-site Truck Maintenance Building (shown in Figure 3.8) without paying additional rent; however, this allowance did not include the cost of utilities.

Each axle in the low-bid configuration was loaded to approximately the federal legal bridge limit of 89 kN, which produced a gross vehicle weight for the entire triple trailer train of approximately 676 kN (shown in Figure 3.9) (Still, 2003). Generally, trucking operations ran from 5:00 AM to approximately 11:00 PM, with about an hour of down time midday to accommodate refueling and driver shift changes. Truck computers were set such that cruise speed was held constant at 72 km/hr, which is the design speed of the test oval. Overall, it is estimated that the total cost to apply traffic to the 2000 Track was approximately $2 million. This equates to an operational cost of $1.25 per mile, or $0.20 per ESAL.

3.3.1.2 Driver Safety

Safety was the number one priority in Track trucking operations and was aggressively enforced. To insure this objective, a set of safety guidelines was instituted before the first truck was allowed to operate. The drivers worked an eight to ten hour shift each day with an hour for lunch and a fifteen-minute break, but they were individually responsible for assessing their ability to drive. If they felt they might be too tired or sick to drive carefully, it was considered a major violation of the safety plan not to pull off the Track and rest.

There was also a strict protocol for entering and exiting the Track in which only one truck could be in a ramp transition at any time. If a driver determined it was necessary to leave the Track for a time to maintain their personal safety, then all other drivers were required to exit just before the departing truck planned to re-enter. Drivers were encouraged to utilize citizen band radios for personal entertainment, reserving special handheld radios for safety communications only (such as coordinating entering and exiting the Track).

Covenant Transport was required to provide an on-site mechanic at any time one or more of their tractors were in operation. This individual also served as the shift manager and maintained a handheld safety radio at all times. Any issues discovered by the drivers in daily pre-shift inspections (tires, leaks, etc.) were immediately brought to the attention of the on-site mechanic, who was then responsible for mitigation. All drivers participated in safety training sessions that included intense briefings on all issues related to Track safety, such as fuel spill protocols, fire safety, etc.

3.3.1.3 Accountability

In addition to scheduling drivers, Covenant Transport's on-site mechanic was also responsible for the maintenance of all trucking related equipment as defined by contract documents. As a matter of necessity, the mechanic's cooperation was critical in the collection of data necessary for fuel consumption, tire wear, and other research activities. It was also necessary for the mechanic(s) (the number varied between one and two) to routinely work directly with their corporate office to insure an adequate supply of drivers, supplies, etc.

All drivers were required to perform visual inspections of their truck before the start of each shift. This included fluid levels, light performance, tire condition, etc. Any identified problems were then reported to the shift mechanics so that corrective action could be taken. Planned maintenance inspections and activities were scheduled for Mondays when trucking operations were suspended to allow for performance data collection on the surface of the Track. Oil was typically changed every month, engine coolant was flushed every three months and fittings were greased as needed. These proactive measures were necessary to ensure both safety and productivity.

Perhaps the most burdensome issue for the on-site mechanic(s) was tire rotation. It was critical that all tires be rotated biweekly (with the exception of steer tires, which were rotated weekly) to reverse the irregular wear patterns that developed as a result of always turning to the left on the unidirectional test oval. Since tire supply was the responsibility of NCAT, this extremely labor-intensive rotation scheme proved to be a burden for the contractor that only worsened as tire wear studies became more common.

Although the trucking operation was ultimately the responsibility of NCAT's Test Track Manager, Covenant's on-site mechanic answered to their out-of-state corporate office. In rare cases where dispute resolution was required, it was necessary for Track personnel to submit verbal and written corporate requests that would then be passed back down to on-site personnel for corrective action.

Redundant methods used to monitor trucking progress and determine the number of laps completed. Rigs were periodically weighed with certified scales so that the number of ESALs per rig could be properly documented, which also facilitated the accurate conversion of laps completed to ESALs applied. At the end of each calendar month, the total number of ESALs applied in the previous work month were computed based on the 3 redundant methods so that payment due to the contractor could be computed. A document was prepared that summarized the increment of work completed, which was faxed to Covenant's corporate office in Chattanooga, Tennessee. Covenant prepared an invoice based upon this progress assessment, which was then sent back to NCAT for processing and payment.

3.3.2 Equipment Utilization

3.3.2.1 Tractors

As seen in Figure 3.9, the trucking contractor utilized four 2000 model FLD-120 Freightliner tractors with 60 Series 430 hp Detroit diesel engines to pull a series of three tandem trailers each. Because of the tremendous weight and stress on the tractors, the frames of each had to be double reinforced and equipped with a high torque drive train that could handle the extreme load. In addition, Track rigs were equipped with a radar-based collision avoidance system and an infrared vehicle identification system.

The collision avoidance system consisted of sensors installed on the front and sides of the tractor. Its purpose was to monitor the proximity of road hazards to the vehicle. Any time the rig passed within a preset limit to a road obstacle a series of progressive steps were automatically taken to avoid a potential accident. Initially, an alarm was sounded within the cab of the truck to alert the driver, which was immediately followed by disengagement of the cruise control. If the system still anticipated a possible collision, more aggressive steps could be taken (such as down shifting gears, killing the engine, etc.).

The vehicle identification system was used to automatically log laps made by each individual truck as it navigated the Track. A unique signature frequency infrared emitter was attached to the cab of each truck to serve as the identification mechanism, which was monitored by a frequency detector mounted at a fixed location on the side of the Track. As each truck passed, an encoded signal was transmitted via hard wiring to the Track laboratory. A datalogging computer then recorded a line of code to document each unique event with a truck number, date stamp and time stamp. In this manner, accurate weights measured for each axle (before traffic began and verified intermittently throughout) could be used to compute accumulated ESALs applied via the limited load spectra.

In addition to simple pneumatic counters (mounted in the East curve and North tangent), drivers' logs were also utilized as a backup to the automated systems. Each driver was required to fill out a log sheet for every shift run, which contained the date, driver's name, tractor number, trailer set number, beginning and ending mileages, number of stops made, and total gallons of fuel required to top off the tank at the end of the shift. Algorithms were developed for each individual truck that used total miles driven and number of times off the Track to estimate the number of laps driven on the shift.

A database was designed to house all of this information and automatically calculate both ESAL numbers and fuel mileage. The design of the database was such that trucks and trailer assemblies accumulated separately, which meant that both tractors and trailer dollies could be switched without sacrificing accuracy in the ESAL count. The complete progress plot of ESAL accumulation versus time is provided as Figure 3.10, where it is seen that after the initial break in period traffic was applied in a more or less linear manner.

3.3.2.2 Trailers

As a consequence of their membership in the Track research sponsor group, FHWA allowed NCAT the use of the Knight brand trailers that were built to apply accelerated loading to experimental pavements on their completed Westrack project previously operated at the Nevada Automotive Test Center. Although four sets of trailers were originally prepared for that project, only three complete sets survived and were serviceable. Covenant Transport shipped these units cross-country after being awarded the NCAT contract and prepared them for the two-year operation. These units were simple flatbed designs atop spring suspension systems.

Since it likely would not have been possible to complete the work with just the three FHWA trailer sets, Covenant Transport supplied a fourth trailer set (flatbeds atop air suspensions) that would become the property of NCAT at the conclusion of testing.

Axle fatigue issues plagued the air suspension trailer set until load distribution was ultimately optimized by moving the stacked plates inward.

3.3.3 Trucking Appurtenances

3.3.3.1 Fuel

Before the contract was let, it was decided that NCAT would furnish fuel for all the trucks. As a government agency, Auburn University could purchase off road fuel tax-free. In contrast, buying fuel through the trucking contractor would have necessitated their passing on all market costs to the University (in addition to their overhead and profit).

A fuel vendor was competitively selected who was required to install a portable tank on a fueling apron constructed at the edge of the off ramp to accommodate refueling after the end of each shift. A tank rental fee was applied as a surcharge to each gallon of fuel supplied, where the trucking mechanic notified the vendor when the tank level (as read by a gauge clearly visible from ground level) had been lowered to a point where an entire tanker truck could be discharged (thus, optimizing delivery for the vendor). This arrangement also gave NCAT the ability to closely monitor fuel consumption and how it might vary as the surface condition of the Track changed with traffic over time.

3.3.3.2 Tires

Tire wear was a major consideration in the trucking operation. The first set of steer tires lasted only 5,000 miles, which did not compare well with conventional long haul expectations. An expert in truck alignment was subsequently brought in to identify methods to extend tire life. The consultant's recommendations resulted in numerous improvements in standard practices at the Track. For example, an aggressive tire rotation plan was implemented to resolve the irregular wear patterns induced by perpetual left turns. Additionally, the fifth-wheel plate was shifted forward to put more weight on the steer axles and less on the drives. Thereafter, each truck was monitored weekly for trailer alignment problems and corrected as soon as possible.

Because of this concerted effort, the life of the steer tires were increased from 5,000 miles to 50,000 miles even though each truck hauled approximately twice the legal gross vehicle weight (since there are no bridges on the Track, gross vehicle weight is not an issue). The life of the trailer and drive tires were also extended to 90,000 and 70,000+ miles, respectively.

3.4 TIMELINE

The ground was broken on the site that would become the NCAT Pavement Test Track on September 15, 1998. Completion of the grading and drainage portion of the project was completed by June 15, 1999. On-site laboratory and truck maintenance buildings were erected between January 27, 1999 and January 6, 2000. The Track's thick perpetual foundation was complete up to the bottom of experimental surface mixes by March of 2000, and research mix was placed in the last experimental section on July 14, 2000. Limited truck traffic began on the new pavements on September 19, 2000, with authorization for full fleet operations granted on November 19, 2000. Traffic operations were completed on the 2000 Track at approximately 2:00 PM on December 17, 2002.

TABLE 3.1 Consensus Experiment Placed on 2000 Track

Track Quad	Section Num	Aggregate Blend Type	Design Method	Design NMA	Grad Type	Binder Grade	Binder Modifier	Approx Length	Lift Type	Design Thick
E	**2**	Granite	Super	12.5	BRZ	67-22	NA	213	Dual	4.0
E	**3**	Granite	Super	12.5	BRZ	76-22	SBR	189	Dual	4.0
E	**4**	Granite	Super	12.5	BRZ	76-22	SBS	204	Dual	4.0
E	**5**	Granite	Super	12.5	TRZ	76-22	SBS	201	Dual	4.0
E	**6**	Granite	Super	12.5	TRZ	67-22	NA	211	Dual	4.0
E	**7**	Granite	Super	12.5	TRZ	76-22	SBR	193	Dual	4.0
E	**8**	Granite	Super	12.5	ARZ	67-22	NA	208	Dual	4.0
E	**9**	Granite	Super	12.5	ARZ	76-22	SBS	198	Dual	4.0
E	**10**	Granite	Super	12.5	ARZ	76-22	SBR	99	Dual	4.0
N	**1**	Slag/Lms	Super	12.5	ARZ	76-22	SBS	201	Dual	4.0
N	**2**	Slag/Lms	Super	12.5	ARZ	76-22+	SBS	200	Dual	4.0
N	**3**	Slag/Lms	Super	12.5	ARZ	67-22+	NA	200	Dual	4.0
N	**4**	Slag/Lms	Super	12.5	ARZ	67-22	NA	199	Dual	4.0
N	**5**	Slag/Lms	Super	12.5	BRZ	67-22+	NA	201	Dual	4.0
N	**6**	Slag/Lms	Super	12.5	BRZ	67-22	NA	197	Dual	4.0
N	**7**	Slag/Lms	Super	12.5	BRZ	76-22+	SBR	203	Dual	4.0
N	**8**	Slag/Lms	Super	12.5	BRZ	76-22	SBR	203	Dual	4.0
N	**9**	Slag/Lms	Super	12.5	BRZ	76-22	SBS	197	Dual	4.0
N	**10**	Slag/Lms	Super	12.5	BRZ	76-22+	SBS	206	Dual	4.0
N	**11**	Granite	Super	19.0	BRZ	67-22	NA	195	Lower	2.5
		Granite	Super	12.5	TRZ	76-22	SBS	195	Upper	1.5
N	**12**	Granite	Super	19.0	BRZ	67-22	NA	201	Lower	2.5
		Granite	SMA	12.5	SMA	76-22	SBS	201	Upper	1.5
N	**13**	Gravel	Super	19.0	BRZ	76-22	SBS	199	Lower	2.5
		Gravel	SMA	12.5	SMA	76-22	SBS	199	Upper	1.5
W	**1**	Granite	SMA	12.5	SMA	76-22	SBR	202	Dual	4.0
W	**2**	Slag/Lms	SMA	12.5	SMA	76-22	SBR	200	Dual	4.0
W	**3**	Granite	Super	12.5	BRZ	76-22	SBR	205	Lower	3.3
		Slag/Lms	OGFC	12.5	OGFC	76-22	SBR	205	Upper	0.7
W	**4**	Limestone	SMA	12.5	SMA	76-22	SBR	199	Lower	3.3
		Granite	OGFC	12.5	OGFC	76-22	SBR	199	Upper	0.7
W	**5**	Limestone	SMA	12.5	SMA	76-22	SBS	203	Lower	3.3
		Granite	OGFC	12.5	OGFC	76-22	SBS	203	Upper	0.7
W	**6**	Slag/Lms	Super	12.5	TRZ	67-22	NA	203	Dual	4.0
W	**7**	Limestone	SMA	12.5	SMA	76-22	SBR	207	Dual	4.0
W	**8**	Sandstn/Slg/Lms	SMA	12.5	SMA	76-22	SBR	197	Dual	4.0
W	**9**	Gravel	Super	12.5	BRZ	67-22	NA	203	Dual	4.0
W	**10**	Gravel	Super	12.5	BRZ	76-22	SBR	102	Dual	4.0
S	**1**	Granite	Super	19.0	BRZ	76-22	SBS	200	Lower	2.5
		Granite	Super	12.5	BRZ	76-22	SBS	200	Upper	1.5
S	**2**	Gravel	Super	19.0	BRZ	76-22	SBS	200	Lower	2.5
		Gravel	Super	9.5	BRZ	76-22	SBS	200	Upper	1.5
S	**3**	Limestone	Super	19.0	BRZ	76-22	SBS	201	Lower	2.5
		Lms/Gravel	Super	9.5	BRZ	76-22	SBS	201	Upper	1.5
S	**4**	Lms/RAP	Super	19.0	ARZ	76-22	SBS	198	Lower	2.5
		Limestone	Super	12.5	ARZ	76-22	SBS	198	Upper	1.5
S	**5**	Lms/Grv/RAP	Super	19.0	BRZ	76-22	SBS	203	Lower	2.5
		Gravel	Super	12.5	TRZ	76-22	SBS	203	Upper	1.5
S	**6**	Lms/RAP	Super	12.5	ARZ	67-22	NA	198	Dual	4.0
S	**7**	Lms/RAP	Super	12.5	BRZ	67-22	NA	202	Dual	4.0
S	**8**	Marble-Schist	Super	19.0	BRZ	67-22	NA	197	Lower	2.1
		Marble-Schist	Super	12.5	BRZ	76-22	SBS	197	Upper	1.5
S	**9**	Granite	Super	12.5	BRZ	67-22	NA	206	Dual	3.0
S	**10**	Granite	Super	12.5	ARZ	67-22	NA	195	Dual	3.0
S	**11**	Marble-Schist	Super	19.0	BRZ	67-22	NA	202	Lower	2.1
		Marble-Schist	Super	9.5	BRZ	76-22	SBS	202	Upper	1.5
S	**12**	Limestone	Hveem	12.5	TRZ	70-28	SB	199	Dual	4.0
S	**13**	Granite	Super	12.5	ARZ	70-28	SB	201	Dual	4.0
E	**1**	Gravel	Super	12.5	ARZ	67-22	NA	199	Dual	4.0

Notes: - Mixes are listed chronologically in order of completion dates (which are presented in Appendix A).
- "dual" lift type indicates that upper and lower lifts were constructed with the same experimental mix.
- ARZ, TRZ, and BRZ refer to gradations above, through, and below the restricted zone, respectively.
- SMA and OGFC refer to stone matrix asphalt and open-graded friction course mixes, respectively.

TABLE 3.2 Early Experiences Comparing Track Sampling Methods

Mix Quad	**Mix Sec**	**Sublot Num**	**Plant AC Setting**	**Shovel AC Measured**	**Robot AC Measured**
E	2	1	5.2	5.06	5.38
E	2	2	5.2	5.04	5.34
E	3	1	5.0	5.39	5.14
E	3	2	5.0	5.11	4.87
		Expected results in laboratory?		No	Yes

Notes: Data from the first two research mix sublot placements
Sublots 1 and 2 refer to inside & outside bottom lane lifts
Measurements via AASHTO T 287 (nuclear method)

TABLE 3.3 Overall Research Plan with Testing Protocol Assignments for Research Specimens

Phase	Mechanism	Objective and Scope
Construction	Test Oval	26 Unique Experimental Tangent Sections (33 Unique Mixes)
	Hot Plant Run Material	SGC Specimens x **18 Research Pills** (See Use Below)
	Hot Plant Run Material	GTM Using Oil Cell Roller x 1 Supplemental Pill
	Reheated Plant Run Material	GTM Using Air Cell Roller x 1 Supplemental Pill
	Testing During Gyratory Compaction	Gyratory Shear in Both SGC (x 3) and GTM (x 1)
Traffic Application	Multi-Depth Thermistors	Mat Temperature Measured with Each Axle Pass
Field Testing	Weekly Transverse Profiling	Change in Rut Depth Resulting from Week of Traffic
Laboratory Testing	Liquid Binder Performance	PG Method for Upper Binder Failure Temperature
	Simulative Mix Performance @ 64°C	Asphalt Pavement Analyzer x **3 Research Pills**
		Hamburg Loaded Wheel Testing x **3 Research Pills**
		Rotary Loaded Wheel Testing x **3 Research Pills**
		Wessex Wheel Testing x **3 Research Pills**
	Findamental Mix Performance @ 64°C	Dynamic Modulus x 3 Research Pills (Prior to Triaxial)
		Seismic Testing x 3 QC Pills and Mat (Mid-Lane & Wheelpath)
		Triaxial Testing (Repeated Load Confined Cyclic) x **3 Research Pills**
		Superpave Shear Testing x **3 Research Pills** (Sliced)

FIGURE 3.1 Oversight from Research Sponsors.

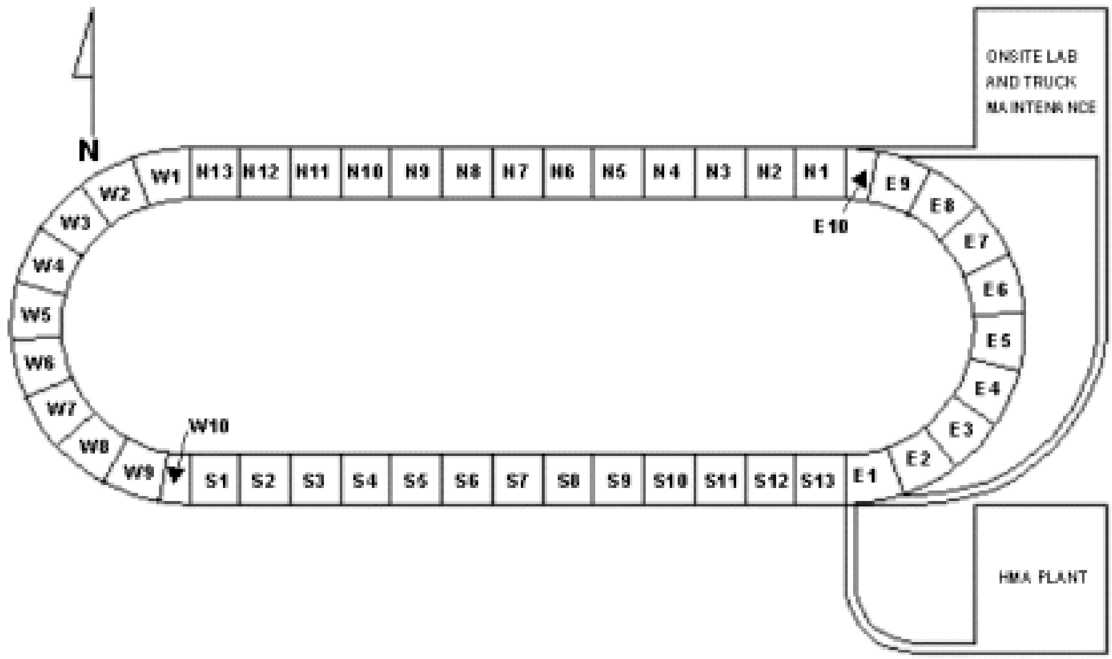

FIGURE 3.2 Schematic of Experimental Section Layout.

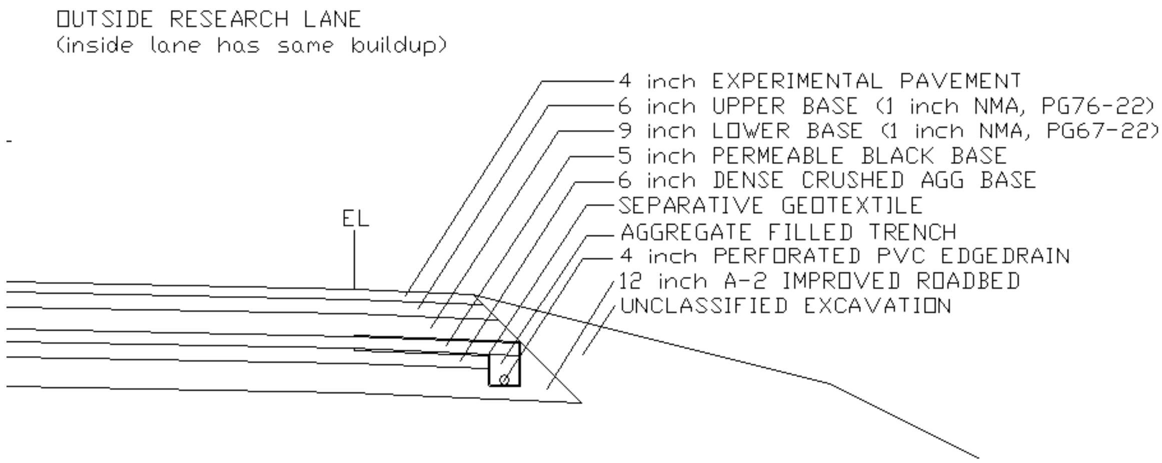

FIGURE 3.3 Track Buildup Cross Section.

FIGURE 3.4 Management of 69 Unique Imported Stockpiles.

FIGURE 3.5 Production and Sampling of Hot-Mix at Track's On-site Plant.

FIGURE 3.6 Testing Materials in Track's On-site Laboratory (Track in Near Background and Plant in Far Background).

FIGURE 3.7 Placement and Compaction of Experimental Mix in East Curve.

FIGURE 3.8 Track's On-site Truck Maintenance Building (Testing Laboratory in Background).

FIGURE 3.9 Triple Trailer Rig Used to Apply Accelerated Loading.

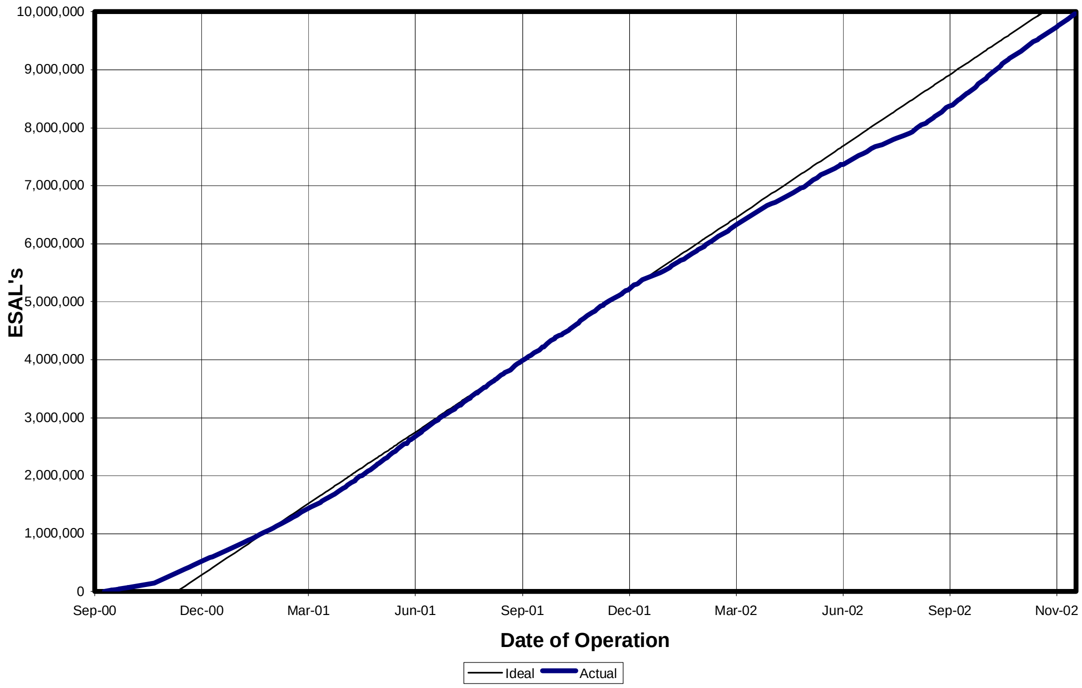

FIGURE 3.10 Record of Track ESAL Applications Over Time.

CHAPTER 4 – PERFORMANCE TEST RESULTS

4.1 FIELD PERFORMANCE

Every Monday, trucking was suspended so that surface condition studies could be conducted to thoroughly document field performance of all experimental test sections over time. The focus of the research was rutting comparisons between sections and field-to-laboratory rutting correlations; however, roughness, texture, and densities were also measured for utilization in other Track-based research efforts (West, 2004).

4.1.1 Rutting Performance Measurements

Because most field performance testing was limited to Mondays when the fleet was idle, it was necessary to measure rutting with several different methods that could each be compared to final values in order to identify which method produced the least amount of error on a weekly basis. Field performance evaluations focused on the middle 45 meters of each 60-meter test section, which eliminated the effect of transitional quality on either end. The middle 45-meter research portion of each test section was then broken into three 15-meter measurement replicates, each containing a randomly located transverse profile across which elevations were measured on a weekly basis.

4.1.1.1 Rutting via Three-Point Approximations

While transverse profiles were characterized weekly using contact measurements, continuous rutting data were also collected with the University's high-speed laser profiler (shown in Figure 4.1). The high-speed approach characterized each section with a single number that represented the average rut depth of the entire 45 m research length. It was important to include the high-speed inertial profiler in Track research because it is used by many DOTs to conduct network level surveys. The 3-laser approach in the high-speed system reports a single rut depth as the average difference in vertical elevation between the wheelpaths and the mid-lane pavement surface. Because this method is not capable of considering rutting geometry outside the wheelpaths, it was expected that transverse profiles would be needed to serve as the primary source of information as shear outside the wheelpaths became more influential. Figure 4.2 is provided to illustrate the basic geometry of different types of rut depth measurements.

4.1.1.2 Rutting via Precision Level Profiles

A precision level (known commercially as a "Dipstick") was walked across each of three stratified random locations within each section (illustrated in Figure 4.3) on a weekly basis. Known elevations on either end of the profile could then be used to close each traverse, producing elevations that could be used to compute both left and right wheelpath rutting in an accurate manner. Viewed individually, these data only provide a snapshot representation of rutting performance; however, when profile-based rutting is sorted by date and viewed as a moving picture, the data reveal how each section performed on a weekly (continuous) basis.

Because this method does not produce continuous data across the entire profile, it is only possible to estimate the actual rut depth that would be computed from a complete record of surface elevation measurements. Two different algorithms can be used to estimate rut depth from weekly surface elevation measurements using geometries that either include (i.e., 6-point) or ignore the contribution of elevations at the outer edges of both wheelpaths (i.e., 3-point). The 6-point algorithm is standardized in AASHTO PP 38-00 ("Determining Maximum Rut Depth in Asphalt Pavements") to include the effect of heave on the outside of each wheelpath, while the 3-point algorithm is similar to the method used with the high-speed laser approach (i.e., average difference in vertical elevation between the wheelpaths and the mid-lane pavement surface). In the 6-point approach, the 3 elevations that define the peaks and valley of the left wheelpath are used to compute a maximum trough depth for the left wheelpath. The same procedure is used to produce a maximum trough depth for the right wheelpath, which is then used to compute a single average rut depth for the entire transverse profile (AASHTO, 2000).

Sample elevation profiles that provided data for this process are presented in Table 4.1. In this Table, points P1 and P3 represent the transverse distances to the geometric "peak" elevations on the outer edges of the left wheelpath, while point P2 represents the transverse distance to the deepest "valley" elevation near the middle of the left wheelpath. Similarly, points P4 and P6 represent the transverse distances to the geometric "peak" elevations on the outer edges of the right wheelpath, while point P5 represents the transverse distance to the deepest "valley" elevation near the middle of the right wheelpath.

In order to quantify the precision of individual rut depths computed from dipstick profiles, a repeatability experiment was run on January 21st of 2002. Thirty independent profiles were measured using the same equipment and the same operator across the second random test location in section N3. Rut depths for the inside and outside wheelpaths were calculated for each profile using the 6-point methodology described above. A statistical analysis of the resulting data revealed a standard deviation in each individual wheelpath of 0.1 mm.

4.1.1.3 Rutting via Wire Line

After traffic operations had been completed (i.e., all 10 million ESALs had been applied), it was possible to invest the time necessary to measure rut depths using more traditional methods. A taunt wire was stretched onto the surface of the pavement and a laboratory depth gauge was utilized to measure the depth of the ruts in both the left and right wheelpaths at the point of maximum deformation (a process illustrated in Figure 4.4). Although deformation could be measured with the depth gauge to the nearest 0.01 mm, no repetitive measurements were made to determine the precision of the method. Final wire line values for the 2000 Track are presented in Table 4.2, in which it is seen that average rut depths for tangent test sections ranged from a low of 0.46 mm in section S2 to a high of 7.27 mm in section N3.

Left and right wheelpath rut depths shown in Table 4.2 are average values based on 9 transverse profile locations per section. Thus, 18 rut depths were recorded for each section (9 in the left wheelpath and 9 in the right wheelpath) after the application of 10 million ESALs. Three of the 9 measurement locations are the same test locations utilized for weekly testing, while the remaining 6 locations were spaced evenly over the remaining longitudinal length of each test section.

It is assumed that wire line measurements represent the actual rut depths that should serve as the basis for all model predictions; however, it was only possible to measure wire line values after traffic operations had been completed. Consequently, it was necessary to compare final wire line measurements to final transverse profile-based values in order to create the most accurate accounting of rutting over time using only weekly transverse profiles. In future Track research cycles, electronic means may be available to measure continuous transverse profiles via contact methods and accommodate accurate virtual wire line analyses on regular intervals (e.g., quarterly).

Final values for wire line, 3-point and 6-point rutting measurements are provided in Table 4.3, which reveals that the 3-point method produces rut depth estimates generally much larger than wire line values. Figure 4.2 illustrates that inaccuracies in the 3-point method can be the result of necessarily ignoring heave on the outside edges of the wheelpaths. Using wire line data as the standard of comparison (i.e., the “correct” rut depth), the superiority of the 6-point algorithm in comparison to the 3-point method is clearly demonstrated in Figure 4.5. As a result of this observation, it was determined that the 6-point method would be utilized to quantify weekly rutting performance throughout the two-year traffic cycle.

Final wire line values were then used to calibrate the entire record of weekly 6-point (profile-based) rut depths by computing the ratio of final wire-based to final profile-based values for each experimental section. This effort generated a constant multiplier for each test section that was then used to correct 6-point rut depths measured at all times, theoretically making them equal to wire line measurements.

Next, the total change in rut depth that had occurred at the time each weekly measurement was made (induced rutting) was then computed for each section. As seen in Table 4.4, induced rutting values can differ slightly from measured rutting values by the amount of apparent “rutting” that existed before any traffic was applied. Slight surface irregularities (e.g., roller marks) can actually produce negative geometric “rutting” at zero traffic, which leads to a total traffic-induced rutting that is more (or less) than the measured wire line value.

For example, a section could be built with a profound roller mark that created a ridge in the wheelpath. Transverse profiling before traffic began could have characterized this section with a negative value for rut depth. If traffic-induced deformation in the wheelpath equaled the height of the ridge, the final wire line value would be zero. In order to account for this effect and eliminate the bias it would induce on both field comparisons and laboratory predictions, it is necessary to use the original negative rut as the starting point for section deformation instead of zero. For this reason, a section with final wire line rutting of 4.1 mm had a total induced rutting of 4.5 mm (section S10).

To support the modeling effort, the weekly change in rutting was visually best fit to produce a single, smooth progression of induced rutting over the life of the two-year project. It was necessary to smooth the data in this manner in order to prevent apparent negative changes in induced rutting (resulting from experimental error) from influencing the modeling effort in an illogical way. Individual data points that served as the basis of the smoothing process are shown in the figures referenced in coming paragraphs for the reader's consideration. Neglecting to smooth the data would result in apparent improvements in induced rutting after a week of truck traffic (e.g., where little or no actual change in rut depth was accompanied by random negative error in the transverse profile). The final product of this effort, shown in Table 4.5, was a record of incremental rutting for each measurement interval. Any apparent rut depths that existed prior to the application of traffic were not included in Table 4.5. Also, reporting dates for Table 4.5 were selected to represent points in time between which rutting progressed at a more or less constant rate.

4.1.2 Other Performance Measurements

The same 3-laser high-speed profiler that estimated 3-point rut depths was also equipped with an inertial compensation system to normalize vehicle dynamics and produce profile-based roughness measurements for each section. Output data were summarized in 8 m increments such that the middle 46 m of each section could be examined without the influence of adjacent transverse joints (affecting approximately 8 m on either end of each section). Roughness is reported in m/km in accordance with the International Roughness Index (IRI) approach, which is a mathematical assessment of amplified profile wavelengths tuned specifically to quantify the feel of the roadway for a traveler in a passenger car (thus, “long” wavelengths of between 50 mm and 60 m) (Cenek, 2000).

Additionally, the laser mounted in the right wheelpath sampled data at a relatively high frequency (64 kHz) (Roadware, 1999). This allows the onboard software to quantify the macrotexture of each experimental pavement by digitizing the rapid vertical distance measurements (read at an accelerated rate of three samples per mm at Track test speeds). Macrotexture is a term used to define short (0.5 to 50 mm) wavelength irregularities in the surface of a pavement, and is a function of the gradation of the aggregates in the mix, void structure, etc. Wavelengths shorter than 0.5 mm are thought to represent the surface texture of the aggregate itself, which is referred to as microtexture. Wavelengths greater than 50 mm but less than 60 m are not indicative of texture but contribute to roadway roughness felt by the motoring public (Cenek, 2000). The significance of various wavelengths is illustrated in Figure 4.6.

Cores were cut every three months from the last 7.5 m of each section so that densification of individual lifts could be quantified. Each time coring was conducted, both nuclear and non-nuclear density testing was performed on the spot where the core would be taken to also facilitate density gauge correction. Subsequent testing with both nuclear and non-nuclear devices could then be conducted within the section in the wheelpaths at stratified random locations. In this manner, densification within the research portion of each section could be monitored on a weekly basis in a nondestructive manner.

Although many changing pavement properties can be either directly or indirectly related to changes in rut depth, no further discussion of roughness, macrotexture or densification is offered herein because it is beyond the scope of this research effort.

4.1.3 Environmental Measurements

Each section on the NCAT Track had to perform in an identical environment, and it was important to accurately document relevant conditions in support of performance models. An automated weather station was installed between the on-site laboratory and the North tangent to serve this purpose. As a bid item in the construction contract, a computerized Campbell Scientific CR10X unit that logged air temperature, rainfall, humidity, wind speed, wind direction, and solar radiation on an hourly basis was hardwired into the on-site laboratory's data acquisition room.

In anticipation that pavements with different textures containing different types of aggregate with varying asphalt contents would absorb and reflect heat at different rates, each individual section on the Track was outfitted with multi-depth temperature instrumentation (shown schematically in Figure 4.7) that also relied upon CR10X datalogging computers (mounted along the perimeter of the Track) to maintain a record of multi-depth temperature versus time for each section. Temperatures were measured via precision thermisters installed during construction at depths of 0, 50, 100, and 250 mm that were logged hourly on the CR10X computers. Roofing shingles were used to protect exposed temperature sensor cables near the pavement's edge, and performed well through the end of the project.

No power was provided around the Track's perimeter, so solar panels were used to trickle charge DC batteries mounted next to each CR10X in an environmental enclosure. Each of twenty-three total CR10X enclosures was mounted on the back of every other section sign around the Track's perimeter such that every other section shared a datalogger with the adjacent section. An asynchronous communication network that ran completely around the Track allowed the field dataloggers to share data with desktop computers in the data acquisition laboratory each hour.

Each Track datalogger also collected information provided by a time domain reflectometry (TDR) subgrade moisture gauge that was installed between adjacent experimental surface mixes. This additional step was taken in order to differentiate the effect of potentially varying subgrade moisture contents on surface mix performance; however, it was learned over the course of the study that subgrade moisture contents (shown in Figure 4.8) did not vary by more than a couple of percent moisture regardless of whether sections were built in cut or fill areas (Brown et al., 2002). Although this effort served to document the small relative differences in moisture contents between sections, a later investigation (in sections built after the conclusion of this study) indicated that TDR measurements were approximately double actual moisture contents. Extensive erosion at edgedrain outlets (installed every 150 meters) was indicative of water being transported along the side of the pavement structure, but it was not possible to determine whether the water was being removed from the pavement or simply infiltrating the drainage trench (shown in Figure 3.3) from soil along the shoulder. No further discussion of potential TDR error is offered herein because it is beyond the scope of this research effort; however, the reason for the discrepancy is under investigation.

All environmental information for each performance hour was automatically summarized on three datalogging computers located in the on-site laboratory. After the project had been completed, the environmental records were reconciled with the historical traffic record into a single database that was used to create the load-temperature spectra record upon which the modeling efforts described herein were developed. Additionally, a comprehensive database was created that will be used in future research efforts. The overall instrumentation program on the 2000 Track was a collaborative effort between McEwen Instrumentation Consultants and the Corps of Engineers' Waterways Experiment Station (WES), who summarized the installation and calibration of each system in a comprehensive report (Freeman et al., 2001).

4.1.4 Field Experiments

The first phase of testing on the NCAT Pavement Test Track was intended to serve primarily as a rutting experiment. As planned, rutting was the most notable surface distress observed within the research portion of all sections at the completion of truck traffic; however, most rut depths were relatively minor.

Other minor distresses, such as shallow surface raveling, fat spots near joints, etc., were noted in several sections. Many centerline pavement markers were crushed by the wandering heavy truck traffic. Table 4.6 is included to provide an overview of changes in surface properties induced by the application of 10 million ESALs. Here it is seen that fifteen of twenty-six sections exhibited a reduction in IRI up to 0.25 m/km, while MTD was observed to decrease in ten of twenty-six sections. Wet ribbed surface friction only increased in section S13; otherwise, surface friction decreased in every section. Longitudinal cracking along the centerline joint appeared in February of 2002 in 3 sections (N8, S2 and S13) after an uncharacteristically harsh winter storm in early January.

Roughness decreased from September of 2000 to February of 2001, but drifted higher at a very mild rate between that time and the end of the project. No references supporting the phenomenon of temporary post-construction smoothness improvement were found in the literature, probably because no one measures roughness with great frequency in the time period immediately after construction. This effect is likely the result of kneading of the mat with rubber tired truck traffic, and was also observed on later (2003) Track test sections.

Based upon final wire line measurements shown in Table 4.2, rutting ranged from a low of 0.5 mm to a high of 7.3 mm. Analyses of weekly transverse profiles revealed that rutting developed as some function of seasonal variation in temperature. In hotter months, the slope of the average rutting curve steepened. In cooler months, the slope of the rutting curve flattened. Seasonal trends are easily seen in Figures 4.9 through 4.17 and 4.19 through 4.25, which illustrate rutting versus ESAL results for section comparisons on both Track tangents over two complete cycles of seasons.

In anticipation of this effect, thermistors embedded in each section provided a continuous record of temperature versus depth for every experimental mix. Resulting multi-depth pavement temperatures were blended with the traffic history into a comprehensive performance database to objectively quantify the relationship between rutting and the environment for each section. Load-temperature spectra performance modeling will be discussed in detail in Chapter 5.

4.1.4.1 Sponsor Field Comparisons

The test section layout for the 2000 Track, shown in Table 4.4, allowed for the direct comparison of tangent section performance in seventeen cases. In each of these comparisons, which are shown graphically in Figures 4.9 through 4.17 and 4.19 through 4.25, the Tukey-Kramer statistical approach was utilized to determine if apparent differences in final rut depths had statistical meaning. The Tukey-Kramer method is applied by calculating confidence intervals between the average rut depths reported for each section. If the confidence interval does not contain zero, the two means are significantly different. If the confidence interval contains zero, it has not been statistically proven that a difference exists between the means. The Tukey-Kramer method was chosen because it utilizes the MS_E to consider the variability of all groups in the dataset, whereas t-testing only utilizes the variance of the two groups being tested.

For these analyses, rutting in each experimental section was measured at nine locations per section. Three of the 9 test locations were previously identified using stratified random methods, with the other 6 locations spaced evenly throughout the middle 45 m (research) portion of each test section. At each location, a taunt wire line was laid across the entire lane in the transverse direction and the average depth of the wheelpath troughs was recorded. The results from this process are included in Table 4.2. An analysis of variance for rutting measurements in all 26 tangent test sections was conducted on these data using the Minitab computer program (included as Table 4.7). Confidence intervals were then computed in the following manner:

$$\textit{Confidence Interval} = \textit{Mean Difference} \pm \left(t_{0.025;208} \cdot \sqrt{\frac{MS_E}{n}} \right) \qquad \text{Equation 4.1}$$

The studentized statistic $t_{0.025;208}$ is determined using $\alpha = 0.05$ and 208 degrees of freedom for error (from twenty-six sections with $n = 9$ observations per section). In this case, the studentized percentage is approximately 5.02. It is seen in Table 4.7 that the within treatment mean square (MS_E) was 0.279, with a between treatment mean square (MS_T) of 31.038. Since the MS_E was much lower than the MS_T, it was likely that treatment means would be found to be unequal. All terms in Equation 4.1 were identified and it was possible to compute a 95 percent Tukey-Kramer confidence interval to scrutinize each section comparison. Table 4.8 is provided as a summary of this process for all comparisons, with discussion of specific section comparisons provided in the following paragraphs.

The first ten sections of the north tangent (section numbers N1 through N10) were jointly sponsored by ALDOT, FHWA and the Indiana Department of Transportation

(INDOT). Research parameters for these sections are described in Table 3.1, but in summary were intended to determine the effect of binder grade, binder modifier, binder content and gradation type on the rutting performance of HMA mixes containing one aggregate source (a mixture of slag and limestone aggregates). All five mix designs used to build sections N1 through N10 were performed by NCAT personnel.

Each mix produced at optimum asphalt content in these sections (N1, N4, N6, N8 and N9) had a comparison section that was produced at an asphalt content that targeted optimum plus ½ percent (N2, N3, N5, N7 and N10). The first four sections (N1 through N4) contained gradations that generally passed above the maximum density line (ARZ), and the last 6 sections (N5 through N10) contained gradations that generally passed below the maximum density line (BRZ). Each ARZ gradation was produced with (N1 and N2) and without (N3 and N4) SBS binder modifier, and each BRZ gradation was produced without modifier (N5 and N6) and with both SBS (N9 and N10) and SBR (N6 and N7) binder modifier. The specified grade for all unmodified binders in these sections was PG64-22, while the specified grade for all modified binders was PG76-22.

Rutting performance curves for these sections are provided in Figures 4.9 through 4.16. No statistically significant difference in performance was noted between sections containing modified asphalt that were produced at optimum asphalt content (ARZ section N1 and BRZ sections N8 and N9) compared with sections containing modified asphalt that were produced with ½ percent additional asphalt content (ARZ section N2 and BRZ sections N7 and N10).

This was not the case with sections produced with unmodified asphalt. Forty-five percent less rutting (a difference of 3.3 mm) was observed in the optimum BRZ section N6 compared to section N5 with ½ percent additional asphalt content. Twenty-seven percent less rutting (a difference of 2.0 mm) was observed in the optimum ARZ section N4 compared to section N3 with ½ percent additional asphalt content.

Additionally, the BRZ mixes all experienced less rutting (with statistical significance) than their ARZ comparison sections. Sixty-three percent less rutting (a difference of 1.6 mm) was observed in the SBS-modified BRZ section N9 compared with SBS-modified ARZ section N1. Nineteen percent less rutting (a difference of 1.5 mm) was observed in the unmodified BRZ section N6 compared with unmodified ARZ section N4. Although ARZ sections exhibited less rutting than BRZ sections, both mix types performed well. Deterioration of the N5 joint (leading into the unmodified BRZ section) was noted at the conclusion of traffic. No statistical difference in performance was noted between SBS- and SBR-modified BRZ sections produced at optimum asphalt contents (sections N9 and N8, respectively), but there was no opportunity to evaluate this relationship for ARZ mixes.

The 11th and 12th sections on the north tangent were sponsored by the Georgia Department of Transportation (GDOT). In their study, a dense-graded granite surface mixture was compared to a SMA granite surface mixture with both sections underlain by the same dense-graded granite binder mix. The purpose of the study was to test Georgia's decision to favor more expensive SMA placement on high volume roadways. As seen in Figure 4.17, no statistical difference was observed in rutting performance between these two sections; however, the section topped with dense-graded mix (N11) exhibited changes in surface appearance that are typically associated with lower durability. The transverse joint leading into section N11 was raveling noticeably by the end of the trucking operation. The centerline joint had begun to crack, and a change in surface appearance was observed in section N11.

A plot of mean texture depth over time for sections N11 and N12 is provided as Figure 4.18, in which the Superpave section (N11) was observed to drift higher at a relatively constant rate of 0.02 mm per million ESALs over the course of the entire traffic cycle. Mean texture depth increases when aggregate particles are dislodged from the mat, leaving exposed surface voids in their place. This cumulative process creates a condition commonly referred to as raveling. In comparison, after an initial period of aggregate seating during which mean texture depth declined the SMA section (N12) was observed to remain constant at a terminal texture value of approximately 0.95 mm. This suggested that durability considerations could potentially favor the SMA section, which would eventually be investigated as part of the second phase of Track testing.

The 13th section on the north tangent was sponsored cooperatively by the entire sponsor oversight committee. It was decided early in the planning process that a gravel SMA (designed by NCAT personnel) would be the preferred mix for this section. Prior to construction, donation of the necessary materials was secured by the Mississippi Department of Transportation (MDOT). Their justification for this effort was contingent upon the same base mix being utilized for section N13 that would be used for their section S2 on the south tangent. Surface mix on section S2 was produced with crushed gravel blended to a BRZ gradation. In this manner, the group's plans for N13 were fulfilled and MDOT could double the meaning of their section S2 research.

As seen in Figure 4.19, section S2 exhibited 88 percent less rutting than section N13; however, rutting in N13 was still less than 3 ½ mm (approximately 5 mm of rutting is shown for section N13 in Figure 4.19 because of a slight amount of negative "rutting" prior to the application of traffic). Cracking along approximately 20 percent of the S2 centerline near the longitudinal joint between the inside and outside lanes appeared in February of 2002 and spread throughout the length of the section by December of 2002. No cracking was observed in section N13, which seemed to indicate that N13 would demonstrate better durability over the long term. Exposed diamond grinding across the transverse joint leading into section N13 was in good condition after the end of trucking in December of 2002.

Section S1 was also sponsored jointly by the entire sponsor group. It was decided that a relatively soft material with a high LA abrasion loss would be used to construct S1 using mix designs that were completed by NCAT personnel. Aggregate stockpile donations for this section were secured by the South Carolina Department of Transportation (SCDOT), which meant that rutting performance results from section S1 produced with relatively soft granite could be compared with the results from SCDOT's section S8 produced with more durable marble schist. As seen in Figure 4.20 (and Table 4.8), there was no statistically significant difference in rutting performance between these sections.

The primary purpose of the 12.5 mm nominal maximum aggregate size (NMAS) mix in section S8 was to facilitate its comparison with a 9.5 mm NMAS design in section S11 (also built with marble schist). These sections were actually placed about 10 mm thinner than other tangent sections at the request of the SCDOT in order for them to be consistent with their standard practice. Sections S8 and S11 were not adjacent to each other because the North Carolina Department of Transportation (NCDOT) made a similar request that their sections S9 and S10 be placed about 25 mm thinner than most other tangent sections. Smoothness was maximized on the surface of the Track by placing the thinnest sections (S9 and S10) together and bracketing them on the outside by the next thinnest sections (S8 and S11).

As seen in Figure 4.20 (and Table 4.8), there is no statistically significant difference between sections S8 and S11. The implication of this finding is that these 9.5 mm and 12.5 mm NMAS blends were interchangeable in terms of rutting performance. This does not mean that all 9.5 mm and 12.5 mm NMAS blends are interchangeable. Further, other considerations may override rutting comparability (e.g., surface friction, noise, economy, etc.). A slightly scarred mat in the inside lane of section S11 was the result of a roller maneuver during construction and was not related to any performance problem with the mix.

Although MDOT supplied the material for section N13, their formally sponsored research mixes were installed in the S2 and S3 locations. In contrast to the gravel SUPERPAVE mix in S2, section S3 was constructed with a gravel and limestone blend SUPERPAVE mix. The purpose of this experiment was to determine if the extra cost of adding limestone as a general policy (hauled into Mississippi at great expense) is cost effective with respect to rutting performance. As seen in Figure 4.21 (and Table 4.8), there was no significant difference in rutting performance between these sections; however, the cracking along the centerline of section S2 was not observed in section S3. In consideration of results obtained in sections N1 through N10 on the North tangent, it may be possible to increase the asphalt content of the S2 mix to improve cracking durability without sacrificing rutting performance. Both sections exhibited a similar loss of fine material (quantified by changing macrotexture in Figure 4.22), which is indicative of no substantial difference in mix durability.

Sections S4 and S5 were sponsored by the Tennessee Department of Transportation (TDOT). Like MDOT's research, TDOT intended to determine if the extra cost of limestone blending is necessary to optimize the rutting performance of resulting SUPERPAVE mixes. Section S4 was built with a blend of limestone and gravel, while section S5 was built exclusively with gravel. As seen in Figure 4.23 and Table 4.8, no significant difference in rutting performance was observed between these sections. At the time truck traffic was completed, there was no cracking of any type in either section.

Sections S6 and S7 were sponsored by the Florida Department of Transportation (FDOT). The purpose of the FDOT comparison, shown in Figure 4.24, was to study the effect of gradation (ARZ versus BRZ) in recycled mixes produced with marine limestone as virgin aggregates. In consideration of this objective, both sections were produced with unmodified asphalt in order to accentuate any differences in performance. After 10 million ESALs, the ARZ section (S6) exhibited 41 percent less rutting than the BRZ section (S7) with statistical significance (a difference of 1.2 mm). Small sticks were visible in the compacted surfaces of both sections as a result of contaminated stockpiles; however, their presence did not appear to create any surface performance problems.

As previously mentioned, the NCDOT sponsored sections S9 and S10 with a construction requirement that they be placed 25 mm thinner than other tangent sections. This was done to maximize the relevance of the Track comparison to the standard practice in North Carolina. Similar to the FDOT study, the purpose of NCDOT's study, shown in Figure 4.25, was to quantify the effect of gradation on rutting performance. Section S9 contained a coarse (BRZ) gradation mix, while section S10 contained a fine (ARZ) gradation mix. Both sections were produced with crushed granite, and unmodified asphalt was utilized in order to accentuate observed differences. With the completion of trucking operations, there was 39 percent less rutting in the BRZ section (S9) with statistical significance (Table 4.8); however, rutting in both sections was relatively minor. The difference in rutting between these sections was only 2.1 mm.

At the end of the south tangent, sections S12 and S13 were sponsored by the Oklahoma Department of Transportation (ODOT). The purpose of their study, shown in Figure 4.26, was to compare the performance of a limestone Hveem mix (in S12) with a granite mix newly designed using SUPERPAVE methodology (in S13). Both mixes were produced and placed with ease. Although they both exhibited very little permanent deformation, as shown in Table 4.8 there was statistically less rutting in section S13 (the granite section designed using SUPERPAVE methodology). Extensive centerline cracking was present in section S13 at the time the trucking operation was completed.

In addition to researching performance comparisons for section sponsors, NCAT was also responsible for pooling the results of the experiment in order to make observations and draw conclusions that are useful to the hot-mix asphalt community as a whole.

4.1.4.2 General Field Comparisons

A summary of rutting performance on the 2000 Track is presented in Figure 4.27, in which it is seen that only seven tangent sections exhibited rut depths greater than 3 mm. Although final rutting values are minor, Figure 4.28 illustrates they are comparable in distribution to those measured on 683 Special Performance Sections (SPS) of types 1, 5, 6, and 9 and documented in the database from the Long Term Pavement Performance Project (LTPP) in Release 15 of Datapave Online in 2003. In comparison to Track mixes, LTPP pavements varied in age at the time between three and twelve years (Jones, 2003).

The effect of changing asphalt content on rutting performance is a point of great interest for the pavement industry. In Figure 4.27, Track mixes produced with intentionally elevated binder contents are shown with light bars. All instances on the north tangent where binder content and grade were varied have been utilized in the construction of Figure 4.29. In this figure, unmodified binders appear on the upper portion of the graph, and fine (ARZ) mixes appear on the right portion of the graph. All optimum asphalt contents are relatively high for mixes on the north tangent due to the use of slag in the aggregate blends. Figure 4.29 clearly illustrates that increasing the binder content by ½ percent did not affect the rutting performance of mixes produced with modified asphalts. By eliminating all instances where binder content was elevated, it is possible to construct Figure 4.30. In this figure, the relationship between upper binder failure grade and Track rutting performance is observed. Based on the slope of the trend line in Figure 4.30, it was found that each 3°C increase in upper binder failure grade decreased rutting by approximately 1 mm.

In addition to the experiment on the north tangent, several other sponsor comparisons also focused on studying the effect of gradation type (BRZ versus ARZ) on mix performance. Although both mix types performed extremely well, Figure 4.31 illustrates that the common result on the 2000 Track was for slightly less rutting in BRZ aggregate blends. The only instance where an ARZ mix produced better rutting performance was also the only case where RAP was utilized for mix design and production; however, there is no evidence that the use of RAP produced this effect. Since both BRZ and ARZ mixes performed extremely well, consideration should be given to other design concerns (such as constructability, compactability, permeability, etc.) when selecting gradation type. For example, it may be worthwhile to switch to an ARZ gradation if permeability is expected to be a problem with coarser (BRZ) blends. Laboratory testing should be used to verify that switching to an ARZ gradation does not sacrifice rutting performance.

4.2 LABORATORY PERFORMANCE

Although it was important to conclude as much as possible from the field performance comparisons, the primary objective of the study was to develop laboratory to field performance correlations. To facilitate this process, laboratory performance data were first collected during the compaction process while research specimens were being fabricated. Shear was measured while samples were compacted in the SGC as well as the GTM. It was determined early on that a practical method of estimating field performance using readily available specimens was necessary; consequently, simulative and fundamental test protocols included in the laboratory portion of the study utilized standard samples prepared with SGCs within ±1 percent of QC air voids (unless ongoing testing indicated that special preparation was essential).

Additionally, all samples were subjected to both simulative and fundamental testing at the compacted height without trimming of any kind wherever possible because many labs are not currently equipped with wet masonry saws. Wet sawing also necessitates drying, which lengthens the time between when QC samples are fabricated and when laboratory performance data are available. With these prescribed limitations, Track testing conditions would serve to maximize the utility of performance predictions for mix design verification as well as for QC during construction.

4.2.1 Testing During Gyratory Compaction

Ideally, all testing during gyratory compaction should have been completed while test sections were being placed. This would have avoided the necessity of reheating materials and would have provided a better simulation for routine testing in a typical QC laboratory where reheating is generally avoided. This was possible in the measurement of fixed angle shear measurements and initial wobble angle testing with the oil cell GTM; however, LPI testing was performed offsite with materials that were shipped from the Track after construction was completed. Additionally, a modified air cell procedure in the GTM was developed after the Track had been completed; therefore, it was again necessary to reheat materials stored during construction. Detailed information on these tests is provided in Chapter 2, and laboratory performance data are summarized in Table 4.9.

4.2.1.1 Gyratory Testing Machine

The first set of samples was compacted in the GTM at the same time the Track was being built. This initial protocol utilized an oil roller, in which samples were compacted until either 300 gyrations was applied or the rate of densification approached zero. Because the effort to conduct this testing was in addition to the construction QC effort, it was only practical to run a single test for each unique mix. In consideration of this limitation, each GTM data point presented in Table 4.9 had much less statistical meaning than results from other laboratory protocols in which the average of 3 samples was reported.

The values reported in Table 4.9 for this first series of tests was the GSI. GSI is defined as the ratio of final shear strain divided by initial shear strain, where a value less than or equal to 1.00 is considered to be indicative of a stable (thus, rut resistant) mix

(McRae, 2001). As seen in Table 4.9, only nine of thirty-three unique Track mixes would be classified as stable using this criterion. Since no mixes on the Track exhibited poor rutting performance, it appears that GSI alone is not a good predictor of rutting performance. Figure 4.32 further reveals that oil cell GSI values do not appear to correlate with measured rutting performance in any way. This is not surprising, since the GTM method was developed to characterize dense blends containing natural aggregates mixed with unmodified binders for airfield construction (Lynch et al., 1999).

A second series of tests was run on loose mix that was sampled during construction, saved in sample storage buckets, reheated and compacted in a GTM that had been retrofitted such that the roller was pressurized with compressible air (to allow for compliance in the angle of compaction). The air cell is thought to be superior to the oil cell because it allows the mix to respond (thus, dilate) more aggressively (McRae, 2001). Based on the literature, it was theorized that any advantage in wobble angle compaction would be maximized through the use of the air cell. Again, it was only possible to compact a single specimen for each unique Track mix.

During air cell testing, two complete sets of test outputs were generated for each sample. Compaction was initiated with each mix at the documented placement temperature behind the paver. The first set of data was generated when the compacted sample height indicated sample VTM was equal to field VTM and testing was suspended. Density in the GTM was estimated via sample height during testing based on the relationship between height and bulk gravity after testing had been completed. The intention of this first stage of testing was to evaluate using the GSI parameter (i.e., shear strain at the time field VTM was attained divided by the initial shear strain) to estimate the compaction potential of each mix during construction. The number of revolutions required to reach field VTM was also a parameter of interest.

When test samples had cooled down to 64°C (the selected temperature for most laboratory performance test protocols), the compaction process was resumed until equilibrium was attained (i.e., the rate of densification went to zero) (McRae, 2001). The intention of this second stage of testing was to evaluate using the GTM to estimate the rutting potential of each mix under traffic. Long-term strain rate in the GTM (intended to represent mix performance under traffic) was computed as the change in bulk density divided by the number of corresponding revolutions. Data from both stages of testing is included in Table 4.9. As seen in Figure 4.32, the modified air cell procedure did not improve the apparent relationship between GTM and Track rutting performance.

4.2.1.2 Superpave Gyratory Compactor

As previously mentioned, one of several SGCs in the Track's on-site laboratory was equipped with a load cell that was used to measure the force required to maintain the prescribed SUPERPAVE angle of 1.25°. Based on the geometry of sample compaction, it was possible to convert applied load into the prescribed mix performance data using the following equations (Dalton, 1999):

$$Gyratory\ Shear = \frac{(Tilt\ Force)\cdot(Lever\ Arm)}{Mix\ Volume} \qquad \text{Equation 4.2}$$

$$Gyratory\ Shear\ Ratio = Sr = \frac{Gyratory\ Shear}{Fixed\ Compaction\ Stress} \qquad \text{Equation 4.3}$$

Pressure is a fixed parameter during sample compaction. Tilt force and mix volume were automatically measured with each gyration using the manufacturer's proprietary method, which was easily converted to Sr via Equations 4.2 and 4.3. In this manner, a value for Sr was computed for each compaction revolution and automatically stored on a removable disk. Height measurements were also recorded for each gyration. A separate file was created for each compaction sample that carried a date and time stamp that made it easier to relate data to the corresponding research mix. This entire process was transparent to laboratory technicians and required no user intervention. Because gyratory shear was measured all the way out to 300 gyrations (significantly past N_{des} for all Track mixes), it was necessary to prepare a completely separate set of volumetric specimens for QC purposes.

One challenge in processing shear data for the many different mixes utilized to build the Track is that data were generated for every compaction revolution. In testing with both the GTM and SGC, this meant potentially generating 300 points of data for each sample. In the case of SGC testing, three samples were processed for each mix type. This represented a large volume of data that had to be digested for inclusion in modeling efforts. To resolve this issue, data were averaged for the 3 SGC samples and reduced to provide frequent data points early in the compaction process and infrequent points late in the compaction process. Gyratory shear ratio and sample height were thus reported at 0, 1, 5, 10, 25, 50, 75, 100, 125, 150, 200, 250 and 300 gyrations. Raw data tables are quite large, and are not presented.

Gyratory shear data in Table 4.9 shows varying levels of maximum Sr values (Sr_{max}) for each mix in the SGC, as well as the number of gyrations where Sr_{max} was measured (N- Sr_{max}). As seen in Figure 4.33, there does not seem to be a significant relationship between mix performance and Sr_{max} or N- Sr_{max}. Other research has indicated Sr methods are more useful for screening bad mixes than for predicting performance (Anderson et al., 2002); however, the lack of "bad" mixes at the Track made it impossible to test this possibility.

By generating data for the same mix materials in both the GTM and SGC, it was also possible to evaluate whether the method of angle application (fixed versus wobble) had a measurable effect on measured shear. As seen in Figure 4.34, there does not appear to be a strong relationship between gyratory shear data generated with fixed (SGC) and wobble (GTM) angle compactors.

Loose materials sampled during construction were also shipped to Worchester Polytechnic Institute, where a select group of mixes were compacted in an SGC that was equipped with hardware to measure the lateral pressure between the mix and the mold wall. This was accomplished via hinged flaps on opposite sides of the mold that were each outfitted with load cells to estimate the amount of horizontal pressure being exerted on the mold wall during the compaction process. Analogous to the theory that horizontal pressure in soils varies by the strength of the material (as indicated by the angle of internal friction), it is believed that stable aggregate structures in stone skeleton mixes will rely less on the side forces of the mold wall to support the ram.

Data from lateral pressure testing is shown in Table 4.9. One would intuitively suspect that lateral pressure late in the compaction process (e.g., beyond the design level of gyrations) would have provided the best correlation with Track rutting performance; however, Figure 4.35 illustrates that lateral pressure at 1 gyration provided the best correlation with Track rutting. In fact, coefficients of determination decreased with all successive gyration levels. It is possible that the very low density condition present during early gyrations allowed the aggregate blends to more effectively mobilize. As the loose mass of mix consolidated early in the compaction process, the active condition developed. As the voids collapsed and the mix pressed against the wall of the mold, the passive condition developed. As in geotechnical applications, the relationship between active and passive pressures would have controlled the degree of shear strength mobilized in the mix. This finding indicates that shear stress levels induced by active lateral pressures in the laboratory better simulated shear stress levels on the surface of the Track.

It is unfortunate that, since testing was done offsite under the control of another research team, not enough sections were characterized to include lateral pressure results in the model development effort. Regardless, the uniqueness of the approach justified its attempted inclusion in the study and results from the limited dataset warrant further investigation.

4.2.2 Simulative Testing

A test plan encompassing simulative methods was developed based upon projected sample availability and access to technologies. All testing was conducted with dry sample conditioning to avoid the additional variability that may have been introduced as a result of varying stripping potential in the variety of aggregates used to construct the Track. It was decided to use the generally accepted terminus for LWT testing as the load limit for this study, which was 16,000 load applications (8,000 cycles in reciprocating machines). This was a deviation from recommended practice in some protocols (e.g., Hamburg testing is usually run out to 20,000 load cycles in order to identify a stripping breakpoint), but this step was deemed necessary to standardize the comparability of results. Samples were only conditioned long enough to achieve the desired test temperature, which was 64°C.

Although the APA is designed to test six samples simultaneously (two samples in each of three test rows), random numbers were used to blindly assign the three samples tested to the six possible locations within the APA to eliminate potential bias induced by sample location. It was decided to test only three samples in the APA as opposed to the standard six in order to avoid creating a statistical advantage for the APA in comparison to the other protocols (each run using only three samples).

Since the APA averages electronic deformation measurements for the 2 samples that make up each test row, it was necessary to interrupt testing at 1000, 2000, and 4000 load cycles so that manual depth measurements could be quickly obtained. Samples were immediately inserted back into the heating chamber and testing was restarted as quickly as possible to avoid the stiffening effect of sample cooling on rutting performance. A final set of manual depth measurements was made after 8000 load cycles when testing had been completed.

For Track testing, it was decided that each 25 mm outside diameter (19 mm inside diameter) rubber tube would be inflated with 827 kPa of pressure and loaded with 534 N, which was the likely recommended standard at the time that testing was initiated (Cooley, 2001). It should be noted that different hose pressures and loads can be utilized, as well as different hose diameters. Temperature is maintained by a heated cabinet in which three load mechanisms each traffic two SGC specimens, which means that six specimens can be tested simultaneously.

To maintain consistency in the Track comparison, three specimens representing two mixes were tested together in the APA. Random numbers were used to assign a location (of the six possible) to each specimen to avoid introducing a potential bias in the results based upon placement in the environmental chamber. Rutting was monitored automatically in real time via instrumentation and a computer with data acquisition software. Loading in the APA was applied at a rate of 60 cycles (120 load applications) per minute.

Conventional APA testing uses 75 mm tall specimens that are significantly shorter than the 115 mm tall QC specimens that were the focus of this experiment; consequently, 115 mm tall sample chocks (rather than the 75 mm tall standard chocks) were used to secure APA specimens during testing. Otherwise, the top 40 mm of each specimen would have been unconfined.

The RLWT was designed to function as a QC tool in a contractor's laboratory; consequently, it can only process a single sample at a time. It was observed that only 3 or 4 samples could be processed in a workday (and still ensure the desired testing temperature), which meant that more time was required to complete the testing program (compared to the APA). Also, the RLWT utilizes a heating jacket that is regulated via an infrared sensor to automatically maintain the desired testing temperature. While much simpler than the enclosed cabinets utilized in other devices, it could also produce greater variability in testing temperatures (thus, potentially greater variability in sample stiffness).

The European perspective in simulative testing was represented by running three samples each in the Hamburg LWT (currently used extensively in the United States) and Wessex Wheel Tracker. A single machine was utilized for both series of tests, with adjustments made in between to ensure that the proper load magnitude and rate were utilized. Due to the simplicity of the machine this simply meant turning a setscrew to adjust the speed of the load wheel and swapping out a hanging weight to adjust load magnitude. Both tests were run dry, although (like their American-origin counterparts) a hardware option exists to submerge specimens under water. Hamburg testing in the United States is typically run with wet samples; however, dry testing was used in this experiment in order to avoid the potentially confounding effect of stripping.

Wooden blocks were used to hold samples securely in place during Hamburg and Wessex testing. Since relatively tall specimens were used (i.e., 115 mm tall SGC specimens), it was necessary to produce blocks that were thicker than those originally provided by the manufacturer. Plaster of Paris was applied to the bottom of the specimens after being inserted into the molds to account for the sample height variability (± 5 mm) possible in SGC specimens.

A point of concern throughout testing was the ability of the plaster to resist punching under load, which could have produced unknown errors in the computerized, transducer generated rutting data. Investigating this possibility was beyond the scope of the experiment; however, visual observations did not seem to indicate it was a problem. Temperature in the heated cabinet was regulated by thermocouples that were inserted into the samples by drilling small holes. It was observed that this method did an excellent job of controlling test temperatures, and the drilling process was only a minor inconvenience.

As expected, testing with all the simulative devices was completed relatively quickly. It is seen in Table 4.10 that the different test protocols produced final deformation values that generally demonstrate the same trends in mix performance, although the magnitude of rutting values are different for each device. This observation is shown graphically in Figure 4.36, where it is seen that final rut depths from other protocols can be used to estimate equivalent values in the APA. Coefficients of determination were found to be 0.56 and 0.53 for the Hamburg and RLWT protocols, respectively. The coefficient of determination for Wessex testing was only 0.25, indicating a poor relationship.

It is useful to find that different protocols can be related to each other because it provides some assurance to practitioners that comparability between testing protocols exists, and what magnitude of difference can be expected in results. From Figure 4.36 it is seen that the Hamburg and RLWT provide the best correlations, with the Wessex device providing a much lower coefficient of determination. This makes intuitive sense in the case of the RLWT because it was designed to load samples with a contact pressure that is approximately equal to the hose pressure applied in the APA.

Although the same machine was used to run Hamburg and Wessex testing in the Track's on-site laboratory, the load used for Hamburg testing is significantly greater and applied at a much higher rate. This change apparently provided for a stronger relationship between the APA and the Hamburg procedure, although the magnitude of the Hamburg results is approximately 50 percent greater than those from the APA. An iterative procedure could be used in future research to optimize the load and load rate in the Hamburg method to produce the greatest correlation with the APA.

4.2.3 Fundamental Testing

As was the case in simulative testing, the goal for the test plan for fundamental testing was to utilize standard QC specimens (compacted in an SGC) that were 115 mm (± 5 mm) tall with minimal preparation in terms of sawing or coring. It was known ahead of time that SST samples would require sawing in order to prepare the 50 mm thick test specimens, but it was hoped that untrimmed samples could be used for RLCC triaxial testing. Results from fundamental testing are presented in Table 4.11, and related graphically to Track rutting in Figure 4.37. Results from each protocol are discussed in the following paragraphs; however, correlations with Track rutting performance are generally poor.

4.2.3.1 Shear Testing

The Track SST test procedure consisted of running frequency sweep testing for dynamic modulus then testing in simple shear at 20, 40, and finally 64°C. Afterward, each of three samples was subjected to 5000 cycles of repeated shear testing at 64°C with the vertical pressure automatically adjusted to maintain a constant height. It should be noted that the upper temperature was increased from 60 to 64°C to enhance the possibility of obtaining results comparable to those observed in the field, as well as with other methods at the same temperature.

As reported by others (Romero and Anderson, 2001), relatively large variability was encountered in SST results. An average coefficient of variation (COV) of 23 percent was encountered in Track RSCH testing based on three QC pills for each unique mix (all COV values for SST testing are shown in Table 4.11). Other researchers have also encountered an average COV of greater than 20 percent for their performance experiment when a variety of mix types was utilized (Stuart, 2002). The SST equipment was not easy to use and it was difficult to keep running for extended periods of time. Attaching the platens to sawed test specimens was a significant inconvenience. Future generations of field and laboratory shear devices would be much easier to use in practice if a method of testing can be identified that does not require the use of epoxy.

4.2.3.2 Dynamic Modulus Testing

Dynamic modulus testing was run on cored samples just before triaxial testing was initiated. In consideration of other research (Pellinen and Witczak, 2002), it was originally planned that dynamic modulus testing would only be conducted at 5 Hz; however, it was very easy to program the equipment to automatically cycle through a range of frequencies while only adding minutes to the total test time. In consideration of the ease in collecting additional data, control software was written for the automated system that ran each sample through a range of frequencies including 10, 5, 1, 0.5 and 0.1 Hz. It was theorized that these data (shown with other fundamental data in Table 4.11) could be used to develop a master curve that could be used to predict mix stiffness as a function of frequency, which could be useful in the construction of rutting performance models.

In Figure 4.38, it is seen that average master curves shift upward as binder failure grade increases. This is a reflection of the increased stiffness of the binder itself. As expected, mixes containing PG68 neat binder had the lowest modulus values. Average moduli from mixes containing SB-modified PG73 were approximately equal to mixes containing SBS-modified PG78, both running about 10 percent higher than mixes containing PG68 neat binder. Average moduli from mixes containing SBR-modified PG80 averaged about 40 percent higher than mixes containing PG68 neat binder.

4.2.3.3 Triaxial Testing

For the purpose of this study, a computer-controlled load frame with an integrated environmental chamber was used to determine dynamic modulus on each Track sample, which is a nondestructive procedure. Without removing the specimen from the environmental chamber, repeated load confined cyclic (RLCC) triaxial testing was conducted to failure. Static creep testing has also shown promise in predicting performance; however, it was decided to assign a higher priority to RLCC testing. As a consequence of logistical limitations, it was ultimately not possible to include static creep testing in this research effort.

As was the case in high temperature SST testing, all triaxial testing was performed at 64°C. A confining stress of 138 kPa was applied via a custom-fabricated triaxial cell housed inside a larger environmental chamber. Procedural information available from the NCHRP 9-19 study to define the SUPERPAVE model draft tests was utilized to run these methods (Procedure "W2" addressed repeated load testing), in addition to the test for dynamic modulus (referred to at the time as procedure "X1").

It was originally planned that triaxial tests would be run utilizing a deviator stress level of 689 kPa; however, preliminary testing revealed that this level of deviator stress would induce so much deformation in 64°C test samples that catastrophic gauge damage would result. In fact, deformation was so great that no supporting data could be collected to report. Following an iterative process, it was decided that a deviator stress level of 552 kPa would provide adequate (but not excessive) deformation to potentially differentiate performance while at the same time avoid costly damage to testing hardware.

Preliminary testing also indicated that it was not possible to attach strain transducers to the sides of uncored specimens in a manner that would prevent the gages from slipping at test temperatures (64°C). The only other option (using existing laboratory hardware) was to utilize actuator displacement to determine strain during a test. In this method, the position of the load frame ram was continuously monitored using a displacement transducer such that sample strain could be computed throughout the test by dividing total ram movement by original sample height. Again, it was found that this approach was ineffective when technicians observed plunger displacements that were unrelated to gage strains. No data is available to support this observation.

Initially, it was hoped that triaxial and dynamic modulus testing could be accomplished with uncut, unmodified SGC specimens to be consistent with other Track test protocols; however, it was determined early on that displacement instrumentation could not be mounted on the sides of uncored specimens with enough reliability to prevent slippage when the binder coating on surface aggregates softens at elevated test temperatures. Additionally, it was noted in early trial testing that end effects in the squatty, unmodified samples were negatively affecting test outcomes by producing overlapping stress cones. These results are consistent with findings from earlier works in which samples with greater height to diameter ratios were found to be desirable (Mallick et al., 1995).

In consideration of these early experiences, it was reluctantly decided that coring and sawing was the only practical way to facilitate meaningful triaxial results. A laboratory cutting stand was developed to hold 150 mm in diameter specimens in place during coring, and samples that were approximately 100 mm in diameter by 115 mm in height (the full height from the SGC) were ultimately utilized. Although the confined cyclic and dynamic modulus procedures were prioritized, other specimens were preserved for future unconfined cyclic triaxial testing and both confined and unconfined static creep testing. Based on the results of others, it was anticipated that results from dynamic modulus testing would be more greatly affected by the unorthodox sample size than repeated loading (Mallick et al., 1995). This is because the confining stress in RLCC testing serves to reduce the end effects that would otherwise develop (Foo, 1994). In comparison, dynamic modulus testing is run in the unconfined condition and is vulnerable to detrimental end effects.

Throughout the RLCC triaxial test procedure, sample strains were recorded as a function of load cycles. For each test, the linear portion of the load curve that occurred in the later portion of testing was identified and used to calculate slope. Based on the coordinates of the linear portion of the load curve and the calculated slope, the zero cycle intercept was also identified. In this manner, an intercept and slope value were identified for each triaxial test. Knowing these parameters, it was also possible to compute a 10,000-cycle strain value that could also be used for performance correlation purposes. As seen in Table 4.11, variability in triaxial testing was even higher than was encountered in SST testing. The average COV of measured slopes was 68 percent, and the average COV of extrapolated intercepts was 37 percent.

4.2.3.4 Seismic Testing

Stiffness differences in Track pavements were also characterized using wave propagation theory. In December of 2001, Dr. Soheil Nazarian from the University of Texas traveled to the Track in order to record stress wave velocities on QC pills and cores cut from the wheelpaths. A hammer was used to induce transient impacts, and fixed geometry transducers were used to measure the velocity of transmitted stress waves.

With wave velocities known, it was then possible to calculate the modulus of each mix via Equation 4.4 (Hugo et al., 2004):

$$Modulus = 2 \cdot Density \cdot (1 + Poissons\ Ratio) \cdot (Stress\ Wave\ Velocity)^2 \qquad \text{Equation 4.4}$$

where

Poissions Ratio = Ratio of Horizontal Elongation to Vertical Shortening Under Load

Results from seismic testing of pills are included in this research because if correlations with field performance can be established using QC pills the resulting methodology would be convenient for use in QC testing. Seismic moduli for QC pills and cores are presented in Table 4.11. Here it is seen that there is a relationship between laboratory and field measured data; however, there does not appear to be any relationship between seismic moduli and Track rutting performance. This lack of relationship with plastic deformation in the field is likely the result of a lack of comparability between very low strain elastic seismic measurements and relatively high strain plastic visco-elastic pavement rutting under heavy truck loads.

Spectral Analysis of Surface Waves (SASW) testing was also conducted by Dr. Nazarian on the surface of the Track, but since it would not be possible to collect those data prior to construction it was beyond the scope of this research effort.

4.3 RESEARCH PLAN FOR MODEL DEVELOPMENT

Data generated by testing during gyratory compaction, simulative testing and fundamental testing that ultimately served as the basis of the modeling effort are shown in Tables 4.10, 4.11 and 4.12, respectively. The process of developing rutting performance prediction models using these data began with an incremental assessment of rutting performance on the Track under actual truck traffic. Each Monday, the total rut depth was compared to the rut depth from the previous week in order to calculate the change in rutting that resulted from the last week's truck traffic.

The ESAL record for that same week was then compared to the record of pavement surface temperatures. The temperature at the time each ESAL was applied was used to group ESALs into predetermined temperature ranges. The logical framework for these ranges was found in the performance grading system for asphalt binders, in which a new band begins every 6°C. A rational method to increase the rut-inducing effect of ESALs applied at hotter temperatures was devised using a mechanistic approach.

For each laboratory test protocol, laboratory performance data were correlated via regression to rutting performance for a specific time interval as some function of temperature banded ESALs. The best interval for this process was determined through a trial and error assessment of all time periods. A method was then devised to allow these predictions to be age adjusted to successfully predict rutting performance within all other time intervals. Since the purpose of the experiment was to develop a performance prediction procedure that could be used in the real world where rutting outcome is not known, only information that was known at the time of Track construction could be used as model inputs.

As a consequence of providing each sponsor with the authority to choose comparisons that best satisfied their research needs, a great variety of mix types and materials were used to place the top 100 mm of the Track's surface. It could be viewed as a disadvantage that NCAT did not have the authority to design the experiment to limit the number of variables and maximize the opportunity to test hypotheses; however, the variety of surface mixes and materials presented an excellent opportunity to develop a rutting prediction methodology that would have a broad practical application.

TABLE 4.1 Profile Data Manipulation to Produce Rut Depth Data (Peaks and Valleys Identified Using Iterative Method to Produce the Greatest Rut Depth Calculation)

Profile Location			Distance to Peaks/Valleys						Difference in Elevation (mm) from Centerline Reference Point with Each Step of Dipstick Profiler (m)											
Quad	Sec	Ran	$P1_{IWP}$	V_{IWP}	$P2_{IWP}$	$P1_{OWP}$	V_{OWP}	$P2_{OWP}$	0.00	0.30	0.61	0.91	1.22	1.52	1.83	2.13	2.44	2.74	3.05	3.35
N	1	1	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-2.61	-5.83	-8.42	-10.79	-12.36	-16.69	-20.93	-25.13	-28.85	-30.45	-32.16
N	1	2	0.30	0.61	1.52	1.52	2.44	3.35	0.00	-3.14	-7.02	-9.66	-12.50	-15.25	-20.40	-26.37	-31.28	-35.42	-38.79	-40.75
N	1	3	0.30	0.61	1.52	1.52	2.44	3.35	0.00	-6.64	-13.70	-19.34	-24.62	-29.68	-37.42	-46.10	-53.86	-59.98	-65.72	-70.72
N	2	1	0.30	0.61	1.52	1.52	2.44	3.35	0.00	-7.76	-15.30	-21.91	-28.64	-35.22	-44.23	-53.93	-62.56	-69.41	-75.21	-81.75
N	2	2	0.30	0.61	1.22	1.52	2.44	3.35	0.00	-5.69	-12.44	-18.25	-24.31	-31.23	-39.52	-48.71	-56.97	-64.40	-70.06	-77.66
N	2	3	0.30	0.61	1.22	1.52	2.44	3.35	0.00	-5.85	-12.40	-17.88	-23.70	-29.73	-38.18	-47.18	-55.52	-62.80	-68.79	-75.59
N	3	1	0.30	0.91	1.22	1.52	2.44	3.35	0.00	-4.39	-11.95	-17.58	-18.25	-23.30	-32.38	-44.69	-59.00	-66.71	-70.79	-75.66
N	3	2	0.30	0.91	1.52	1.52	2.44	3.35	0.00	-3.36	-12.72	-19.69	-23.65	-28.44	-38.13	-48.23	-61.36	-68.17	-71.97	-75.73
N	3	3	0.30	0.91	1.52	1.52	2.44	3.35	0.00	-7.29	-16.72	-22.28	-24.62	-28.46	-36.91	-47.23	-59.80	-65.86	-68.20	-72.06
N	4	1	0.30	0.91	1.52	1.83	2.44	3.35	0.00	-6.77	-15.38	-21.42	-25.69	-28.60	-36.40	-47.64	-57.42	-65.03	-71.95	-76.42
N	4	2	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-6.62	-15.40	-21.47	-25.37	-29.18	-37.55	-48.66	-59.04	-67.22	-72.96	-77.21
N	4	3	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-5.50	-12.38	-16.93	-19.30	-20.46	-26.88	-35.75	-43.37	-49.50	-53.78	-56.28
N	5	1	0.30	0.91	1.52	1.52	2.44	3.35	0.00	-1.12	-5.66	-7.39	-7.47	-7.54	-13.80	-22.34	-29.98	-34.87	-35.34	-37.20
N	5	2	0.30	0.61	1.52	1.52	2.44	3.35	0.00	-0.40	-5.91	-8.54	-9.56	-12.40	-21.05	-30.67	-42.36	-48.04	-50.74	-53.86
N	5	3	0.30	0.61	1.52	1.52	2.44	3.35	0.00	-2.32	-9.25	-10.74	-11.70	-14.08	-21.92	-30.37	-42.52	-47.01	-48.74	-52.04
N	6	1	0.30	0.91	1.52	1.52	2.44	3.35	0.00	-3.03	-7.37	-9.51	-10.02	-10.08	-16.43	-24.12	-31.03	-35.82	-38.60	-42.11
N	6	2	0.30	0.61	1.52	1.52	2.44	3.35	0.00	-0.70	-3.51	-4.53	-5.00	-6.29	-12.84	-19.94	-26.25	-30.14	-32.03	-36.81
N	6	3	0.30	0.61	1.22	1.52	2.44	3.05	0.00	-2.42	-7.01	-9.69	-11.91	-16.93	-25.22	-34.48	-43.05	-47.79	-52.37	-59.87
N	7	1	0.30	0.61	1.22	1.52	2.44	3.35	0.00	-5.83	-11.17	-15.91	-20.49	-25.77	-32.59	-39.54	-46.85	-53.26	-59.23	-64.99
N	7	2	0.30	0.61	1.52	1.83	2.44	3.35	0.00	-7.21	-13.07	-17.61	-21.89	-25.46	-31.29	-37.99	-45.14	-51.19	-56.52	-61.66
N	7	3	0.30	0.91	1.52	1.83	2.44	3.35	0.00	-4.73	-9.72	-14.42	-18.58	-22.48	-28.09	-34.98	-42.05	-48.36	-54.31	-60.10
N	8	1	0.61	1.22	1.52	1.83	2.44	3.35	0.00	-0.03	-2.75	-6.07	-9.58	-12.79	-17.90	-24.96	-32.58	-39.04	-45.16	-50.82
N	8	2	0.61	0.91	1.52	1.83	2.44	3.05	0.00	0.25	-3.69	-8.49	-12.87	-17.41	-23.98	-32.39	-41.55	-49.12	-56.49	-64.84
N	8	3	0.61	0.91	1.52	1.83	2.44	3.35	0.00	-0.08	-4.45	-9.07	-13.34	-17.48	-22.89	-30.55	-39.01	-46.13	-52.61	-59.56
N	9	1	0.30	1.22	1.52	1.83	2.44	3.35	0.00	0.72	-5.83	-12.06	-18.54	-24.32	-31.78	-39.73	-47.65	-55.11	-62.04	-69.46
N	9	2	0.30	0.91	1.52	1.83	2.44	3.05	0.00	-1.85	-8.57	-15.10	-21.22	-27.04	-34.45	-42.49	-50.65	-57.85	-65.11	-73.09
N	9	3	0.30	1.22	1.52	1.83	2.44	3.05	0.00	-1.79	-8.19	-14.71	-21.26	-27.22	-34.98	-43.57	-52.39	-59.92	-67.45	-75.63
N	10	1	0.61	1.22	1.52	1.83	2.44	3.05	0.00	-3.49	-8.65	-14.44	-20.24	-25.30	-33.01	-41.77	-50.67	-57.96	-64.98	-73.35
N	10	2	0.61	0.91	1.52	1.83	2.44	3.05	0.00	-0.80	-4.79	-9.89	-14.36	-18.71	-26.07	-34.29	-42.72	-49.72	-55.43	-63.37
N	10	3	0.61	0.91	1.52	1.83	2.44	3.05	0.00	-1.06	-4.31	-8.07	-11.51	-14.66	-20.57	-27.74	-34.47	-40.01	-44.98	-52.35
N	11	1	0.30	1.22	1.52	1.83	2.44	3.05	0.00	-2.88	-7.95	-13.39	-18.47	-22.94	-29.60	-36.03	-42.41	-48.66	-54.25	-60.86
N	11	2	0.61	1.22	1.52	1.52	2.74	3.35	0.00	-3.10	-8.25	-13.87	-19.28	-23.83	-31.19	-38.24	-45.09	-51.72	-57.51	-62.18
N	11	3	0.30	0.91	1.52	1.52	2.44	3.35	0.00	-5.17	-11.77	-18.23	-23.83	-28.94	-36.51	-42.19	-48.96	-55.13	-61.58	-67.15
N	12	1	0.30	1.22	1.52	1.83	2.44	3.35	0.00	-1.32	-6.74	-11.63	-16.20	-19.42	-25.76	-33.35	-40.65	-47.05	-53.40	-58.47
N	12	2	0.30	1.22	1.52	1.83	2.44	3.35	0.00	-1.63	-5.11	-7.73	-9.83	-10.79	-14.11	-19.19	-23.90	-27.43	-30.40	-32.86
N	12	3	0.30	0.91	1.52	1.52	2.44	3.35	0.00	0.57	-4.02	-7.92	-11.00	-13.45	-19.87	-26.13	-31.80	-36.46	-41.19	-45.05
N	13	1	0.30	0.91	1.52	1.52	2.44	3.05	0.00	3.12	0.21	-2.43	-3.16	-3.39	-8.94	-14.75	-20.84	-24.84	-26.80	-32.71
N	13	2	0.30	0.91	1.52	1.52	2.44	3.35	0.00	-3.05	-8.20	-12.31	-15.32	-17.26	-25.78	-34.56	-43.63	-50.80	-56.21	-62.04
N	13	3	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-3.91	-9.47	-14.00	-17.65	-21.14	-29.94	-39.76	-49.78	-58.83	-64.96	-73.70
S	1	1	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-1.69	-3.44	-5.43	-5.38	-5.26	-7.27	-10.66	-14.46	-16.94	-18.18	-19.53
S	1	2	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-0.64	-2.75	-4.03	-4.08	-5.39	-9.41	-14.73	-19.91	-23.91	-26.21	-28.35
S	1	3	0.30	0.91	1.22	1.83	2.44	3.05	0.00	-4.36	-9.21	-13.53	-17.21	-21.82	-27.27	-33.54	-39.14	-44.24	-49.19	-54.79
S	2	1	0.30	0.91	1.22	1.52	2.44	3.05	0.00	-1.66	-3.02	-4.10	-4.90	-6.98	-10.76	-14.77	-18.45	-22.00	-24.72	-28.85
S	2	2	0.30	0.91	1.22	1.83	2.13	3.35	0.00	-3.68	-6.60	-9.28	-11.55	-14.60	-18.53	-23.20	-27.08	-30.93	-34.74	-38.70
S	2	3	0.30	0.91	1.22	1.83	2.44	3.05	0.00	-2.99	-5.55	-8.03	-9.95	-12.92	-16.48	-20.74	-24.56	-28.15	-31.06	-34.78
S	3	1	0.30	0.91	1.22	1.52	2.13	3.05	0.00	-3.31	-6.40	-9.63	-12.34	-15.92	-20.48	-25.33	-29.44	-33.27	-36.52	-43.15
S	3	2	0.30	0.91	1.22	1.52	2.13	3.05	0.00	-1.67	-4.50	-7.66	-9.80	-12.67	-16.86	-21.40	-25.17	-28.69	-31.74	-37.91
S	3	3	0.30	0.91	1.52	1.83	2.13	3.05	0.00	-1.75	-3.22	-4.80	-5.24	-6.16	-8.10	-10.46	-12.35	-14.16	-16.10	-19.50
S	4	1	0.30	0.91	1.22	1.83	2.13	3.05	0.00	-0.20	0.19	0.29	1.02	-0.08	-2.30	-4.99	-7.50	-10.08	-11.29	-15.72
S	4	2	0.30	0.61	1.22	1.83	2.13	3.05	0.00	-0.63	-0.46	0.04	0.46	-0.47	-2.14	-4.67	-6.68	-8.81	-9.53	-12.95
S	4	3	0.30	0.61	0.91	1.52	2.13	3.05	0.00	-3.60	-6.65	-9.39	-12.62	-17.02	-23.19	-29.59	-35.17	-41.17	-46.24	-52.32
S	5	1	0.61	0.91	1.52	1.52	2.74	3.35	0.00	2.33	0.30	-2.53	-4.82	-7.13	-10.04	-13.86	-16.89	-19.85	-22.08	-24.20
S	5	2	0.30	0.91	1.52	1.52	2.13	3.35	0.00	-4.55	-8.91	-13.50	-17.20	-21.41	-27.14	-32.28	-36.95	-41.26	-45.03	-49.49
S	5	3	0.30	0.91	1.52	1.52	2.13	3.05	0.00	-2.95	-8.25	-13.94	-18.98	-23.88	-30.50	-36.84	-42.38	-48.03	-52.94	-58.89
S	6	1	0.30	0.91	1.52	1.83	2.74	3.35	0.00	0.72	-1.16	-3.16	-3.89	-4.66	-8.12	-13.94	-18.81	-23.01	-25.76	-28.36
S	6	2	0.30	0.91	1.52	1.83	2.74	3.05	0.00	1.13	-3.51	-7.03	-9.53	-12.40	-17.97	-26.23	-33.13	-39.28	-43.22	-47.23
S	6	3	0.30	0.91	1.52	1.83	2.74	3.35	0.00	0.51	-3.73	-7.74	-11.00	-14.58	-19.89	-27.91	-34.79	-40.81	-45.25	-49.69
S	7	1	0.30	0.91	1.52	1.52	2.74	3.35	0.00	0.51	-3.60	-7.26	-8.04	-10.13	-18.30	-27.41	-35.60	-42.57	-46.42	-51.05
S	7	2	0.30	0.91	1.52	1.52	2.44	3.35	0.00	0.64	-4.22	-8.59	-9.90	-12.52	-20.37	-30.18	-39.05	-46.08	-50.61	-55.13
S	7	3	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-0.26	-4.74	-8.72	-9.63	-11.69	-18.42	-26.84	-35.84	-42.85	-47.29	-50.91
S	8	1	0.30	0.91	1.22	1.83	2.74	3.35	0.00	-3.61	-6.85	-10.32	-12.40	-15.35	-20.34	-27.41	-33.32	-38.98	-43.48	-46.66
S	8	2	0.61	0.91	1.22	1.52	2.74	3.35	0.00	-3.82	-7.21	-10.92	-14.30	-18.04	-24.56	-31.91	-38.69	-45.04	-50.28	-54.58
S	8	3	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-3.74	-8.04	-12.35	-15.83	-19.84	-26.21	-33.37	-40.19	-46.49	-51.84	-56.06
S	9	1	0.30	0.61	1.22	1.83	2.74	3.35	0.00	-2.55	-4.61	-7.85	-11.78	-17.40	-25.04	-34.73	-43.61	-51.93	-58.78	-65.30
S	9	2	0.61	0.91	1.22	1.83	2.74	3.35	0.00	-2.02	-3.68	-6.35	-9.69	-14.40	-21.71	-30.75	-39.03	-46.73	-53.31	-58.87
S	9	3	0.61	0.91	1.22	1.83	2.74	3.35	0.00	-2.40	-4.16	-6.41	-9.51	-13.94	-21.02	-29.90	-38.02	-45.86	-52.32	-57.77
S	10	1	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-3.67	-7.86	-11.55	-13.81	-16.91	-22.28	-30.01	-37.08	-43.18	-48.40	-51.63
S	10	2	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-4.07	-7.55	-10.17	-11.18	-12.26	-15.31	-21.50	-27.40	-31.91	-34.77	-35.74
S	10	3	0.30	0.91	1.52	1.52	2.74	3.35	0.00	-4.68	-8.29	-11.52	-13.41	-14.85	-19.44	-26.97	-32.46	-37.09	-40.28	-41.57
S	11	1	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-2.20	-5.08	-7.20	-9.25	-11.25	-14.66	-20.29	-25.33	-30.14	-34.03	-37.83
S	11	2	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-6.06	-9.71	-12.78	-15.01	-17.43	-20.68	-25.64	-30.67	-35.63	-39.35	-41.97
S	11	3	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-4.55	-8.66	-12.56	-15.37	-18.85	-23.87	-30.63	-37.13	-43.27	-47.91	-52.73
S	12	1	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-2.04	-4.28	-6.20	-6.85	-6.83	-8.55	-12.75	-16.76	-20.20	-22.14	-23.53
S	12	2	0.30	0.91	1.83	1.83	2.74	3.35	0.00	-1.91	-4.00	-5.22	-5.19	-4.74	-5.15	-8.27	-10.74	-12.83	-13.67	-14.10
S	12	3	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-0.90	-1.33	-1.78	-0.01	1.83	1.80	-0.01	-1.74	-2.92	-2.64	-1.56
S	13	1	0.30	0.91	1.52	1.83	2.74	3.35	0.00	-0.03	0.35	0.58	1.82	2.91	3.56	1.74	-0.24	-2.01	-3.20	-3.90
S	13	2	0.30	0.91	1.83	1.83	2.74	3.35	0.00	-0.86	-0.98	-0.91	0.03	0.78	1.45	-1.64	-3.73	-5.72	-6.98	-7.92
S	13	3	0.30	0.91	1.83	2.13	2.74	3.35	0.00	-1.27	-1.34	-1.13	-0.25	0.53	1.73	2.24	0.03	-1.58	-2.06	-2.27

TABLE 4.2 Wire Line Rutting Measurements (Corrected for Wire Thickness) after 10 Million ESALS via 9 Measurement Locations per Section

Track Quad	Sec Num	Left Rut Avg (mm)	Right Rut Avg (mm)	Both Wheelpaths Avg (mm)	Both Wheelpaths Std Dev (mm)
N	1	0.50	3.64	2.07	0.55
N	2	0.95	3.04	2.00	0.44
N	3	4.88	9.65	7.27	0.60
N	4	4.30	6.28	5.29	0.41
N	5	4.81	9.31	7.06	0.95
N	6	2.65	4.95	3.80	0.60
N	7	1.52	1.63	1.58	0.42
N	8	0.26	1.53	0.89	0.28
N	9	0.12	0.91	0.51	0.22
N	10	0.23	1.63	0.93	0.14
N	11	1.01	1.83	1.42	0.35
N	12	1.50	2.71	2.10	0.31
N	13	2.70	4.19	3.44	0.56
S	1	1.31	2.35	1.83	0.66
S	2	0.36	0.57	0.46	0.14
S	3	0.30	0.74	0.52	0.19
S	4	0.55	0.77	0.66	0.52
S	5	0.22	1.15	0.68	0.33
S	6	0.92	3.19	2.06	0.27
S	7	2.14	4.47	3.30	0.31
S	8	1.47	2.03	1.75	0.58
S	9	0.77	3.24	2.01	0.45
S	10	3.12	5.16	4.14	1.08
S	11	1.16	2.04	1.60	0.49
S	12	2.13	2.91	2.52	0.32
S	13	1.94	1.23	1.58	0.99
Average		**1.61**	**3.12**	**2.36**	**0.47**

TABLE 4.3 Rutting Measurements from 3 Different Methods after 10 million ESALs

(Wire Line and Elevation Profile Data Using 3- versus 6-Point Methodologies)

Track	Sec	Wire Line	Profile Based Rut Depths (mm)	
Quad	**Num**	**Avg Rut (mm)**	**Avg Using 3 Pt Geometry**	**Avg Using 6 Pt Geometry**
N	1	2.07	4.36	2.22
N	2	2.00	4.25	1.88
N	3	7.27	11.08	6.75
N	4	5.29	8.41	4.58
N	5	7.06	11.09	5.98
N	6	3.80	9.23	3.38
N	7	1.58	4.33	1.15
N	8	0.89	5.95	0.82
N	9	0.51	3.60	0.73
N	10	0.93	5.12	1.10
N	11	1.42	3.56	1.02
N	12	2.10	4.68	1.79
N	13	3.44	7.14	2.66
S	1	1.83	5.42	1.67
S	2	0.46	3.47	0.55
S	3	0.52	3.15	0.72
S	4	0.66	3.43	0.45
S	5	0.68	2.07	0.93
S	6	2.06	5.93	1.91
S	7	3.30	7.68	3.87
S	8	1.75	5.28	1.60
S	9	2.01	6.89	1.08
S	10	4.14	7.79	2.85
S	11	1.60	5.25	1.37
S	12	2.52	5.63	2.28
S	13	1.58	4.66	1.18
Average		2.36	5.75	2.10

TABLE 4.4 Final Rut Depths Induced by the Application of 10 million ESALs (Shown with 2000 Track Experiment Design) via (Wire Line) Corrected 6-Point Method

Track Quad	Section Num	Aggregate Blend Type	Design Method	Design NMA	Grad Type	Binder Grade	Binder Modifier	Lift Type	Design Thick	Initial Apparent "Rutting" (mm)	Final Wire Line Rutting (mm)	Traffic Induced Rutting (mm)
N	1	Slag/Lms	Super	12.5	ARZ	76-22	SBS	Dual	4.0	0.17	2.07	1.90
N	2	Slag/Lms	Super	12.5	ARZ	76-22+	SBS	Dual	4.0	-0.15	2.00	2.15
N	3	Slag/Lms	Super	12.5	ARZ	67-22+	NA	Dual	4.0	-0.08	7.27	7.35
N	4	Slag/Lms	Super	12.5	ARZ	67-22	NA	Dual	4.0	-0.11	5.29	5.40
N	5	Slag/Lms	Super	12.5	BRZ	67-22+	NA	Dual	4.0	-0.99	7.06	8.05
N	6	Slag/Lms	Super	12.5	BRZ	67-22	NA	Dual	4.0	-0.60	3.80	4.40
N	7	Slag/Lms	Super	12.5	BRZ	76-22+	SBR	Dual	4.0	-0.72	1.58	2.30
N	8	Slag/Lms	Super	12.5	BRZ	76-22	SBR	Dual	4.0	-0.46	0.89	1.35
N	9	Slag/Lms	Super	12.5	BRZ	76-22	SBS	Dual	4.0	-0.19	0.51	0.70
N	10	Slag/Lms	Super	12.5	BRZ	76-22+	SBS	Dual	4.0	0.28	0.93	0.65
N	11	Granite	Super	19.0	BRZ	67-22	NA	Lower	2.5			
		Granite	Super	12.5	TRZ	76-22	SBS	Upper	1.5	-0.23	1.42	1.65
N	12	Granite	Super	19.0	BRZ	67-22	NA	Lower	2.5			
		Granite	SMA	12.5	SMA	76-22	SBS	Upper	1.5	-0.40	2.10	2.50
N	13	Gravel	Super	19.0	BRZ	76-22	SBS	Lower	2.5			
		Gravel	SMA	12.5	SMA	76-22	SBS	Upper	1.5	-1.51	3.44	4.95
S	1	Granite	Super	19.0	BRZ	76-22	SBS	Lower	2.5			
		Granite	Super	12.5	BRZ	76-22	SBS	Upper	1.5	0.03	1.83	1.80
S	2	Gravel	Super	19.0	BRZ	76-22	SBS	Lower	2.5			
		Gravel	Super	9.5	BRZ	76-22	SBS	Upper	1.5	-0.14	0.46	0.60
S	3	Limestone	Super	19.0	BRZ	76-22	SBS	Lower	2.5			
		Lms/Gravel	Super	9.5	BRZ	76-22	SBS	Upper	1.5	-0.08	0.52	0.60
S	4	Lms/RAP	Super	19.0	ARZ	76-22	SBS	Lower	2.5			
		Limestone	Super	12.5	ARZ	76-22	SBS	Upper	1.5	-0.04	0.66	0.70
S	5	Lms/Grv/RAP	Super	19.0	BRZ	76-22	SBS	Lower	2.5			
		Gravel	Super	12.5	TRZ	76-22	SBS	Upper	1.5	0.08	0.68	0.60
S	6	Lms/RAP	Super	12.5	ARZ	67-22	NA	Dual	4.0	-0.04	2.06	2.10
S	7	Lms/RAP	Super	12.5	BRZ	67-22	NA	Dual	4.0	-0.25	3.30	3.55
S	8	Marble-Schist	Super	19.0	BRZ	67-22	NA	Lower	2.1			
		Marble-Schist	Super	12.5	BRZ	76-22	SBS	Upper	1.5	0.15	1.75	1.60
S	9	Granite	Super	12.5	BRZ	67-22	NA	Dual	3.0	-0.74	2.01	2.75
S	10	Granite	Super	12.5	ARZ	67-22	NA	Dual	3.0	-0.36	4.14	4.50
S	11	Marble-Schist	Super	19.0	BRZ	67-22	NA	Lower	2.1			
		Marble-Schist	Super	9.5	BRZ	76-22	SBS	Upper	1.5	-0.75	1.60	2.35
S	12	Limestone	Hveem	12.5	TRZ	70-28	SB	Dual	4.0	-0.28	2.52	2.80
S	13	Granite	Super	12.5	ARZ	70-28	SB	Dual	4.0	-0.72	1.58	2.30

TABLE 4.5 Rut Depths on Track Tangents Calibrated to Wire Line Measurements and Corrected for Zero Traffic Deformation (thus, Induced Rutting) at Select Times

Initial	**Date:**	9/17/2000	9/24/2000	10/1/2000	10/29/2000	11/26/2000	4/8/2001	4/22/2001	6/10/2001	8/19/2001
Deform	**ESALs:**	**0**	**20,098**	**41,291**	**120,576**	**380,658**	**1,791,670**	**1,988,598**	**2,724,036**	**3,741,447**
0.17	**N1**	0.00	0.10	0.20	0.25	0.25	0.25	0.30	0.60	1.25
-0.15	**N2**	0.00	0.10	0.20	0.30	0.30	0.35	0.35	0.80	1.15
-0.08	**N3**	0.00	0.15	0.30	0.35	0.45	0.50	0.60	2.00	3.95
-0.11	**N4**	0.00	0.15	0.30	0.50	0.55	0.65	0.70	2.05	3.50
-0.99	**N5**	0.00	0.20	0.40	0.70	0.70	0.85	1.35	2.25	4.25
-0.60	**N6**	0.00	0.20	0.40	0.45	0.45	0.45	0.50	1.05	2.70
-0.72	**N7**	0.00	0.10	0.20	0.40	0.40	0.40	0.40	0.70	1.20
-0.46	**N8**	0.00	0.08	0.15	0.30	0.35	0.40	0.45	0.60	0.90
-0.19	**N9**	0.00	0.03	0.05	0.10	0.10	0.15	0.25	0.30	0.35
0.28	**N10**	0.00	0.05	0.10	0.10	0.10	0.15	0.15	0.25	0.40
-0.23	**N11**	0.00	0.10	0.20	0.35	0.35	0.35	0.35	0.75	1.00
-0.40	**N12**	0.00	0.05	0.10	0.15	0.15	0.15	0.25	1.10	1.60
-1.51	**N13**	0.00	0.08	0.15	0.25	0.25	0.55	0.55	1.60	2.25
0.03	**S1**	0.00	0.25	0.50	0.55	0.60	0.70	0.95	1.05	1.40
-0.14	**S2**	0.00	0.03	0.05	0.05	0.05	0.20	0.20	0.20	0.25
-0.08	**S3**	0.00	0.08	0.15	0.15	0.20	0.20	0.20	0.25	0.35
-0.04	**S4**	0.00	0.03	0.05	0.10	0.10	0.15	0.15	0.20	0.25
0.08	**S5**	0.00	0.03	0.05	0.10	0.10	0.15	0.15	0.20	0.45
-0.04	**S6**	0.00	0.15	0.30	0.40	0.45	0.45	0.45	0.90	1.35
-0.25	**S7**	0.00	0.20	0.40	0.45	0.55	0.55	0.55	1.10	2.00
0.15	**S8**	0.00	0.08	0.15	0.25	0.35	0.35	0.35	0.75	1.10
-0.74	**S9**	0.00	0.08	0.15	0.45	0.60	0.65	0.70	1.15	1.90
-0.36	**S10**	0.00	0.18	0.35	0.60	0.70	0.70	0.80	1.37	2.65
-0.75	**S11**	0.00	0.10	0.20	0.45	0.50	0.55	0.60	1.05	1.60
-0.28	**S12**	0.00	0.23	0.45	0.50	0.55	0.75	0.75	1.30	1.75
-0.72	**S13**	0.00	0.15	0.30	0.40	0.40	0.45	0.50	0.90	1.30

Date:	9/23/2001	10/21/2001	4/21/2002	6/9/2002	6/30/2002	7/28/2002	12/22/2002	**Final**	**Final**	**Corr**
ESALs:	**4,225,016**	**4,614,708**	**6,882,819**	**7,435,908**	**7,677,743**	**7,907,234**	**10,000,132**	**Wireline**	**6-Point**	**Factor**
N1	1.30	1.30	1.30	1.40	1.40	1.50	1.90	2.07	2.22	0.93
N2	1.25	1.25	1.30	1.40	1.45	1.55	2.15	2.00	1.88	1.06
N3	4.30	4.35	4.35	4.90	5.05	5.45	7.35	7.27	6.75	1.08
N4	3.70	3.80	3.80	3.85	3.90	4.30	5.40	5.29	4.58	1.16
N5	4.80	4.80	5.05	5.30	5.60	6.15	8.05	7.06	5.98	1.18
N6	2.80	2.80	2.85	3.05	3.05	3.35	4.40	3.80	3.38	1.12
N7	1.35	1.35	1.35	1.60	1.60	1.65	2.30	1.58	1.15	1.38
N8	1.00	1.10	1.10	1.10	1.10	1.15	1.35	0.89	0.82	1.09
N9	0.40	0.40	0.45	0.55	0.55	0.60	0.70	0.51	0.73	0.71
N10	0.45	0.45	0.45	0.50	0.50	0.60	0.65	0.93	1.10	0.84
N11	1.10	1.10	1.15	1.25	1.30	1.35	1.65	1.42	1.02	1.40
N12	1.75	1.80	1.80	2.05	2.05	2.10	2.50	2.10	1.79	1.17
N13	2.80	2.80	2.80	3.50	3.70	4.30	4.95	3.44	2.66	1.29
S1	1.45	1.50	1.55	1.55	1.55	1.55	1.80	1.83	1.67	1.10
S2	0.30	0.35	0.50	0.50	0.50	0.50	0.60	0.46	0.55	0.84
S3	0.40	0.40	0.40	0.40	0.45	0.50	0.60	0.52	0.72	0.72
S4	0.25	0.30	0.40	0.40	0.55	0.60	0.70	0.66	0.45	1.47
S5	0.45	0.45	0.45	0.45	0.50	0.50	0.60	0.68	0.93	0.73
S6	1.35	1.40	1.45	1.45	1.50	1.80	2.10	2.06	1.91	1.08
S7	2.15	2.15	2.15	2.30	2.40	2.85	3.55	3.30	3.87	0.85
S8	1.10	1.15	1.20	1.20	1.25	1.30	1.60	1.75	1.60	1.10
S9	1.90	1.90	2.10	2.15	2.20	2.30	2.75	2.01	1.08	1.86
S10	2.95	3.00	3.00	3.40	3.45	3.70	4.50	4.14	2.85	1.45
S11	1.65	1.70	1.75	1.80	1.85	2.05	2.35	1.60	1.37	1.17
S12	1.95	2.00	2.00	2.00	2.05	2.25	2.80	2.52	2.28	1.10
S13	1.45	1.50	1.50	1.50	1.50	2.10	2.30	1.58	1.18	1.34

TABLE 4.6 Summary of Distresses at End of Truck Traffic

Location		Description of Research in Experimental Sections								Initial Surface Mix Characteristics						Final Performance Characteristics				
Track Quad	Sec Num	Aggregate Blend Type	Design Method	Design NMA	Grad Type	Binder Grade	Binder Modifier	Lift Type	Design Thick	<#8 (%)	<#16 (%)	Plant AC (%)	Field VTM (%)	Lab VTM (%)	Lab VMA (%)	Induced Rut (mm)	CtrLine Crack	ΔIRI (in/mile)	ΔMTD (mm)	ΔFriction (x 100)
N	1	Slag/Lms	Super	12.5	ARZ	76-22	SBS	Dual	4.0	52	33	7.2	6.5	2.5	14.7	1.90		3.2	0.065	-30.5
N	2	Slag/Lms	Super	12.5	ARZ	76-22+	SBS	Dual	4.0	50	33	7.7	4.9	2.2	15.4	2.15		6.3	0.052	-23.3
N	3	Slag/Lms	Super	12.5	ARZ	67-22+	NA	Dual	4.0	51	33	7.5	6.3	3.2	15.4	7.35		-1.9	-0.134	-27.7
N	4	Slag/Lms	Super	12.5	ARZ	67-22	NA	Dual	4.0	52	35	7.1	7.8	4.3	15.0	5.40		-0.8	-0.013	-31.9
N	5	Slag/Lms	Super	12.5	BRZ	67-22+	NA	Dual	4.0	38	26	6.9	7.2	3.0	12.4	8.05		7.6	-0.353	-23.7
N	6	Slag/Lms	Super	12.5	BRZ	67-22	NA	Dual	4.0	37	25	6.5	4.2	3.3	12.6	4.40		-3.7	-0.289	-17.0
N	7	Slag/Lms	Super	12.5	BRZ	76-22+	SBR	Dual	4.0	36	24	6.9	5.9	2.0	12.6	2.30		-6.0	-0.084	-22.6
N	8	Slag/Lms	Super	12.5	BRZ	76-22	SBR	Dual	4.0	37	24	6.4	4.7	4.0	13.1	1.35	X	-7.2	0.021	-24.4
N	9	Slag/Lms	Super	12.5	BRZ	76-22	SBS	Dual	4.0	40	26	6.4	5.5	3.2	12.2	0.70		-16.0	-0.074	-23.1
N	10	Slag/Lms	Super	12.5	BRZ	76-22+	SBS	Dual	4.0	34	23	6.9	6.0	3.5	13.5	0.65		-10.9	-0.050	-25.4
N	11	Granite	Super	19.0	BRZ	67-22	NA	Lower	2.5											
		Granite	Super	12.5	TRZ	76-22	SBS	Upper	1.5	37	30	4.5	6.9	3.4	13.4	1.65		3.4	0.198	-25.6
N	12	Granite	Super	19.0	BRZ	67-22	NA	Lower	2.5											
		Granite	SMA	12.5	SMA	76-22	SBS	Upper	1.5	23	21	6.1	7.2	2.8	15.0	2.50		-9.8	-0.275	-11.0
N	13	Gravel	Super	19.0	BRZ	76-22	SBS	Lower	2.5											
		Gravel	SMA	12.5	SMA	76-22	SBS	Upper	1.5	25	23	6.7	7.8	4.0	15.4	4.95		-1.7	-0.322	-11.9
S	1	Granite	Super	19.0	BRZ	76-22	SBS	Lower	2.5											
		Granite	Super	12.5	BRZ	76-22	SBS	Upper	1.5	36	28	5.2	5.9	3.0	14.3	1.80		4.7	0.131	-17.4
S	2	Gravel	Super	19.0	BRZ	76-22	SBS	Lower	2.5											
		Gravel	Super	9.5	BRZ	76-22	SBS	Upper	1.5	41	29	6.3	6.0	4.7	16.2	0.60	X	4.0	0.065	-12.6
S	3	Limestone	Super	19.0	BRZ	76-22	SBS	Lower	2.5											
		Lms/Gravel	Super	9.5	BRZ	76-22	SBS	Upper	1.5	43	29	5.9	7.6	3.5	16.0	0.60		-6.0	0.042	-15.9
S	4	Lms/RAP	Super	19.0	ARZ	76-22	SBS	Lower	2.5											
		Limestone	Super	12.5	ARZ	76-22	SBS	Upper	1.5	46	33	5.3	6.4	2.2	13.5	0.70		-0.5	0.032	-15.4
S	5	Lms/Grv/RAP	Super	19.0	BRZ	76-22	SBS	Lower	2.5											
		Gravel	Super	12.5	TRZ	76-22	SBS	Upper	1.5	45	33	5.5	5.3	3.4	10.8	0.60		2.6	0.114	-20.2
S	6	Lms/RAP	Super	12.5	ARZ	67-22	NA	Dual	4.0	53	41	6.6	7.1	4.5	14.2	2.10		-2.5	0.074	-13.9
S	7	Lms/RAP	Super	12.5	BRZ	67-22	NA	Dual	4.0	34	25	6.9	6.7	3.3	13.9	3.55		-3.7	-0.126	-22.7
S	8	Marble-Schist	Super	19.0	BRZ	67-22	NA	Lower	2.1											
		Marble-Schist	Super	12.5	BRZ	76-22	SBS	Upper	1.5	38	25	4.2	8.5	2.7	13.5	1.60		8.4	0.072	-19.6
S	9	Granite	Super	12.5	BRZ	67-22	NA	Dual	3.0	36	27	4.8	7.0	3.7	14.9	2.75		-7.3	0.143	-17.3
S	10	Granite	Super	12.5	ARZ	67-22	NA	Dual	3.0	52	38	5.3	6.7	3.2	15.8	4.50		-1.1	0.205	-14.8
S	11	Marble-Schist	Super	19.0	BRZ	67-22	NA	Lower	2.1											
		Marble-Schist	Super	9.5	BRZ	76-22	SBS	Upper	1.5	47	30	4.2	6.7	3.1	13.8	2.35		11.5	0.203	-19.5
S	12	Limestone	Hveem	12.5	TRZ	70-28	SB	Dual	4.0	46	32	4.7	5.7	3.8		2.80		5.8	0.119	-20.7
S	13	Granite	Super	12.5	ARZ	70-28	SB	Dual	4.0	50	37	5.3	5.8	4.8	14.7	2.30	X	9.7	0.186	2.6

TABLE 4.7 Analysis of Variance of Track Tangent Profile Results

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F Test Statistic	Probability
Track Test Sections	25	775.946	31.038	111.37	0.000
Error	208	57.969	0.279		
Total	233	833.915			

TABLE 4.8 Statistical Tests for Differences in Rutting Between Sections

Two Sections Involved In Direct Comparison (Avg of 9 Profiles)						95% Confidence Interval		Conf Interv	Statistical	Tukey-Kramer Method
Section	Avg	Std Dev	Section	Avg	Std Dev	via Equation 4.1		Cont Zero?	Decision	Conclusion in Research Comparison
N1	2.07	0.55	N2	2.00	0.44	-0.853	1.002	Y	Cannot Reject Ho	No Statistical Difference Between N1 and N2
N3	7.27	0.60	N4	5.29	0.41	1.047	2.902	N	Reject Ho	Less Rutting in N4 than N3
N5	7.06	0.95	N6	3.80	0.60	2.336	4.191	N	Reject Ho	Less Rutting in N6 than N5
N7	1.58	0.42	N8	0.89	0.28	-0.240	1.615	Y	Cannot Reject Ho	No Statistical Difference Between N7 and N8
N9	0.51	0.22	N10	0.93	0.14	-1.339	0.516	Y	Cannot Reject Ho	No Statistical Difference Between N9 and N10
N4	5.29	0.41	N6	3.80	0.60	0.568	2.423	N	Reject Ho	Less Rutting in N6 than N4
N1	2.07	0.55	N9	0.51	0.22	0.628	2.483	N	Reject Ho	Less Rutting in N9 than N1
N8	0.89	0.28	N9	0.51	0.22	-0.552	1.303	Y	Cannot Reject Ho	No Statistical Difference Between N8 and N9
N11	1.42	0.35	N12	2.10	0.31	-1.609	0.246	Y	Cannot Reject Ho	No Statistical Difference Between N11 and N12
N13	3.44	0.56	S2	0.46	0.14	2.052	3.906	N	Reject Ho	Less Rutting in S2 than N13
S1	1.83	0.66	S8	1.75	0.58	-0.852	1.003	Y	Cannot Reject Ho	No Statistical Difference Between S1 and S8
S8	1.75	0.58	S11	1.60	0.49	-0.772	1.083	Y	Cannot Reject Ho	No Statistical Difference Between S8 and S11
S2	0.46	0.14	S3	0.52	0.19	-0.983	0.872	Y	Cannot Reject Ho	No Statistical Difference Between S2 and S3
S4	0.66	0.52	S5	0.68	0.33	-0.950	0.905	Y	Cannot Reject Ho	No Statistical Difference Between S4 and S5
S6	2.06	0.27	S7	3.30	0.31	-2.171	-0.316	N	Reject Ho	Less Rutting in S6 than S7
S9	2.01	0.45	S10	4.14	1.08	-3.063	-1.208	N	Reject Ho	Less Rutting in S9 than S10
S12	2.52	0.32	S13	1.58	0.99	0.014	1.869	N	Reject Ho	Less Rutting in S13 than S12

Note: No significant difference when 95% confidence interval contains zero in Tukey-Kramer statistical method

TABLE 4.9 Results from Testing During Gyratory Compaction Used for Model Development

Track	Test	Lift	Standard QC Samples in SGC				Samples Compacted in 1° GTM (Everything After Oil GSI Reheated with Air Cell)					Lateral Pressure (kPa)					
Quad	Sec	Type	dH[illegible] (mm)	dH[illegible] (mm)	St_{max}	N-St_{max}	Oil GSI	GSI @ Field VTM	Revs to Field VTM	GSI Long Term	Strain Rate Long Term	1	5	10	30	100	150
N	1	Bind + Surf	27.1	1.7	0.892	85	1.13	0.96	32	1.41	0.0264						
N	2	Bind + Surf	26.2	1.6	0.883	38	1.13	1.00	38	1.49	0.0235						
N	3	Bind + Surf	26.3	2.0	0.871	62	1.11	0.98	52	1.08	0.0208	206	262	277	310	346	380
N	4	Bind + Surf	26.8	2.0	0.822	163	1.06	0.96	98	1.00	0.0181						
N	5	Bind + Surf	NA	NA	NA	NA	1.07	0.97	57	1.19	0.0210	255	271	285	312	290	291
N	6	Bind + Surf	28.5	2.1	0.779	62	1.05	1.02	130	1.08	0.0178						
N	7	Bind + Surf	29.7	2.0	0.770	62	1.15	1.02	72	1.18	0.0194						
N	8	Bind + Surf	31.9	2.2	0.774	93	0.97	1.35	495	1.35	0.0000						
N	9	Bind + Surf	30.9	2.3	0.746	296	1.00	1.06	310	1.03	0.0097						
N	10	Bind + Surf	26.9	1.9	0.712	43	1.05	0.99	51	1.24	0.0214						
N	11	Surface	21.0	1.2	0.847	90	1.09	0.95	47	1.07	0.0217						
		Binder	21.0	1.2	0.877	46	1.19	0.99	121	1.01	0.0141						
N	12	Surface	28.2	1.6	0.836	39	1.25	1.00	31	1.03	0.0242						
		Binder	21.0	1.2	0.877	46	1.19	0.99	121	1.01	0.0141						
N	13	Surface	22.9	1.4	0.859	62	0.98	0.86	73	0.96	0.0170						
		Binder	24.5	1.4	0.967	299	0.99	0.96	495	1.01	0.0032						
S	1	Surface	23.9	1.4	0.888	90	1.10	1.05	144	1.03	0.0113						
		Binder	28.8	1.7	0.871	74	1.00	1.15	296	1.06	0.0081						
S	2	Surface	26.0	1.6	0.867	288	1.00	0.99	215	0.98	0.0065						
		Binder	24.5	1.4	0.967	299	0.99	0.96	495	1.01	0.0032						
S	3	Surface	28.1	1.8	0.888	43	1.07	0.99	134	0.99	0.0145						
		Binder	27.1	1.9	0.809	51	1.06	0.96	56	1.15	0.0168						
S	4	Surface	23.9	1.4	0.904	59	1.04	1.00	126	1.02	0.0129						
		Binder	19.3	1.1	0.960	35	0.98	1.02	495	1.01	0.0032						
S	5	Surface	20.6	1.2	0.951	15	1.02	1.09	255	1.01	0.0065						
		Binder	25.5	1.5	0.901	295	0.99	0.85	495	0.98	0.0032						
S	6	Bind + Surf	19.8	1.2	0.938	296	1.01	0.92	188	0.98	0.0065						
S	7	Bind + Surf	29.3	2.3	0.895	148	1.12	1.01	270	1.03	0.0081						
S	8	Surface	27.7	1.3	0.872	31	1.49	0.98	27	1.20	0.0270						
		Binder	28.2	1.9	0.831	152	1.10	1.06	160	1.06	0.0152						
S	9	Bind + Surf	23.7	1.4	0.900	129	1.07	0.96	53	1.26	0.0189	173	232	267	327	314	335
S	10	Bind + Surf	19.7	1.1	0.889	31	1.06	1.00	89	1.04	0.0150	200	226	239	249	211	215
S	11	Surface	24.9	1.3	0.985	35	1.41	0.99	48	1.05	0.0180						
		Binder	28.2	1.9	0.831	152	1.10	1.06	160	1.06	0.0152						
S	12	Bind + Surf	23.2	1.5	0.829	70	1.03	1.05	132	1.06	0.0145						
S	13	Bind + Surf	20.2	1.2	0.982	171	1.03	1.04	355	1.04	0.0000						

TABLE 4.10 Simulative Test Results Used for Model Development ("4k" and "8k" Refer to Induced Rutting after 4,000 and 8,000 Load Cycles, Respectively)

Track	Test	Lift	APA		Hamburg		RLWT		Wessex	
Quad	Sec	Type	4k (mm)	8k (mm)	4k (mm)	8k (mm)	4k (mm)	8k (mm)	4k (mm)	8k (mm)
N	1	Bind + Surf	1.83	2.14	3.53	3.65	1.23	1.63	1.51	1.67
N	2	Bind + Surf	1.56	1.92	2.77	3.09	1.19	1.59	2.86	3.15
N	3	Bind + Surf	2.45	3.26	3.85	4.34	2.59	4.11	2.39	2.68
N	4	Bind + Surf	3.47	4.28	4.98	5.74	1.98	2.55	3.24	3.68
N	5	Bind + Surf	2.14	2.68	3.85	4.13	1.61	2.07	2.10	2.47
N	6	Bind + Surf	5.89	6.54	5.31	5.87	2.45	3.12	3.01	3.50
N	7	Bind + Surf	2.20	2.50	1.85	2.04	1.06	1.19	0.85	1.01
N	8	Bind + Surf	1.16	1.51	2.24	2.50	1.20	1.40	1.41	1.67
N	9	Bind + Surf	2.13	2.42	3.17	3.95	0.91	1.04	1.97	2.27
N	10	Bind + Surf	1.85	2.10	4.41	4.71	1.69	1.96	2.67	2.88
N	11	Surface	1.51	1.70	2.44	2.63	0.68	0.74	1.62	1.89
		Binder	2.10	2.39	2.23	2.43	0.88	0.97	1.34	1.47
N	12	Surface	1.45	1.66	2.39	2.67	0.80	0.88	1.26	1.42
		Binder	2.10	2.39	2.23	2.43	0.88	0.97	1.34	1.47
N	13	Surface	2.92	3.30	2.71	3.53	2.22	2.68	2.30	2.54
		Binder	0.78	0.91	0.86	0.96	0.72	0.80	1.22	1.31
S	1	Surface	1.66	1.86	3.12	4.61	0.83	0.97	2.14	2.44
		Binder	1.08	1.20	1.61	1.81	0.81	0.91	1.61	1.85
S	2	Surface	1.29	1.43	2.31	2.62	0.70	0.81	1.57	1.96
		Binder	0.78	0.91	0.86	0.96	0.72	0.80	1.22	1.31
S	3	Surface	1.09	1.34	2.31	2.51	1.06	1.13	1.16	1.30
		Binder	1.12	1.17	2.37	2.56	0.98	1.07	1.08	1.17
S	4	Surface	1.07	1.18	1.82	2.31	0.79	0.86	0.91	1.01
		Binder	1.87	2.16	1.96	2.14	1.25	1.34	0.91	1.01
S	5	Surface	2.49	3.17	4.95	5.95	1.69	2.07	0.84	1.09
		Binder	0.89	1.51	1.71	1.87	0.80	0.86	0.84	0.98
S	6	Bind + Surf	1.41	1.70	3.01	3.40	0.90	1.05	0.94	1.08
S	7	Bind + Surf	1.98	2.60	3.07	3.33	1.36	1.72	0.87	1.01
S	8	Surface	1.07	1.17	1.92	2.04	0.80	0.87	1.01	1.15
		Binder	1.43	1.73	2.05	2.32	1.26	1.39	1.64	2.01
S	9	Bind + Surf	1.71	2.15	3.57	4.75	1.71	2.40	3.11	3.60
S	10	Bind + Surf	2.00	2.55	3.09	3.51	1.96	2.61	3.70	4.41
S	11	Surface	0.96	1.12	1.56	1.66	0.84	0.88	1.27	1.40
		Binder	1.43	1.73	2.05	2.32	1.26	1.39	1.64	2.01
S	12	Bind + Surf	1.37	1.58	1.56	1.65	1.46	1.66	0.87	0.98
S	13	Bind + Surf	1.14	1.24	1.61	1.77	1.24	1.34	2.04	2.34

TABLE 4.11 Fundamental Test Results Used for Model Development

Track	Test	Lift	Confined Cyclic (RLCC)				Dyn Mod by Hz (ksi)					SST (%)		Seismic Stiffness (ksi)	
Quad	Sec	Type	Intercept (us)	COV	Slope (us/Cyc)	COV	10	5	1	0.5	0.1	RSCH	COV	Pills	Cores
N	1	Bind + Surf	8100	18.2%	0.480	51.6%	0.196	0.158	0.113	0.099	0.085	1.769	12.3%	856	860
N	2	Bind + Surf	7444	12.0%	3.004	54.7%	0.167	0.135	0.096	0.085	0.076	0.945	34.8%	869	850
N	3	Bind + Surf	7926	47.9%	2.216	84.3%	0.211	0.142	0.094	0.081	0.068	2.991	18.1%	901	818
N	4	Bind + Surf	6379	18.2%	10.086	3.1%	0.202	0.156	0.102	0.087	0.073	2.522	34.6%	910	858
N	5	Bind + Surf	6203	0.8%	10.201	4.3%	0.251	0.196	0.128	0.109	0.092	1.428	24.1%	1011	877
N	6	Bind + Surf	10918	32.3%	7.739	32.9%	0.196	0.159	0.110	0.093	0.075	3.624		935	938
N	7	Bind + Surf	6112	51.3%	1.009	80.4%	0.279	0.219	0.148	0.128	0.110	1.417	19.8%	928	809
N	8	Bind + Surf	6549	12.5%	0.415	57.3%	0.262	0.211	0.140	0.119	0.099	1.540	24.8%	871	766
N	9	Bind + Surf	6086	72.4%	1.081	90.1%	0.310	0.249	0.171	0.148	0.121	1.837	26.2%	923	802
N	10	Bind + Surf	14128	29.0%	1.962	95.7%	0.175	0.139	0.093	0.081	0.068	2.319	4.5%	830	846
N	11	Surface	5081	62.8%	0.253	58.0%	0.190	0.154	0.103	0.088	0.074	0.358	25.5%	938	851
		Binder	5979	57.7%	2.534	101.2%	0.220	0.165	0.114	0.094	0.079	0.812	23.9%	NA	NA
N	12	Surface	8911	37.5%	0.689	59.1%	0.166	0.125	0.082	0.071	0.061	0.302	62.2%	992	967
		Binder	5979	57.7%	2.534	101.2%	0.220	0.165	0.114	0.094	0.079	0.812	23.9%	NA	NA
N	13	Surface	10036	13.0%	13.184	26.2%	0.126	0.105	0.067	0.058	0.047	1.011	3.6%	871	827
		Binder	6710	49.1%	0.693	44.3%	0.374	0.261	0.163	0.128	0.104	0.716	11.2%	NA	NA
S	1	Surface	4122	51.7%	1.065	114.4%	0.194	0.150	0.096	0.081	0.068	0.538	15.7%	948	781
		Binder	7732	16.9%	1.818	120.1%	0.146	0.113	0.075	0.067	0.060	0.259	20.2%	NA	NA
S	2	Surface	6710	20.0%	0.579	69.0%	0.223	0.168	0.101	0.086	0.070	0.232	31.6%	892	619
		Binder	6710	49.1%	0.693	44.3%	0.374	0.261	0.163	0.128	0.104	0.716	11.2%	NA	NA
S	3	Surface	5352	6.2%	0.500	55.0%	0.235	0.186	0.121	0.103	0.087	0.171	20.3%	935	792
		Binder	8352	15.9%	0.842	13.3%	0.174	0.139	0.092	0.078	0.066	0.872	9.4%	NA	NA
S	4	Surface	7070	46.4%	0.896	62.2%	0.185	0.147	0.098	0.085	0.071	0.793	7.1%	913	708
		Binder	3763	38.3%	0.813	114.7%	0.238	0.185	0.126	0.107	0.088	1.259	30.1%	NA	NA
S	5	Surface	4371	19.4%	2.539	93.0%	0.140	0.112	0.078	0.067	0.057	1.999	4.8%	862	793
		Binder	3773	43.2%	0.530	69.7%	0.288	0.220	0.134	0.110	0.083	0.396	8.2%	NA	NA
S	6	Bind + Surf	3768	42.8%	0.376	186.9%	0.247	0.194	0.127	0.108	0.089	1.859	32.1%	896	728
S	7	Bind + Surf	6731	59.3%	2.038	77.2%	0.230	0.175	0.115	0.097	0.082	2.789	24.9%	898	835
S	8	Surface	4693	3.6%	0.276	20.1%	0.226	0.178	0.123	0.108	0.099	0.670	49.0%	1139	1016
		Binder	3109	74.6%	2.817	110.1%	0.215	0.164	0.100	0.086	0.066	1.890	19.9%	NA	NA
S	9	Bind + Surf	4719	29.4%	10.322	38.8%	0.142	0.104	0.063	0.055	0.047	1.982	9.9%	1066	890
S	10	Bind + Surf	5046	53.4%	8.867	6.8%	0.127	0.095	0.065	0.057	0.051	2.134	10.4%	922	935
S	11	Surface	2330	28.9%	0.180	55.9%	0.267	0.213	0.142	0.123	0.101	0.735	13.2%	1047	1052
		Binder	3109	74.6%	2.817	110.1%	0.215	0.164	0.100	0.086	0.066	1.890	19.9%	NA	NA
S	12	Bind + Surf	7518	27.3%	0.832	66.1%	0.311	0.240	0.162	0.137	0.112	1.238	41.7%	969	881
S	13	Bind + Surf	6922	42.6%	0.367	60.2%	0.131	0.099	0.066	0.057	0.049	0.750	63.8%	805	718

FIGURE 4.1 Auburn University's High Speed Inertial Profiler Used to Make Continuous 3-Point Rut Measurements.

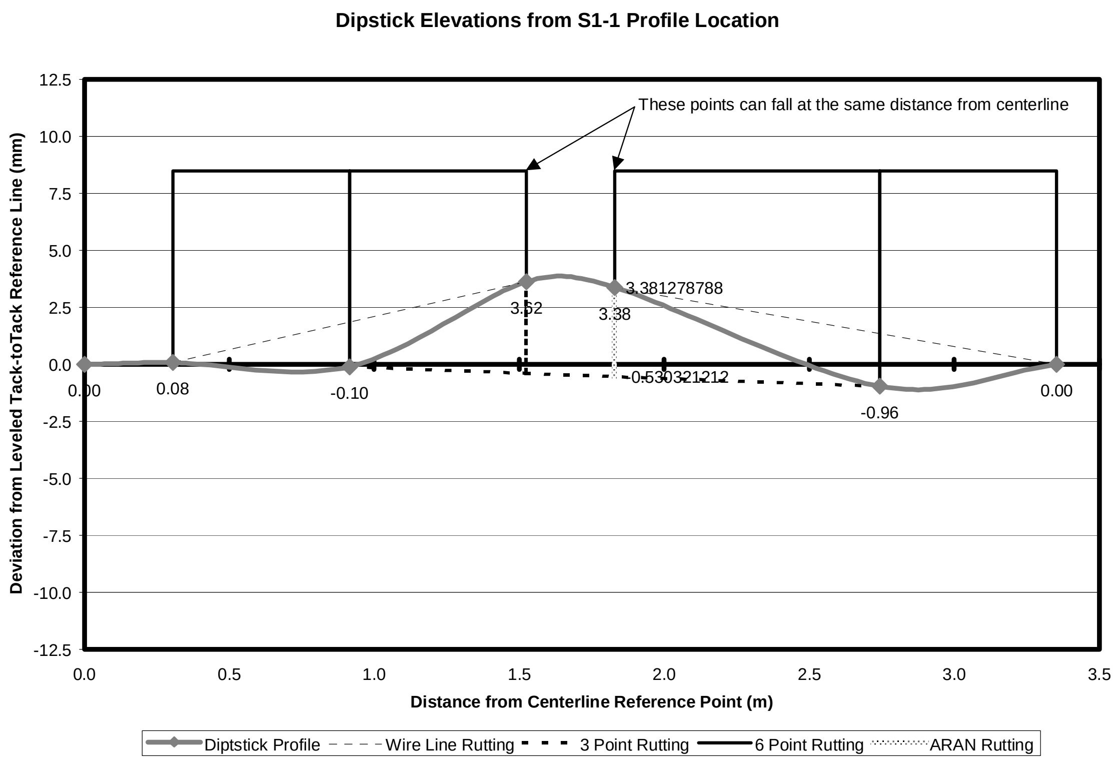

FIGURE 4.2 Geometry of Various Methods of Measuring and Computing Rut Depths.

FIGURE 4.3 Dipstick Elevation Measurements Along Transverse Test Location.

FIGURE 4.4 Stringline Measurements After Completion of Traffic.

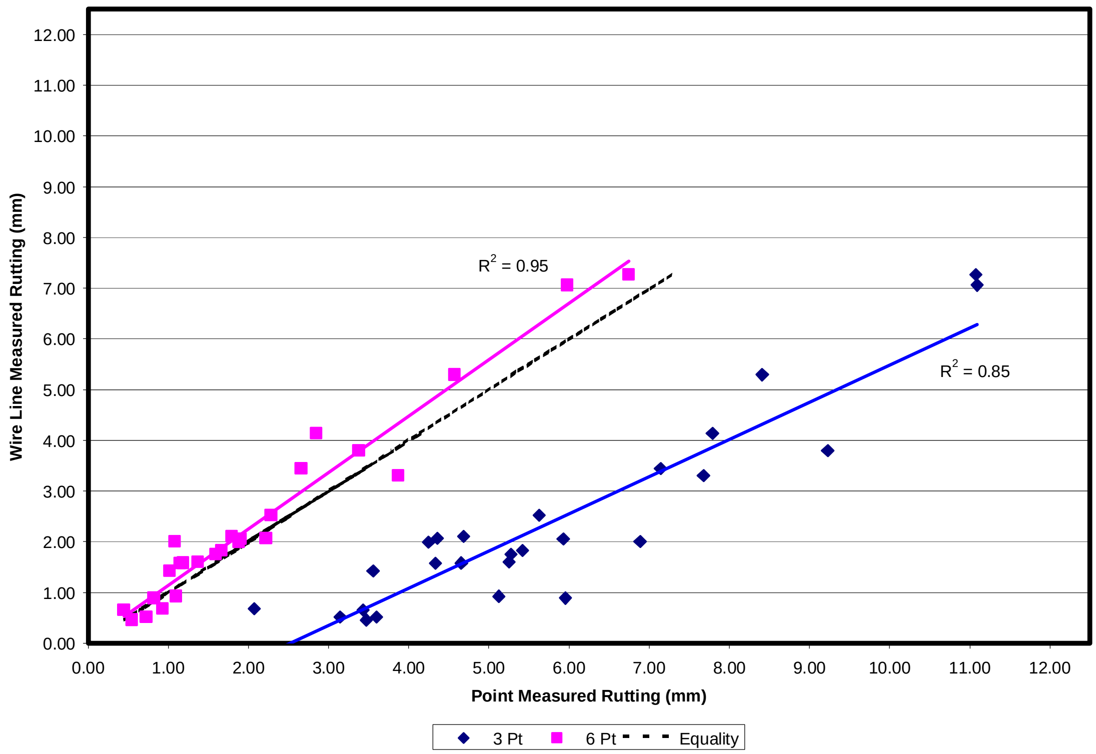

FIGURE 4.5 Graphic Comparison of Final 3- and 6-Point Rutting versus Final Wire Line Rutting (Same Profile, Different Algorithms).

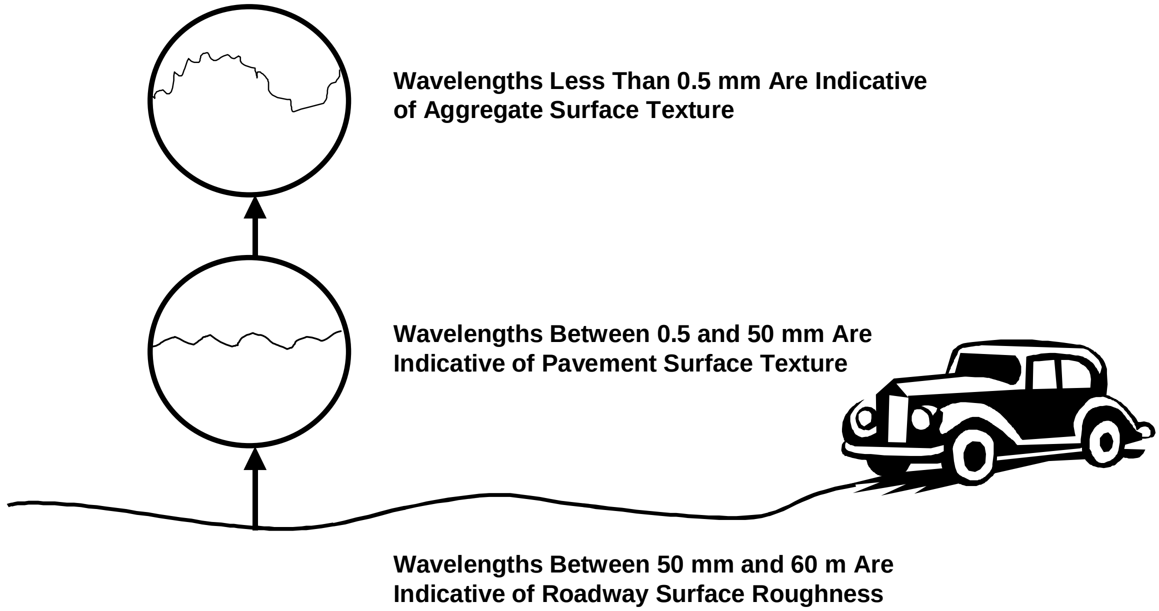

FIGURE 4.6 Schematic Illustration of Significance of Measured Wavelength in Roadway Surface Testing (Cenek, 2000).

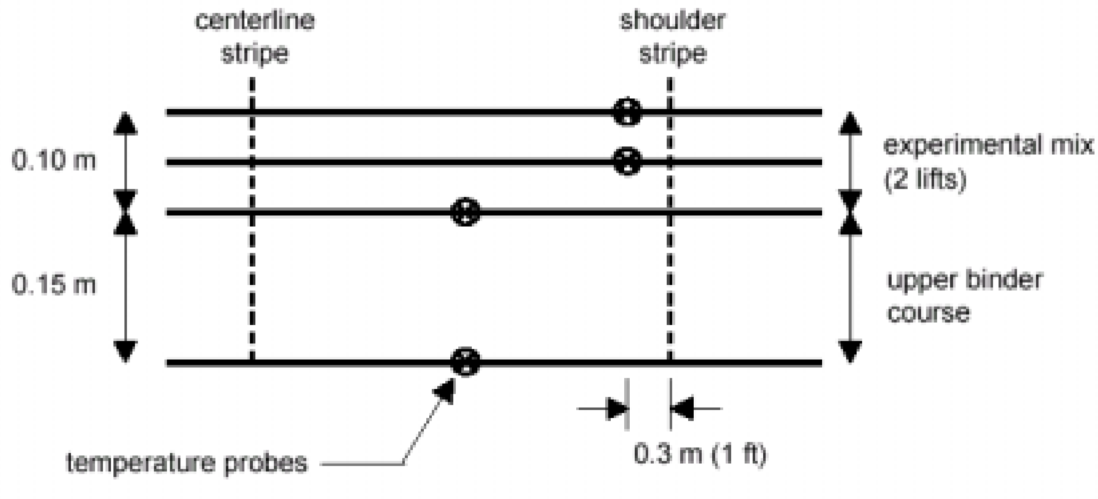

FIGURE 4.7 Multi-Depth Temperature Measurement Configuration.

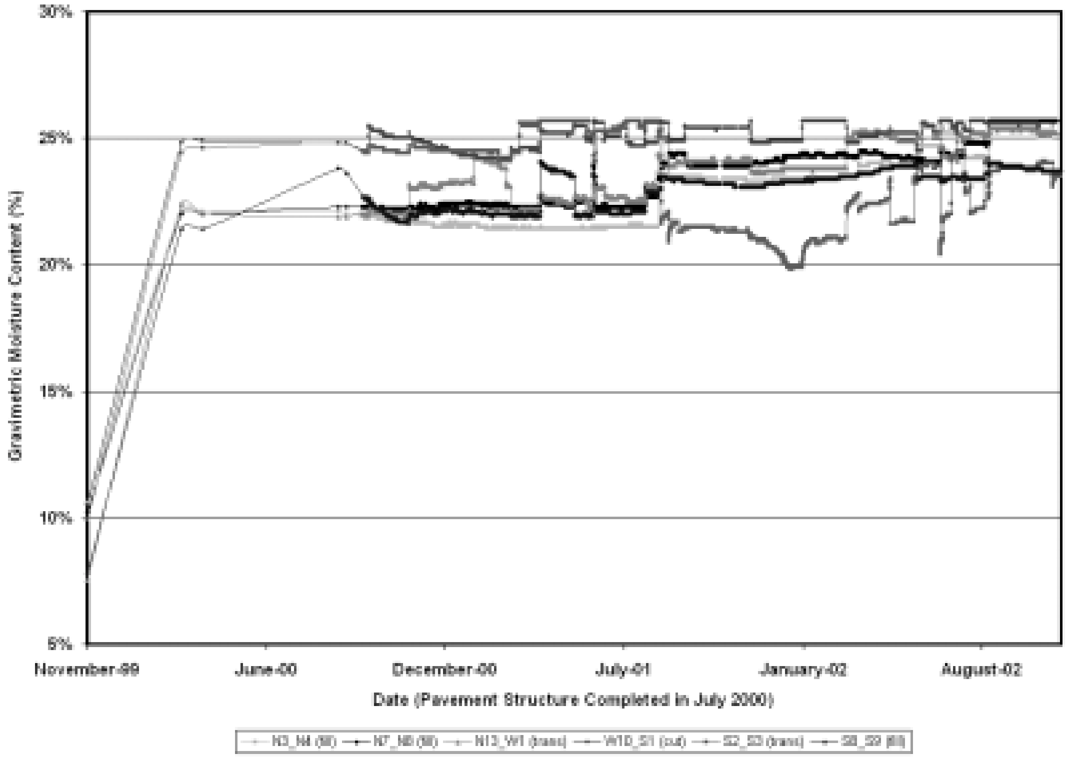

FIGURE 4.8 Track Tangent Moisture Contents from Construction to Completion of Traffic.

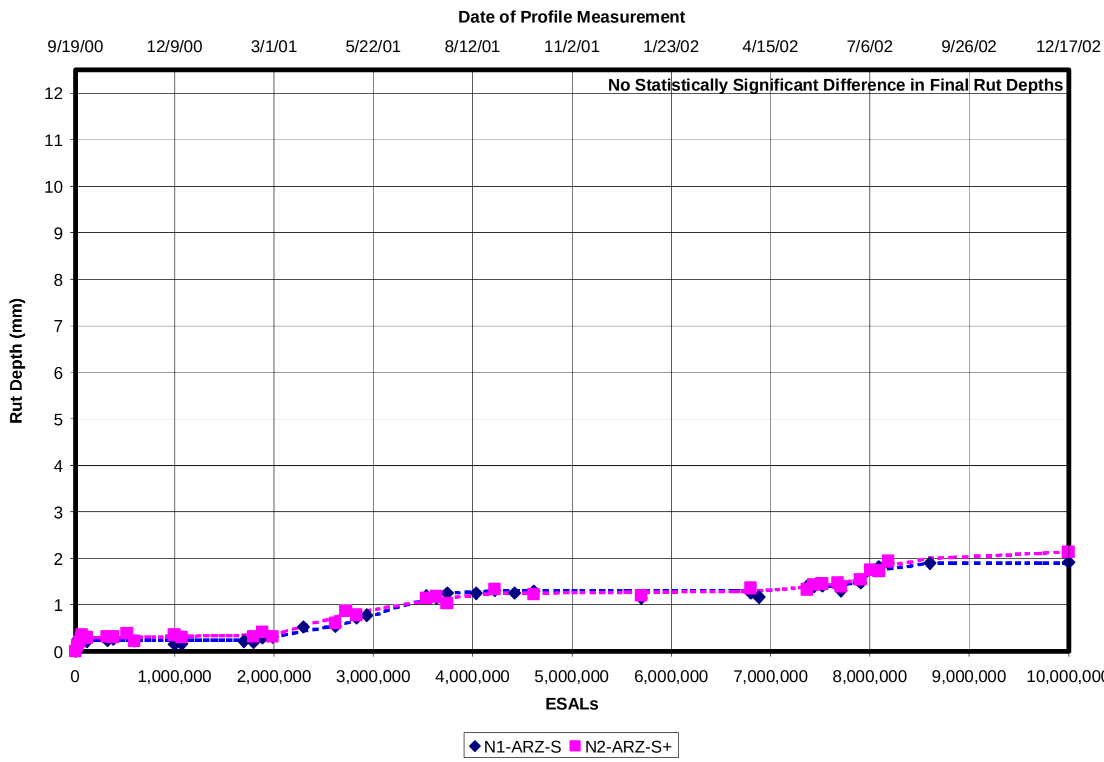

FIGURE 4.9 Graphical Comparison of Fine-Graded Mix Blended with SBS-Modified Binder at Optimum (N1) and Optimum Plus 0.5 Percent (N2).

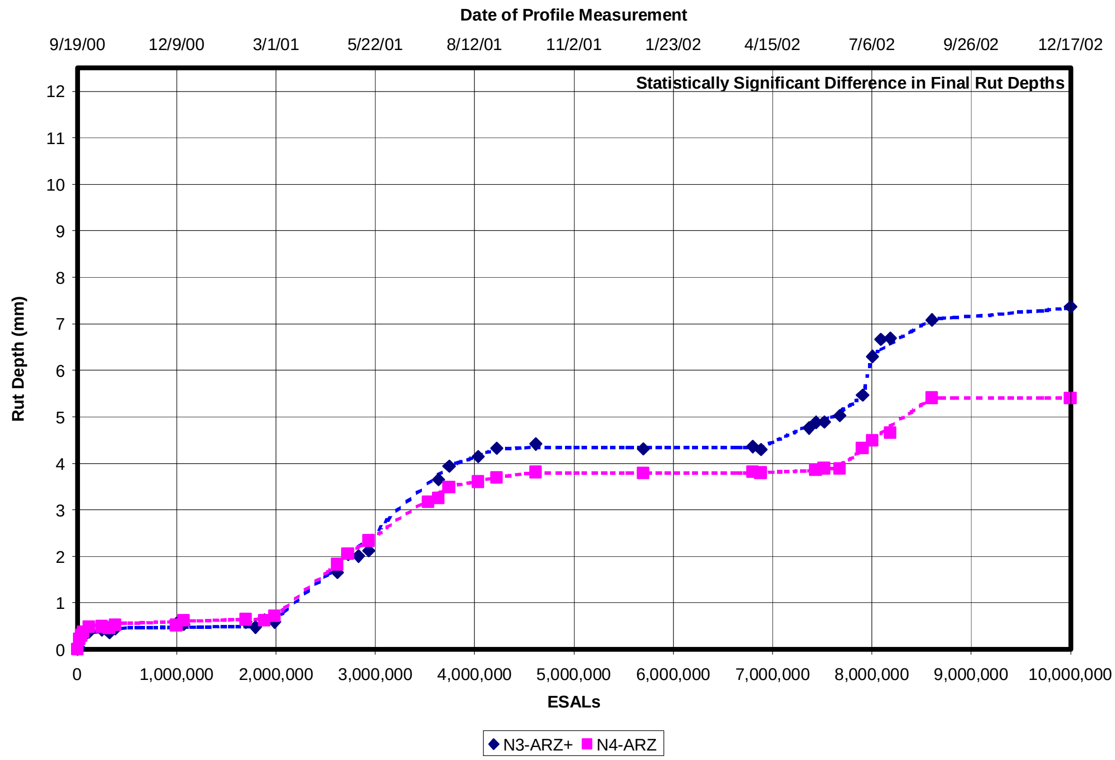

FIGURE 4.10 Graphical Comparison of Fine-Graded Mix Blended with Unmodified Binder at Optimum (N4) and Optimum Plus 0.5 Percent (N3).

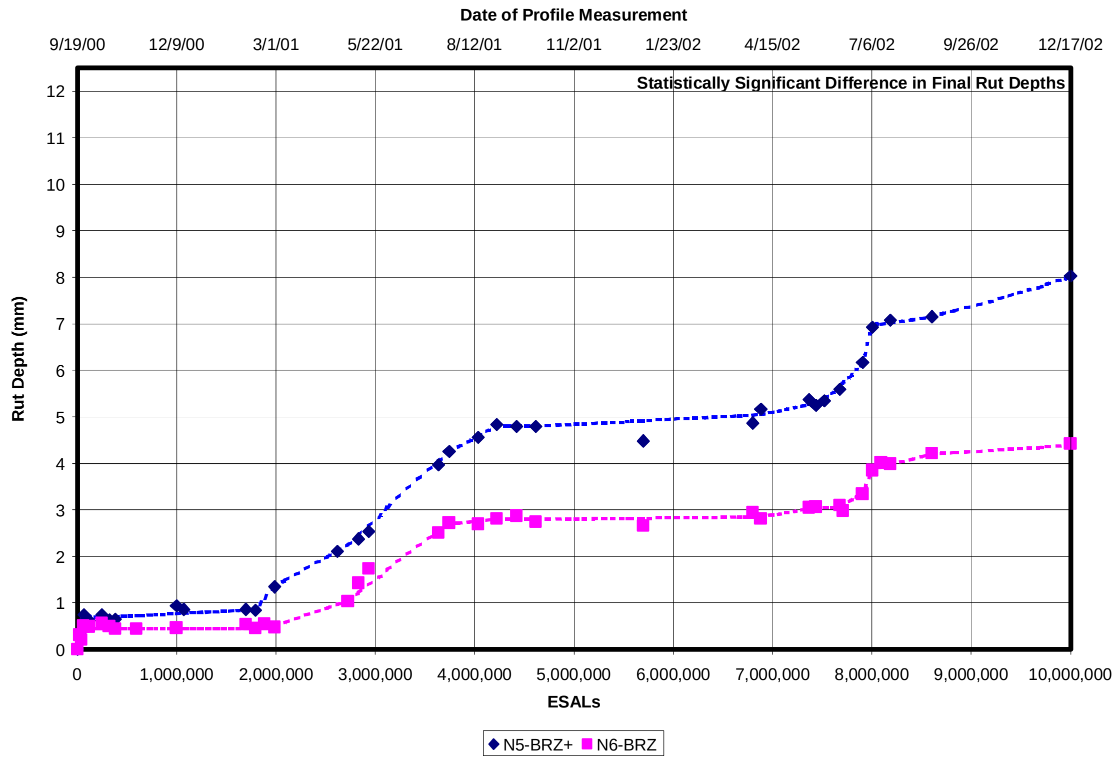

FIGURE 4.11 Graphical Comparison of Coarse-Graded Mix Blended with Unmodified Binder at Optimum (N6) and Optimum Plus 0.5 Percent (N5).

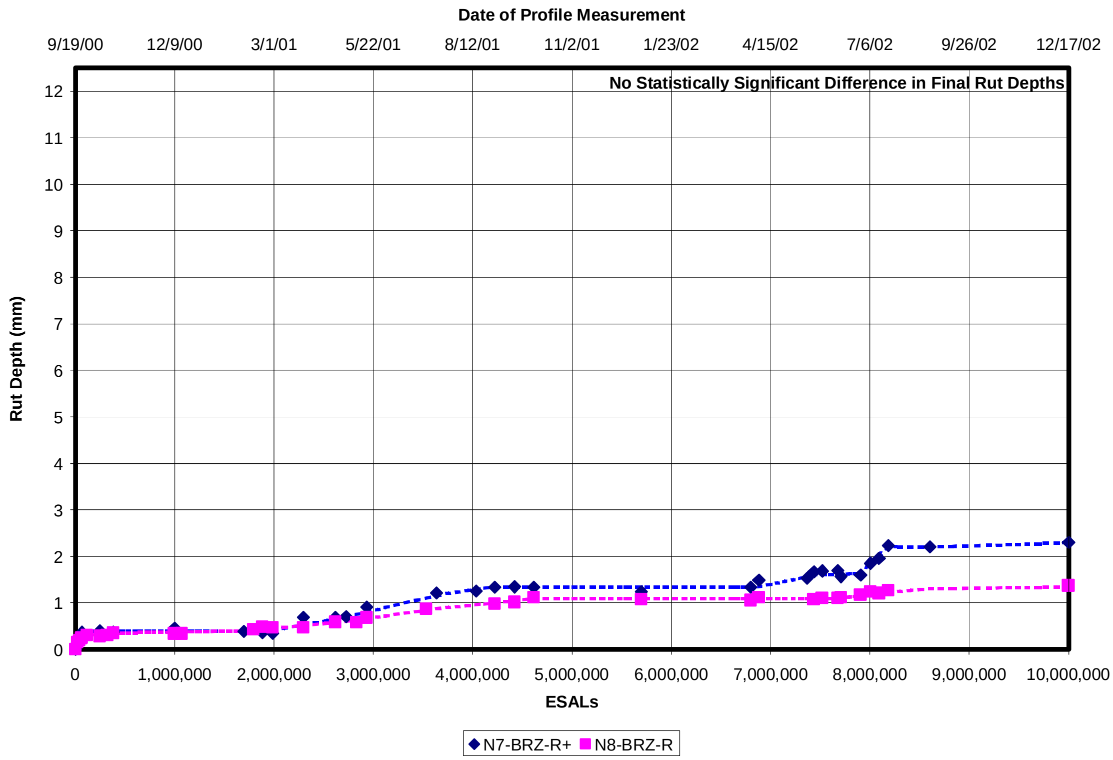

FIGURE 4.12 Graphical Comparison of Coarse-Graded Mix Blended with SBR-Modified Binder at Optimum (N8) and Optimum Plus 0.5 Percent (N7).

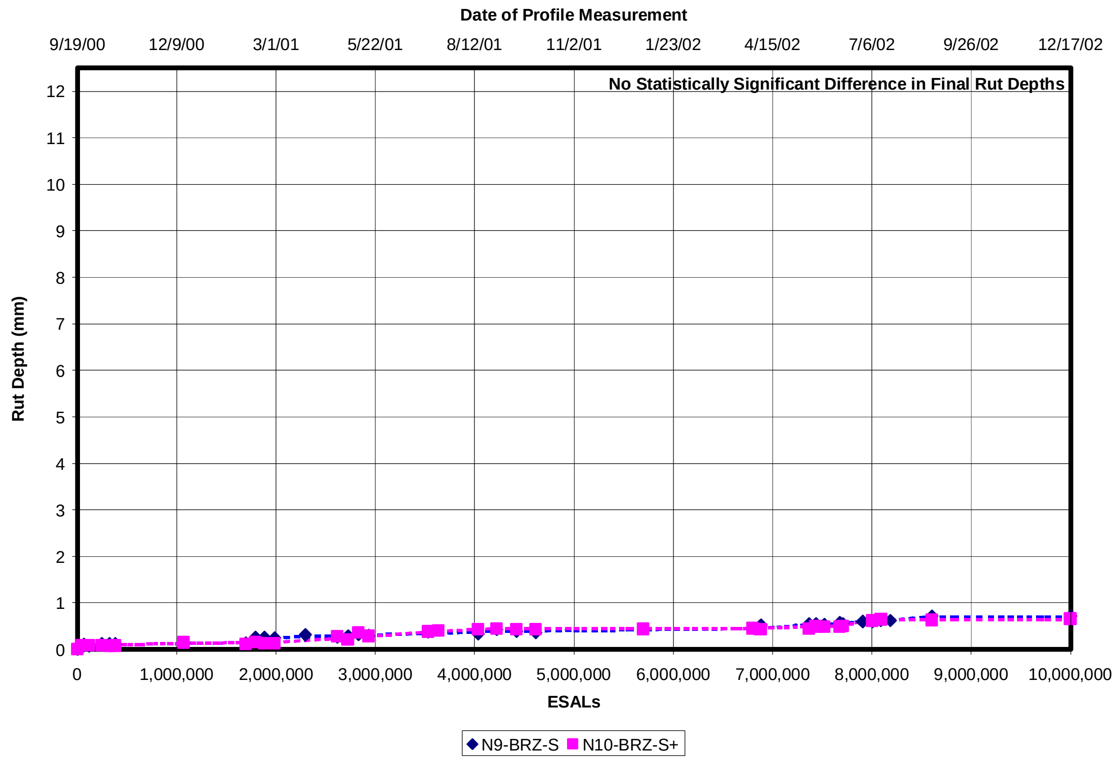

FIGURE 4.13 Graphical Comparison of Coarse-Graded Mix Blended with SBS-Modified Binder at Optimum (N9) and Optimum Plus 0.5 Percent (N10).

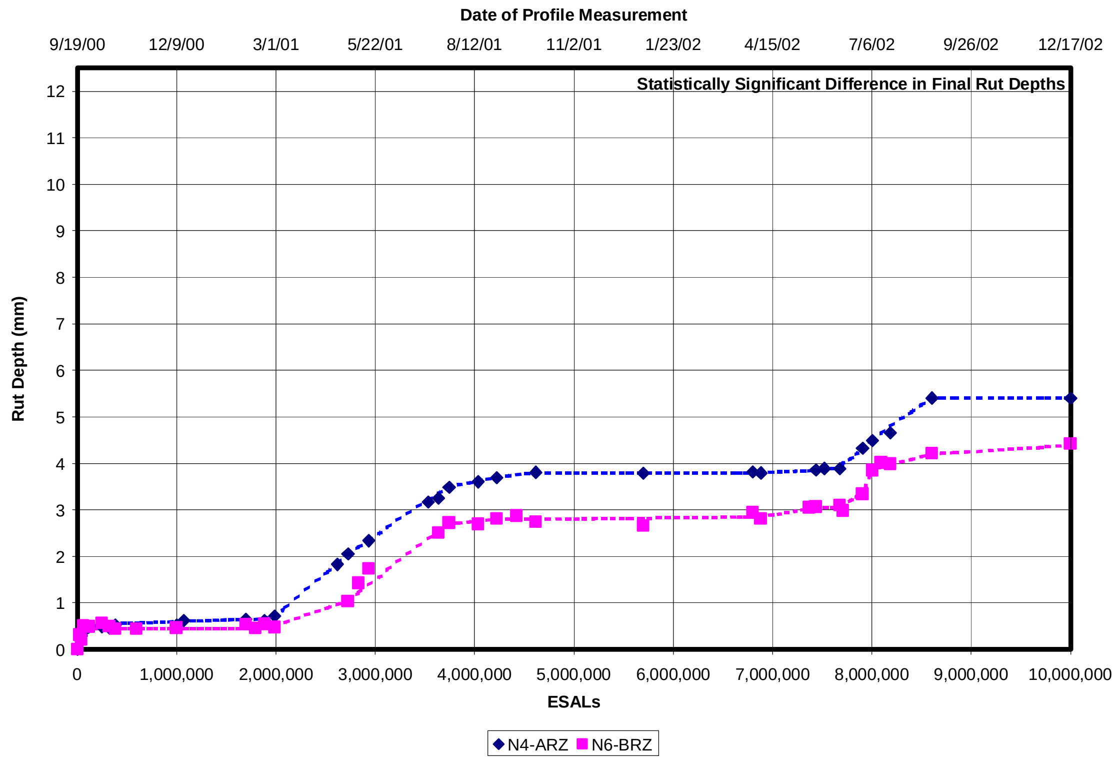

FIGURE 4.14 Graphical Comparison of Coarse-Graded (N6) versus Fine-Graded (N4) Mix Blended with Unmodified Binder at Optimum.

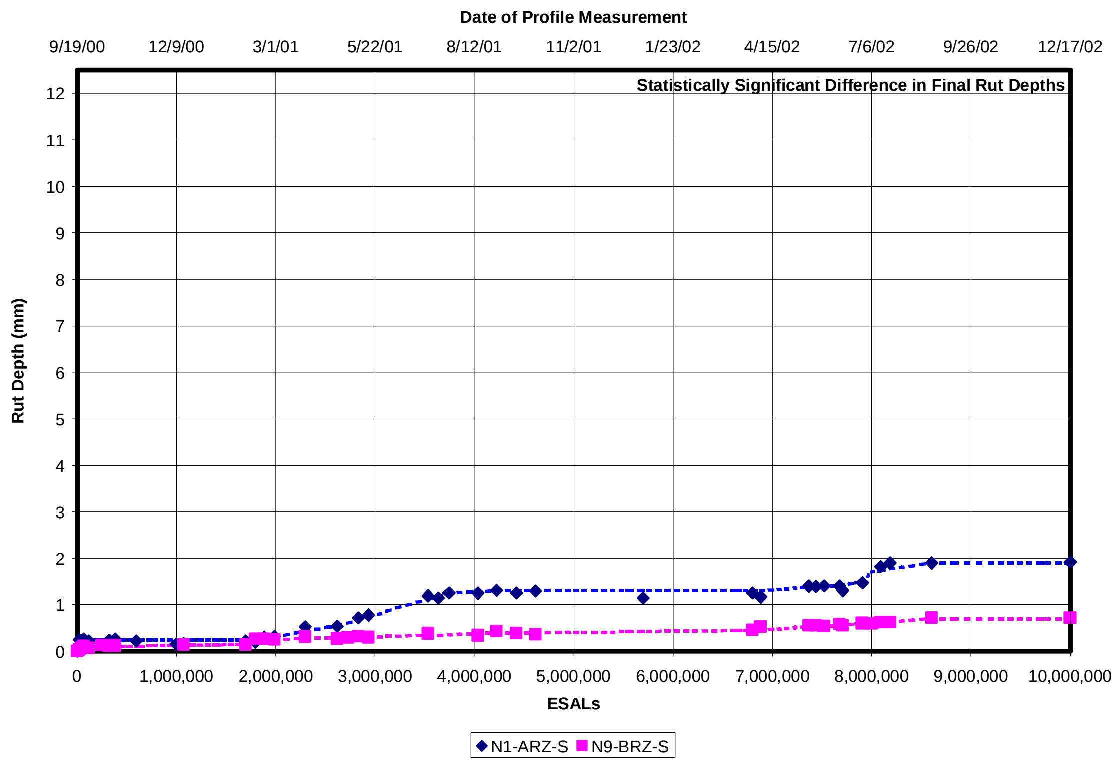

FIGURE 4.15 Graphical Comparison of Coarse-Graded (N9) versus Fine-Graded (N1) Mix Blended with SBS-Modified Binder at Optimum.

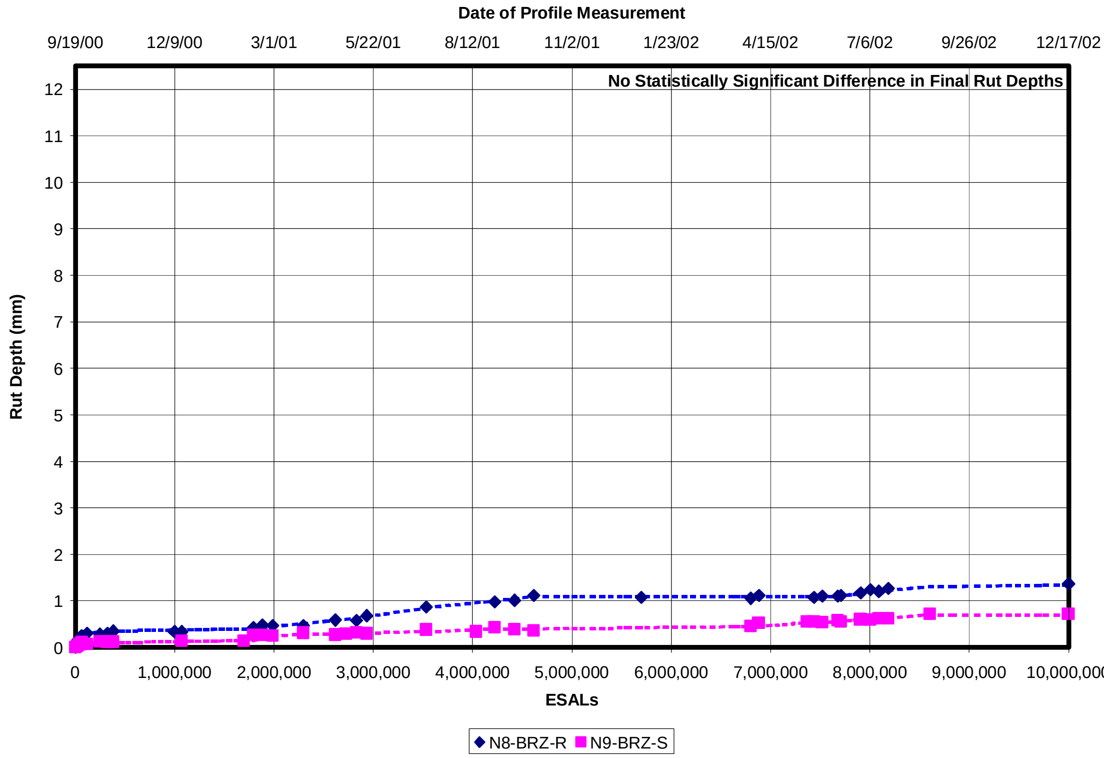

FIGURE 4.16 Graphical Comparison of Coarse-Graded Mix Blended with SBS-Modified (N9) versus SBR-Modified (N8) Binder at Optimum.

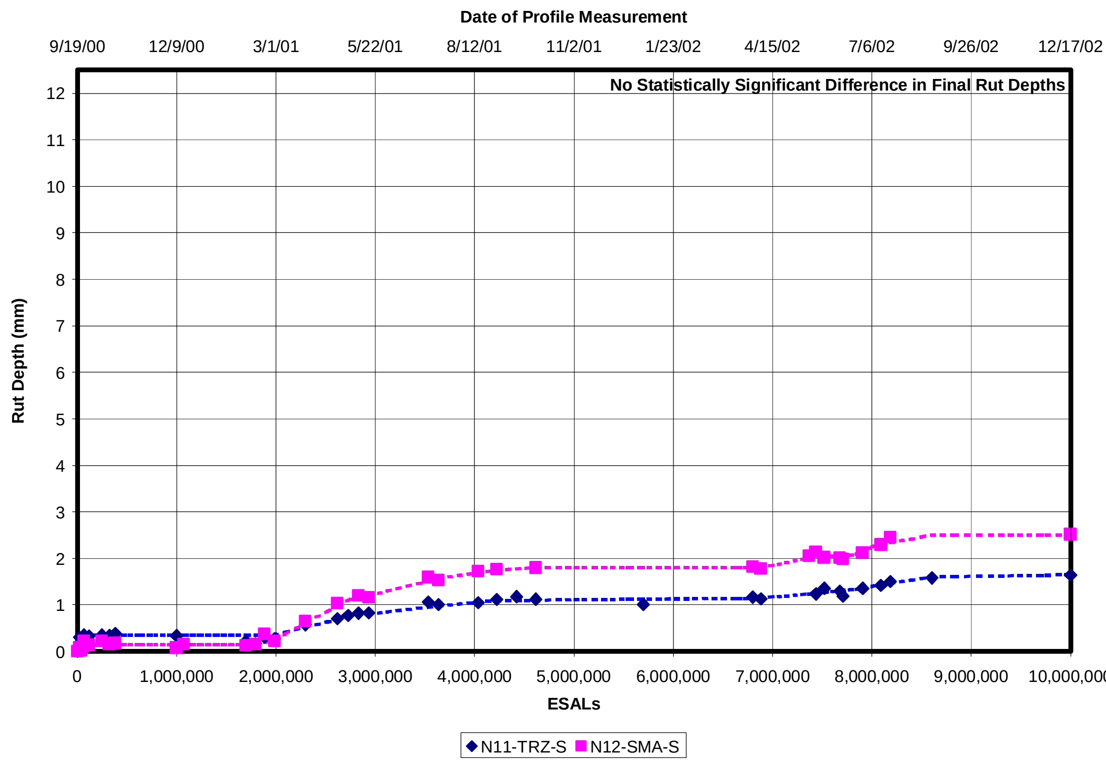

FIGURE 4.17 Graphical Comparison of Dense-Graded HMA (N11) versus SMA (N12) Mix.

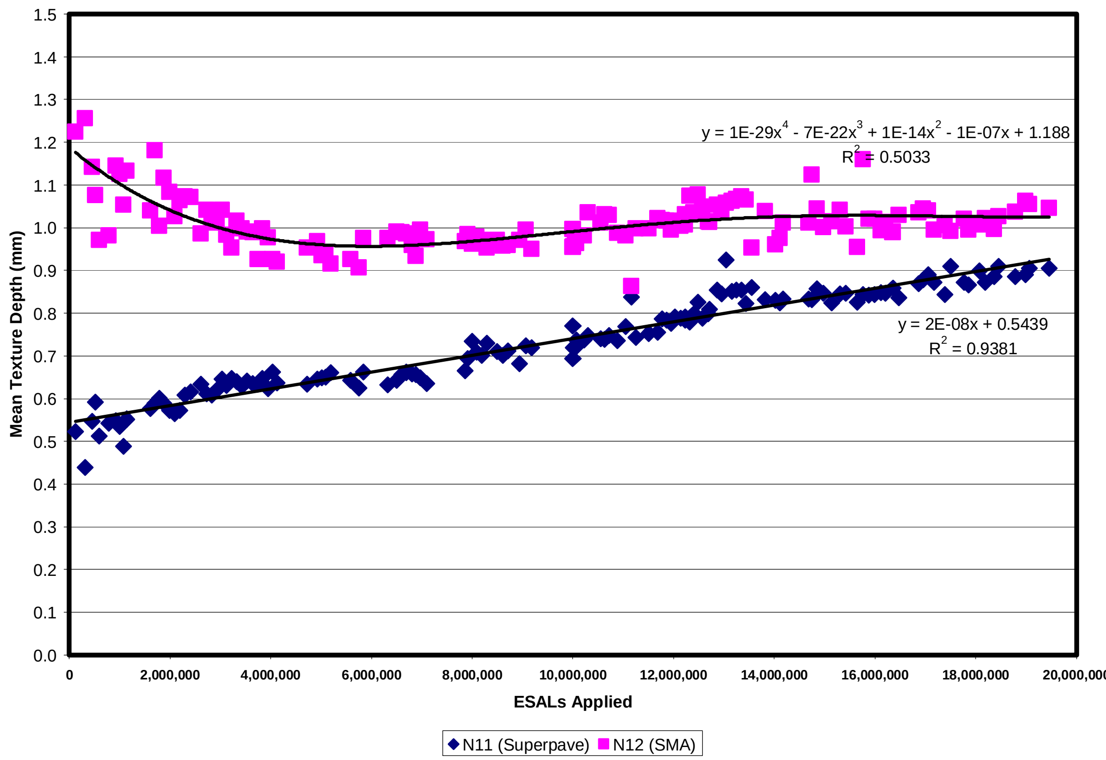

FIGURE 4.18 Changing Macrotexture in Sections N11 and N12.

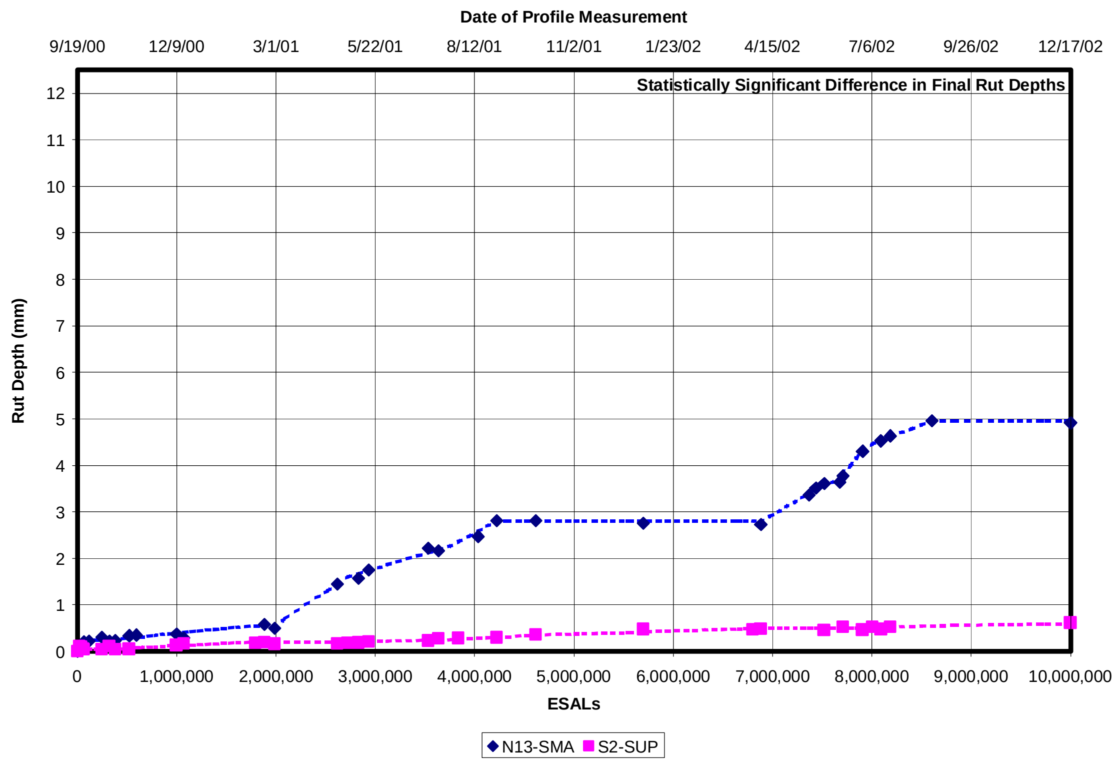

FIGURE 4.19 Graphical Comparison of Gravel HMA Designed with SUPERPAVE (S2) versus SMA (N13) Methodologies.

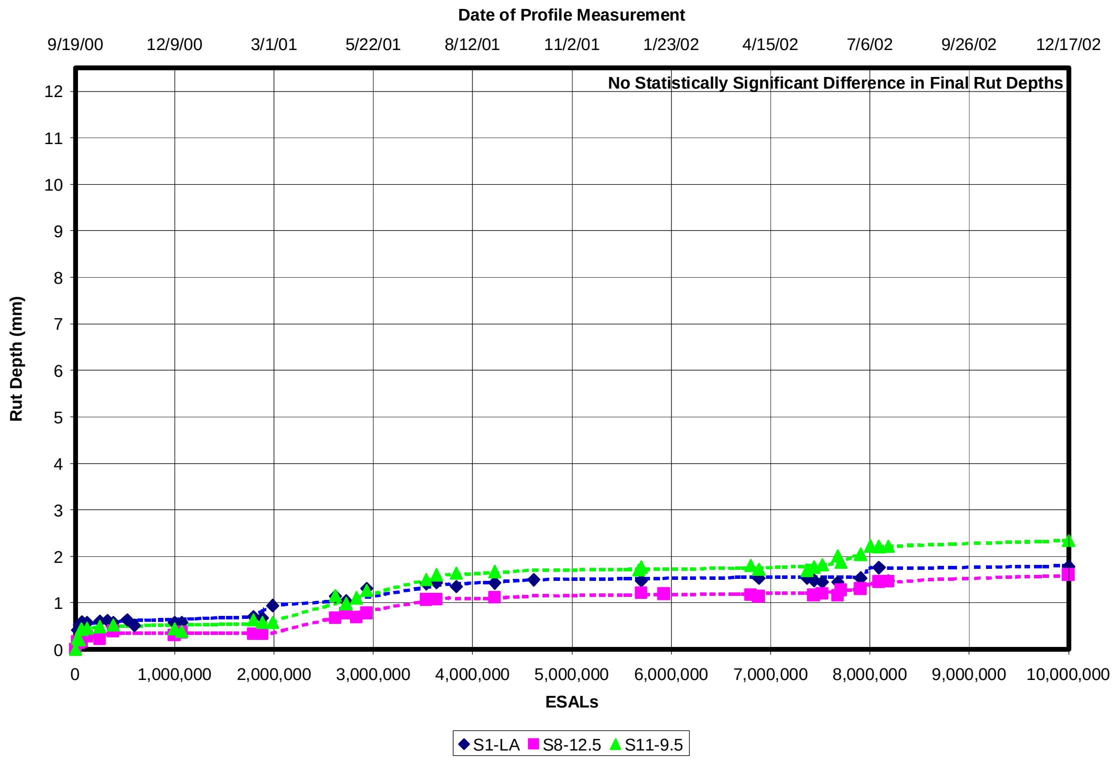

FIGURE 4.20 Graphical Comparison of High LA Abrasion Loss (S1) and Different NMAS Stone SUPERPAVE (S8 and S11) Mixes.

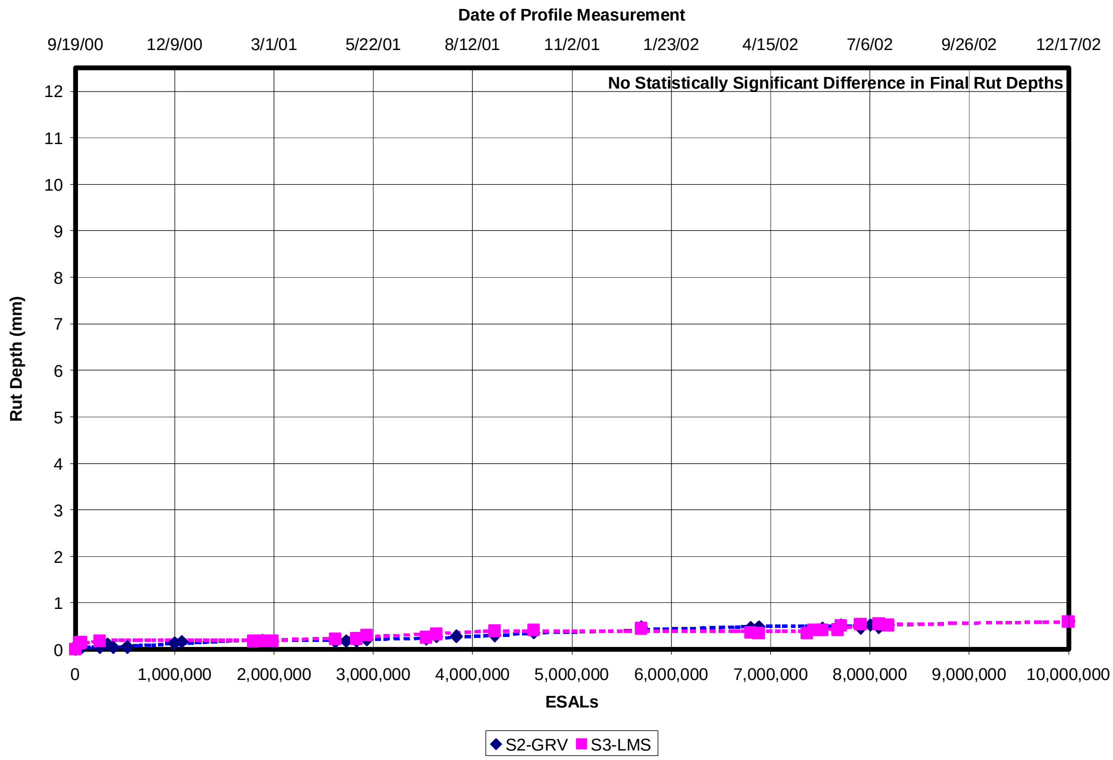

FIGURE 4.21 Graphical Comparison of HMA Gravel (S2) versus Stone (S3) Mix.

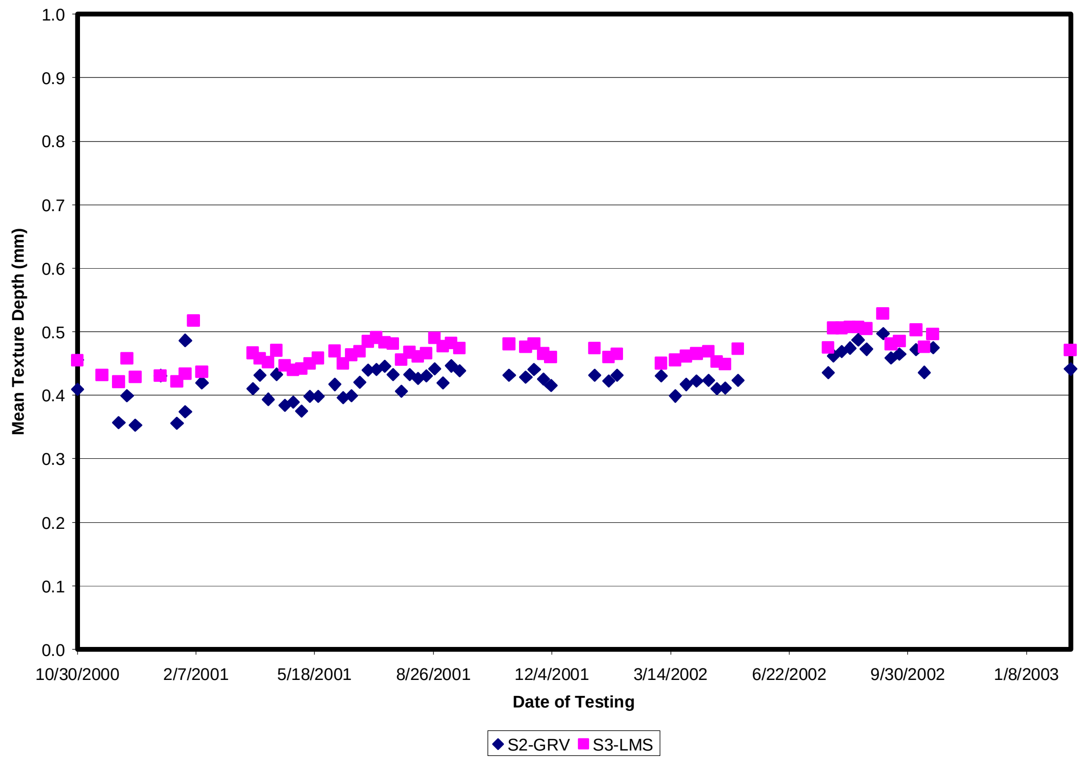

FIGURE 4.22 Macrotexture Comparison of HMA Gravel (S2) versus Stone (S3) Mix.

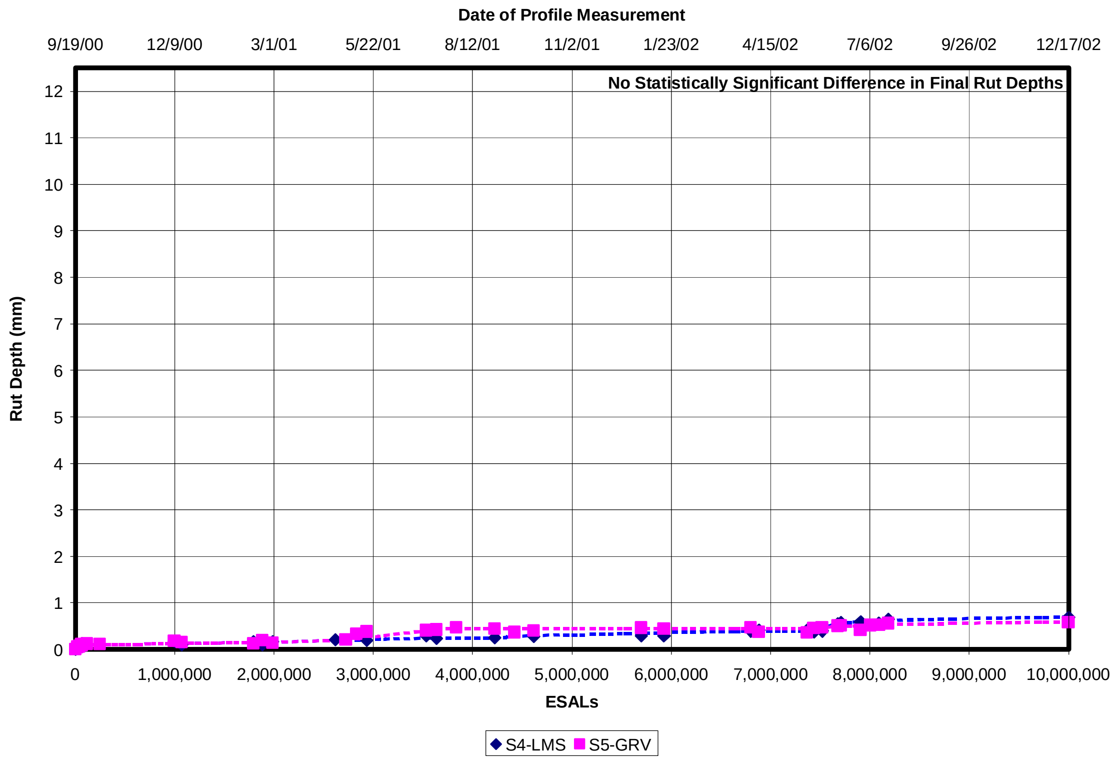

FIGURE 4.23 Graphical Comparison of HMA Gravel (S5) versus Stone (S4) Mix.

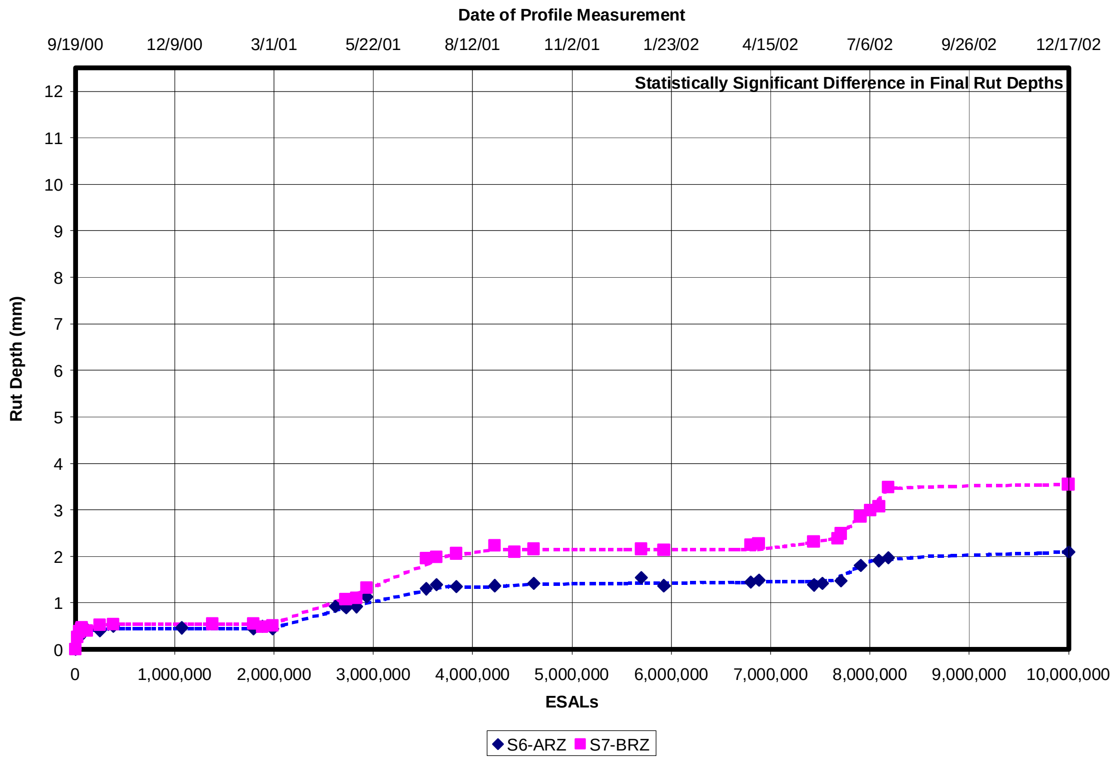

FIGURE 4.24 Graphical Comparison of Coarse (S7) versus Fine (S6) Stone Mix Blended with Unmodified Binder at Optimum.

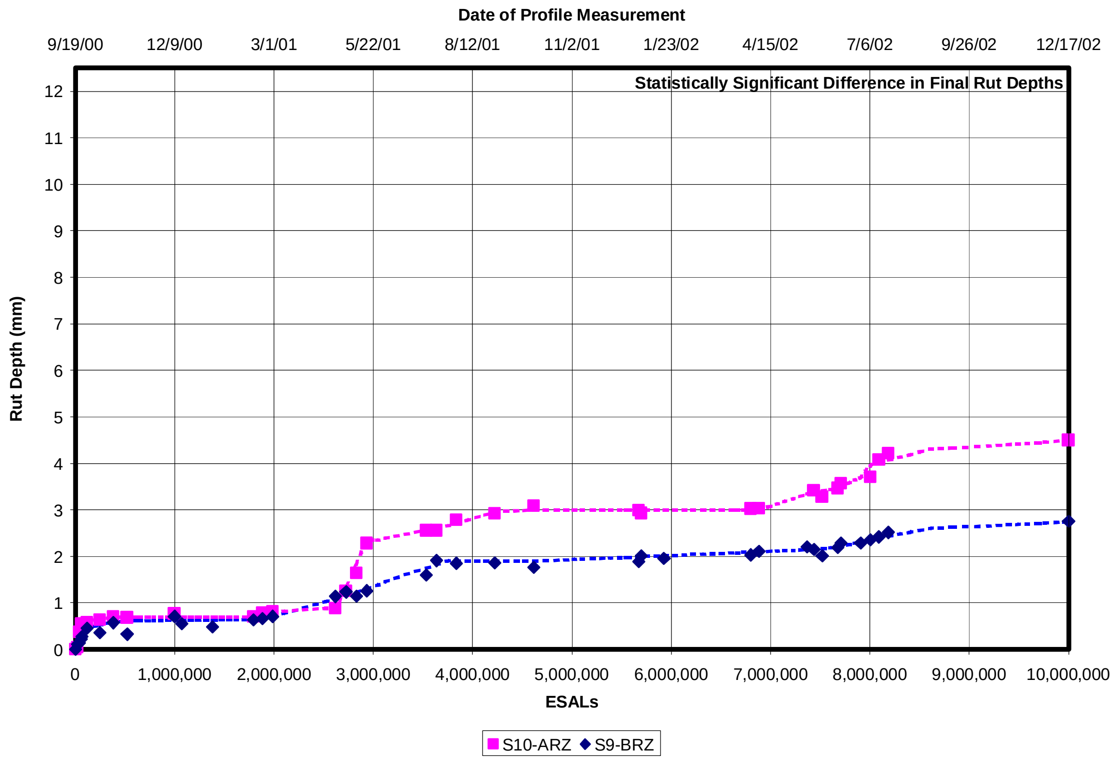

FIGURE 4.25 Graphical Comparison of Fine (S10) versus Coarse (S9) Stone Mix Blended with Unmodified Binder at Optimum.

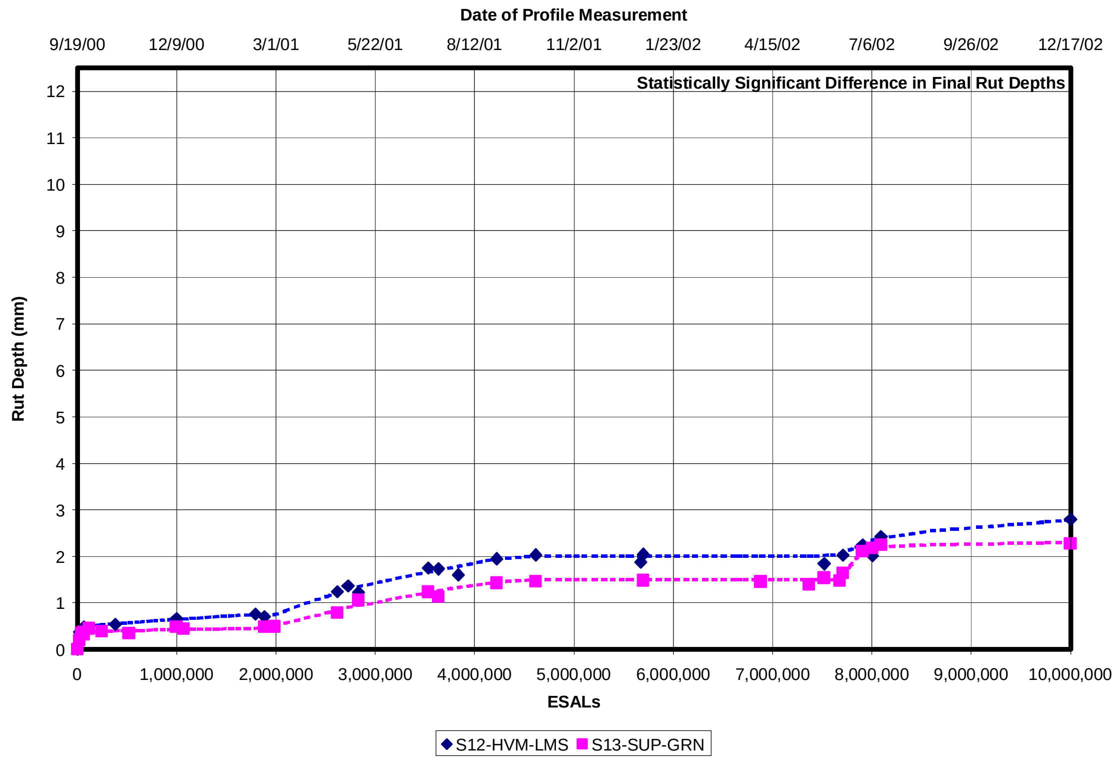

FIGURE 4.26 Graphical Comparison of HMA Mix Designed with Hveem (S12) versus SUPERPAVE (S13) Methodologies.

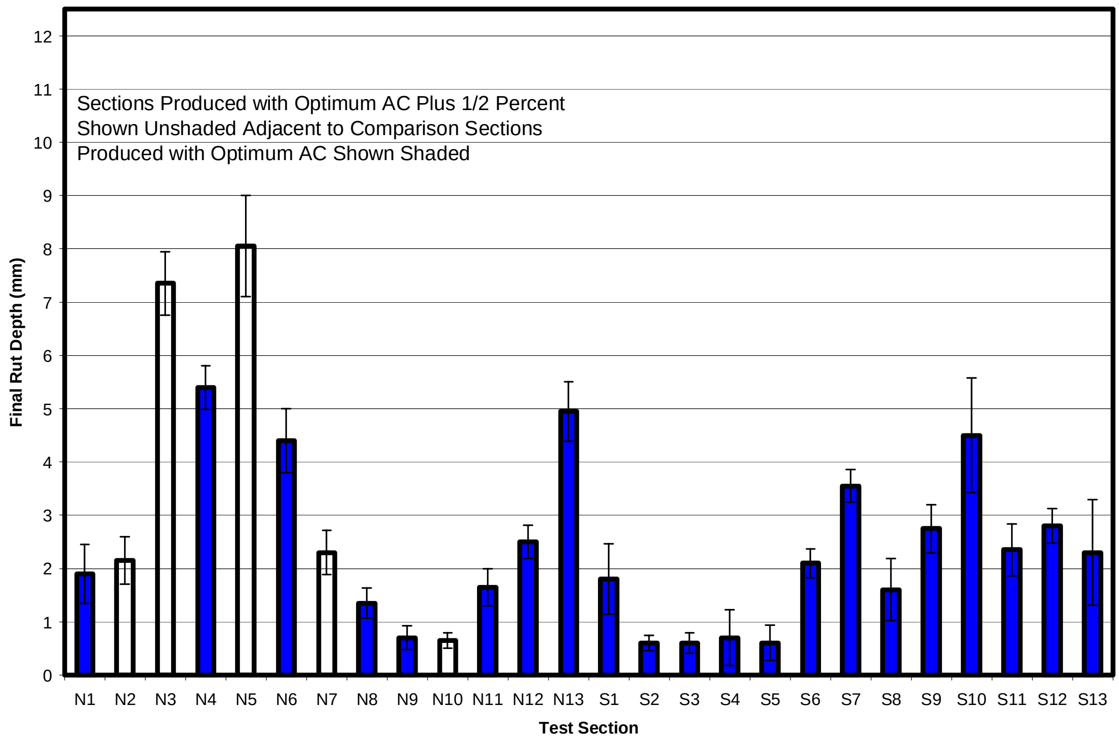

FIGURE 4.27 Final Traffic Induced Rutting on 2000 Track (± One Standard Deviation).

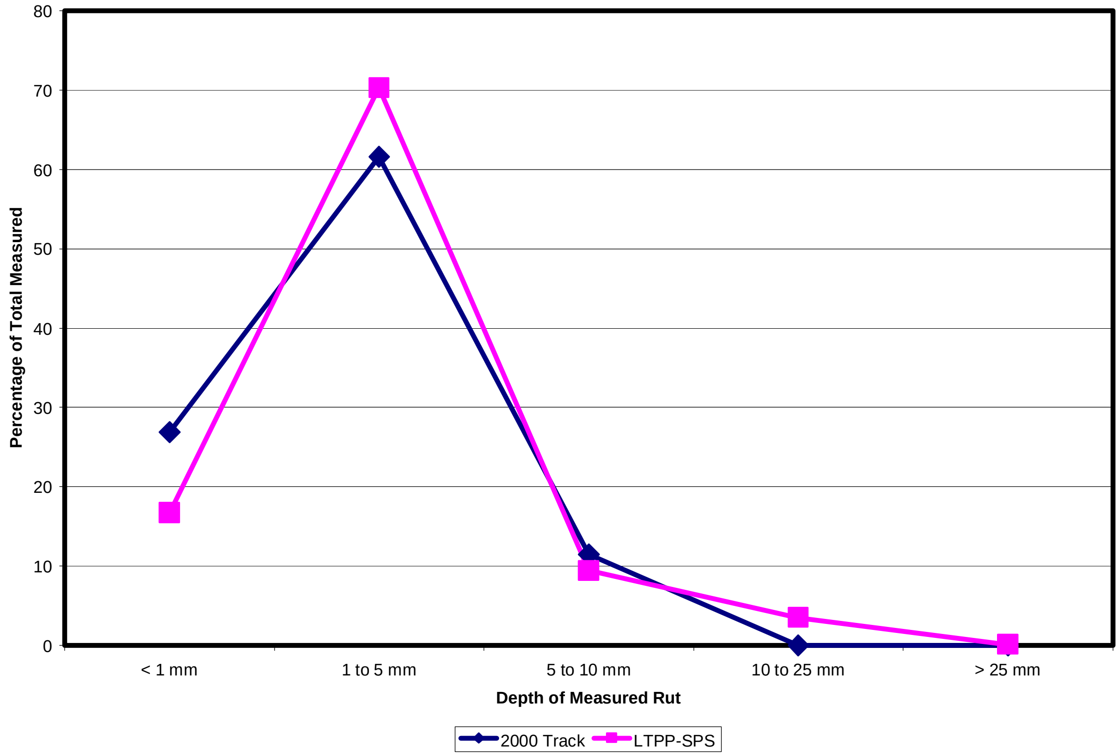

FIGURE 4.28 Relationship Between Track Rutting and LTPP-SPS Rutting (Jones, 2003).

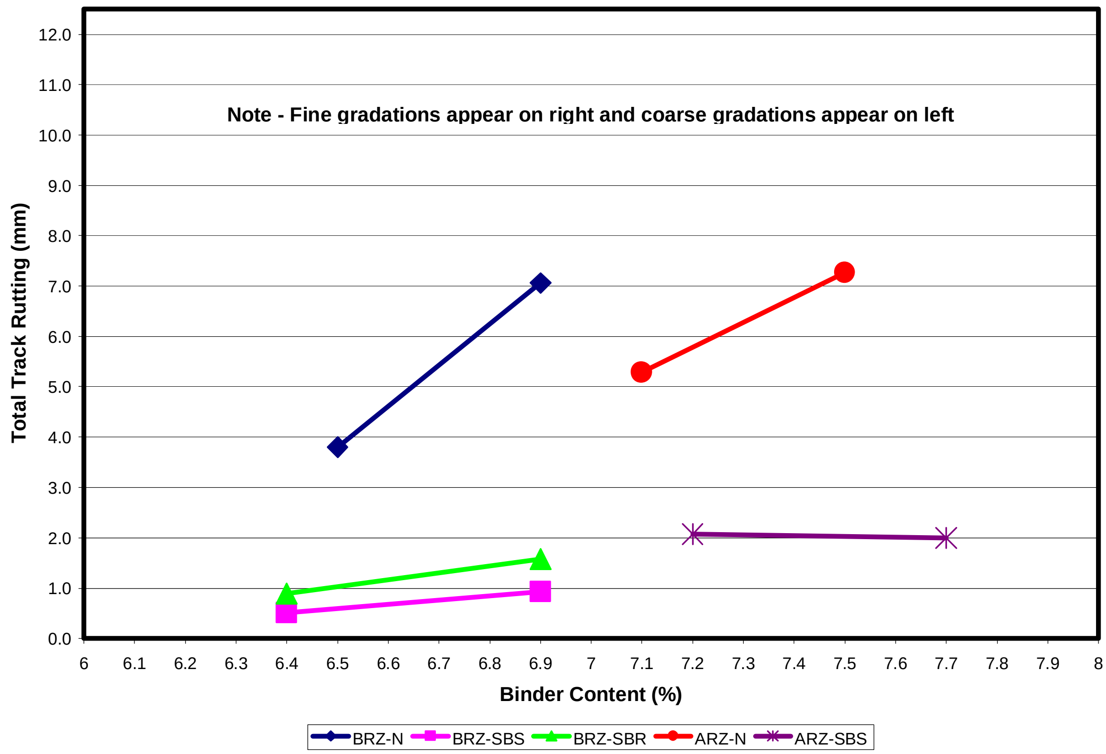

FIGURE 4.29 Effect of Asphalt Grade and Content on Rutting Performance.

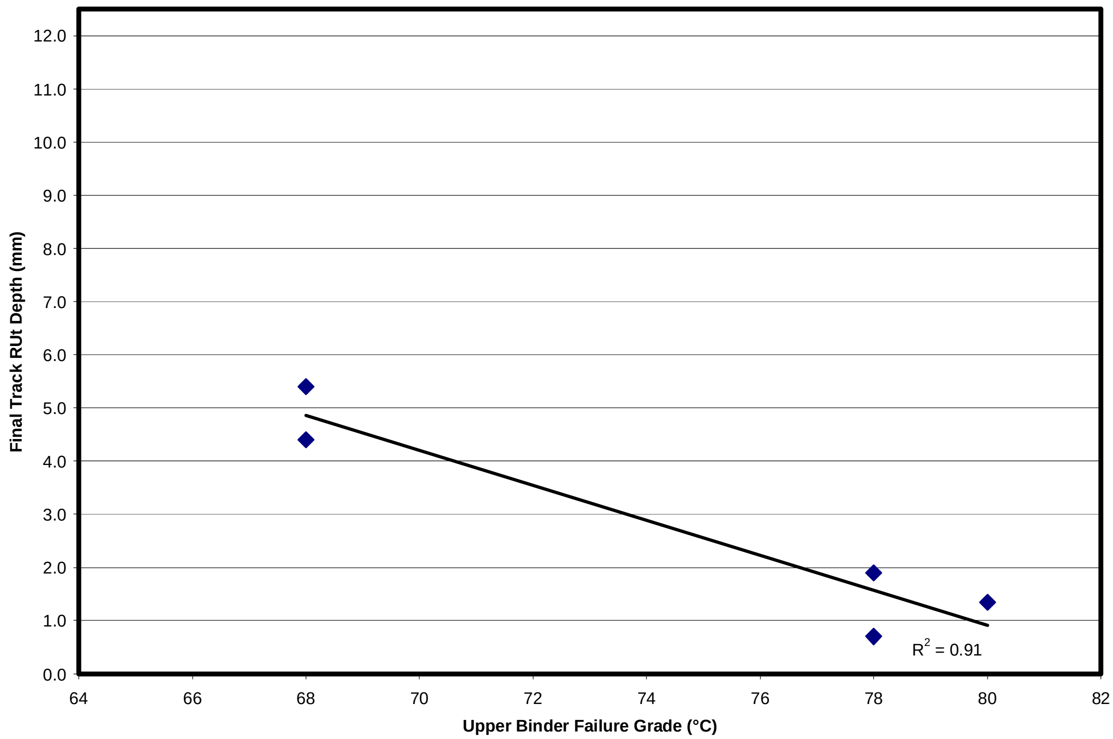

FIGURE 4.30 Effect of Asphalt Grade on Rutting Performance at Optimum Asphalt Contents.

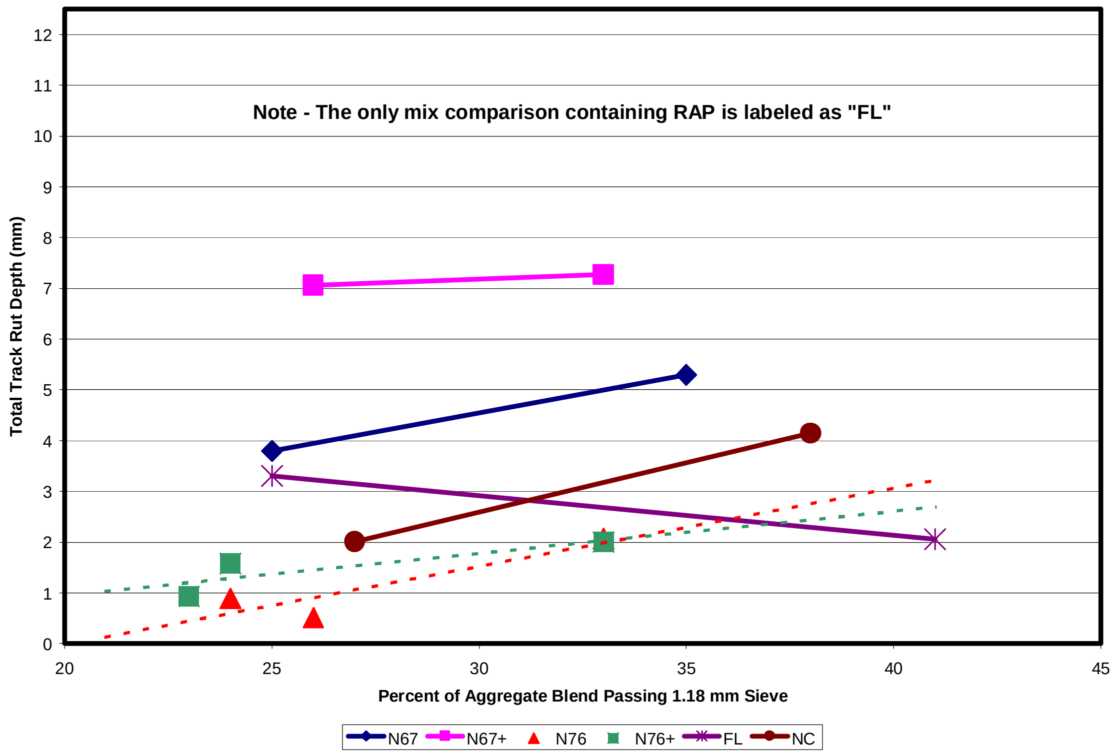

FIGURE 4.31 Effect of Gradation Type on Rutting Performance.

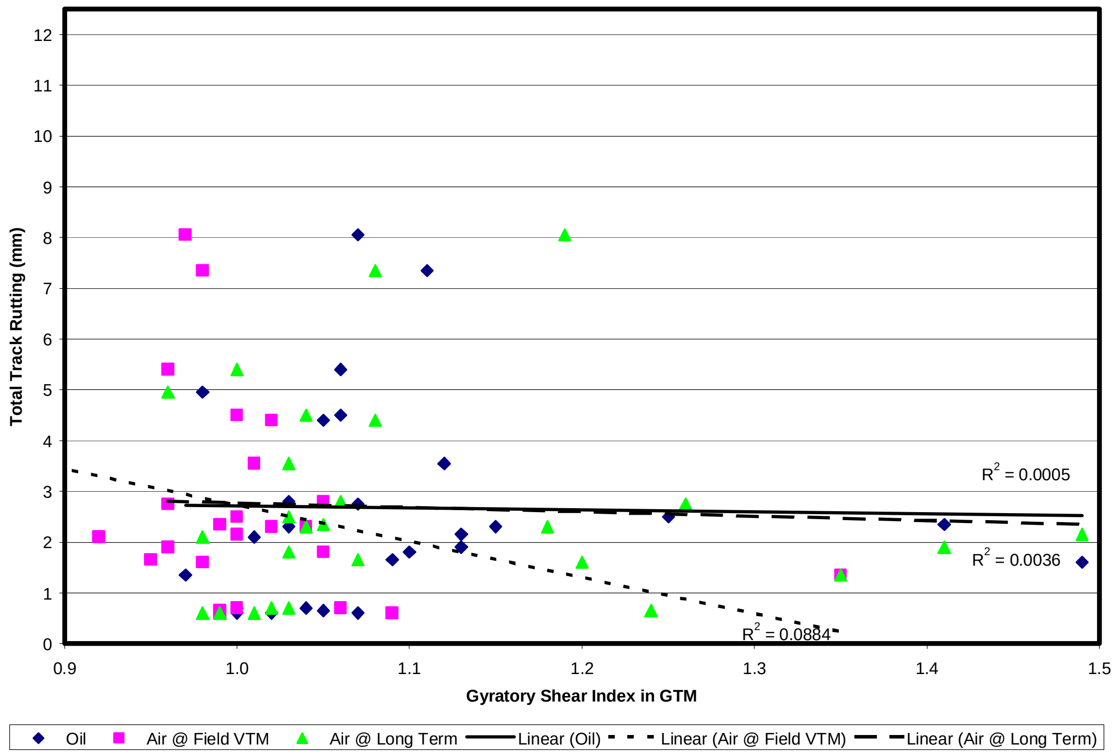

FIGURE 4.32 Relationship Between GTM Gyratory Shear Index and Track Rutting Performance.

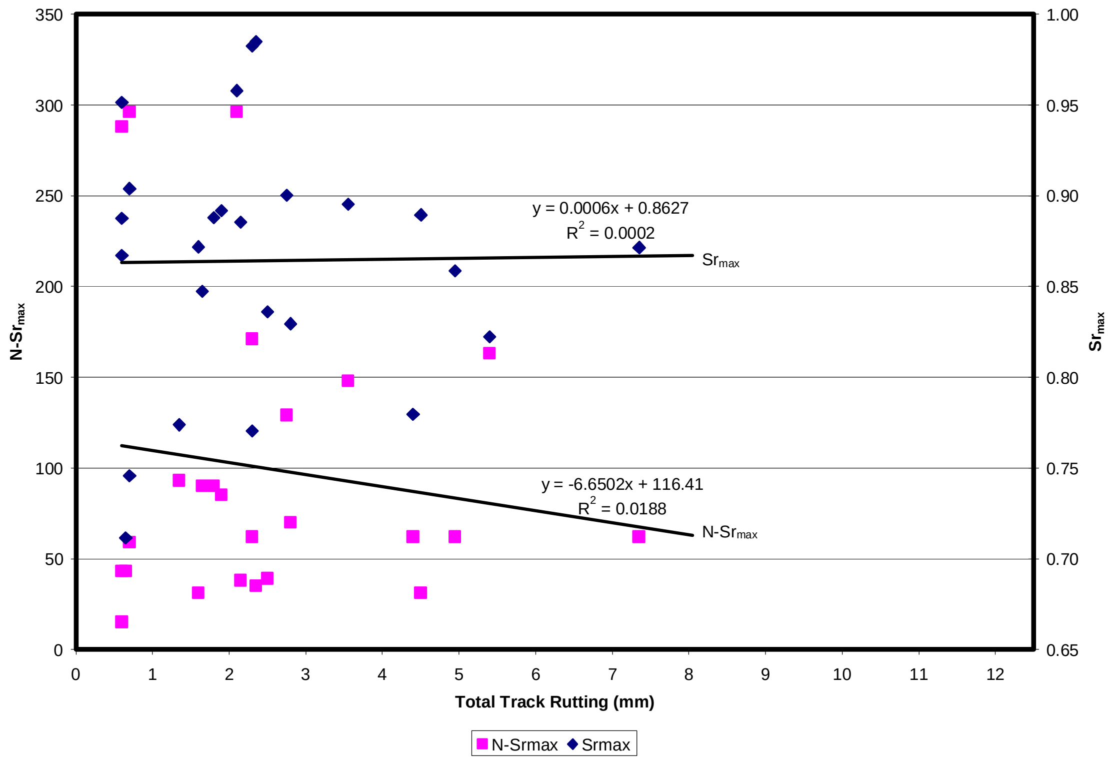

FIGURE 4.33 Relationship Between SGC Gyratory Shear Properties and Track Rutting Performance.

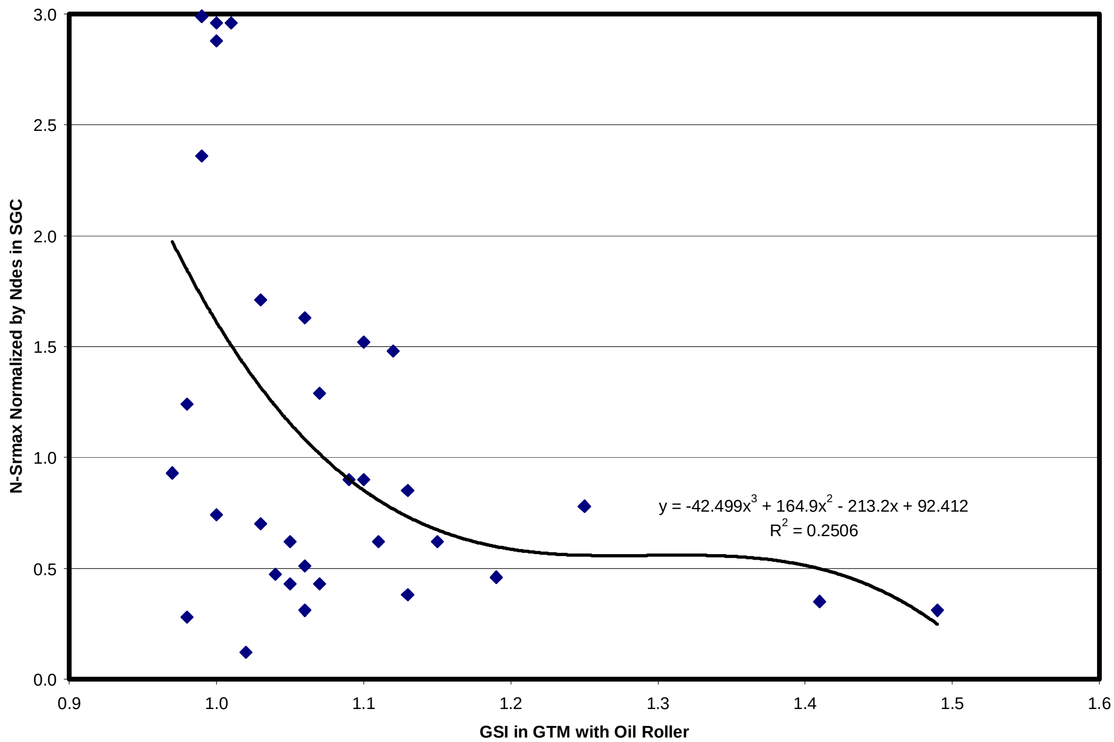

FIGURE 4.34 Relationship Between Results Generated with GTM versus SGC Compactors.

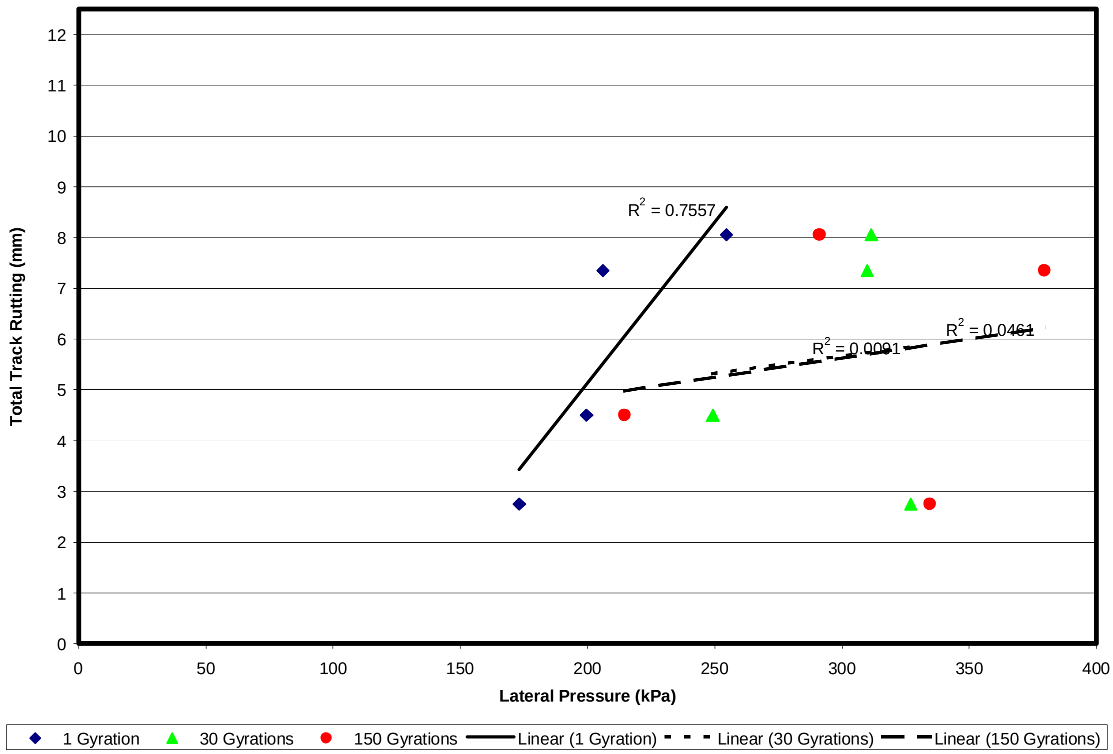

FIGURE 4.35 Relationship Between Track Rutting and Lateral Pressure Measured at Various Gyration Levels.

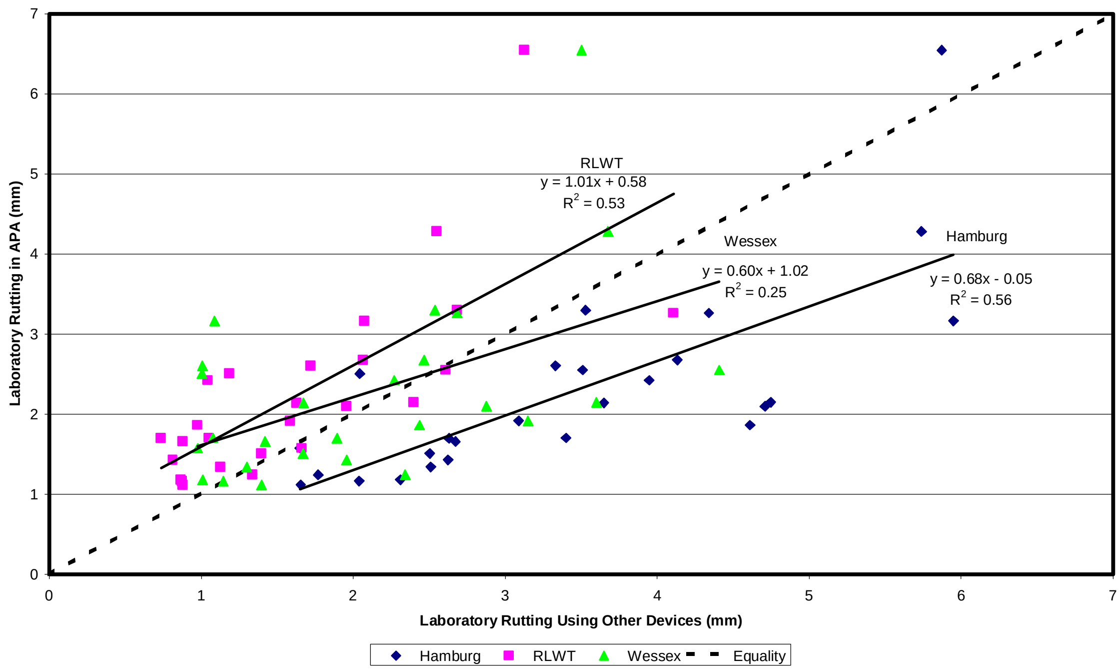

FIGURE 4.36 Relationship Between Other Simulative Laboratory Test Results and Results from the APA.

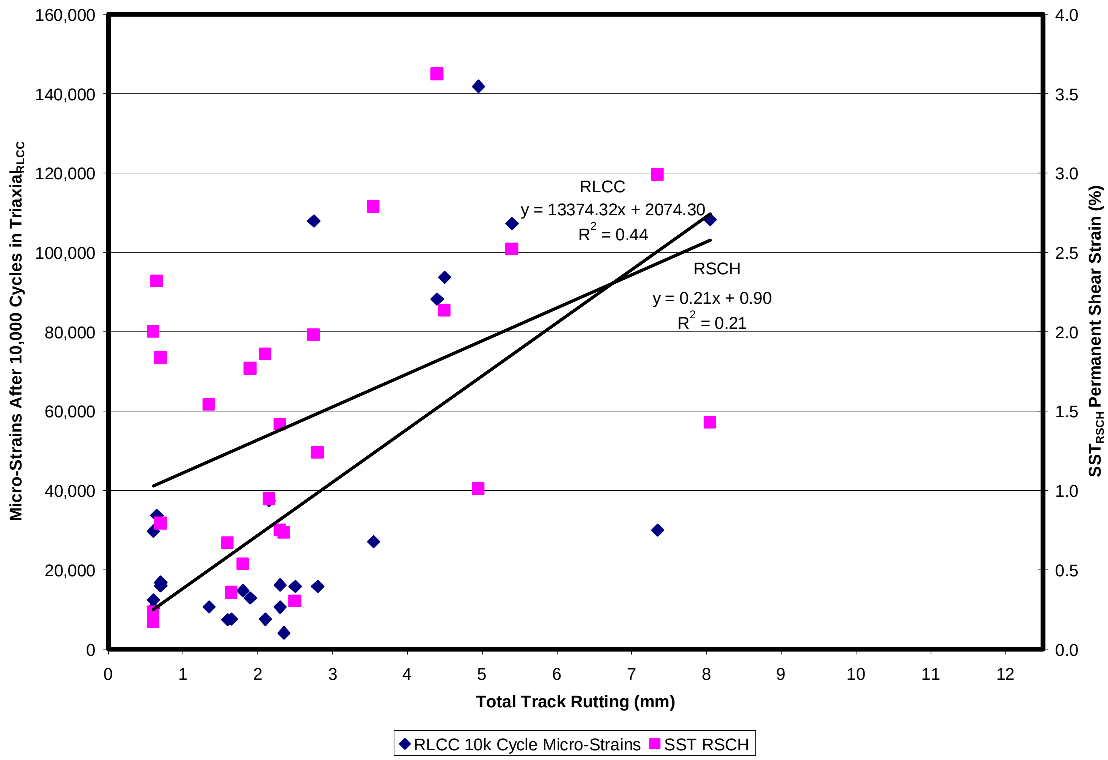

FIGURE 4.37 Relationship Between Fundamental Test Results and Track Rutting Performance.

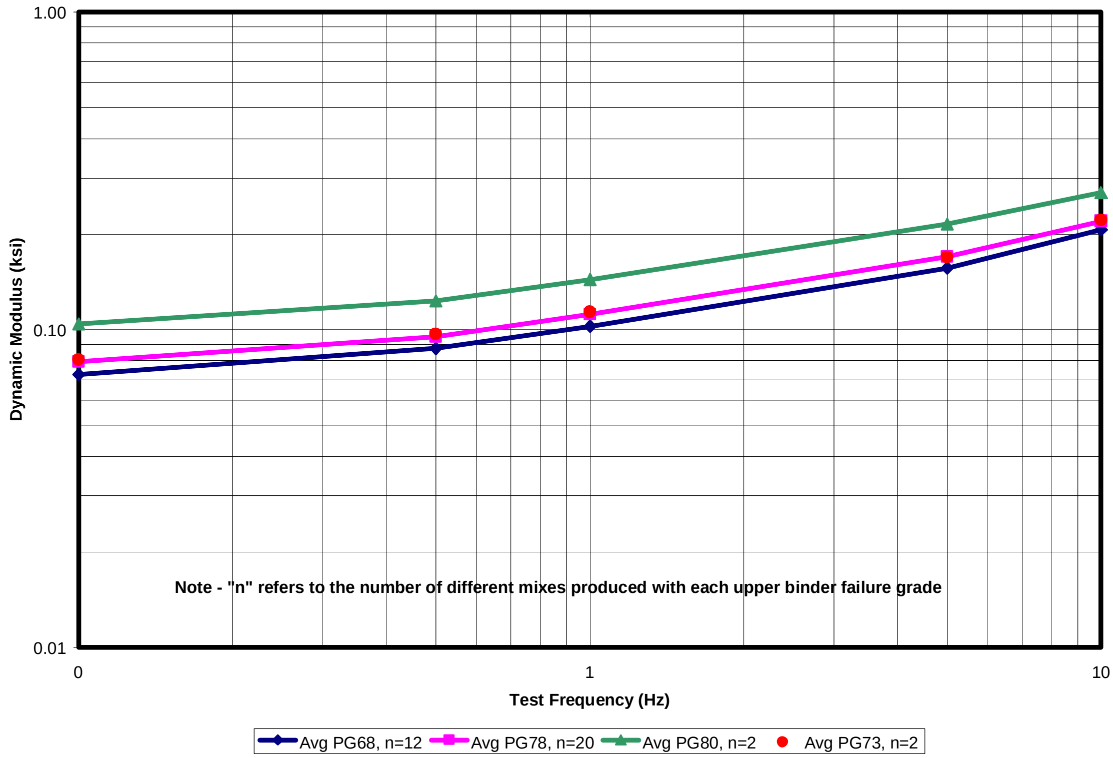

FIGURE 4.38 Average Dynamic Modulus Master Curves for Mixes with Different Upper Binder Failure Grades.

CHAPTER 5 – MODEL DEVELOPMENT

5.1 CONSTRUCTABILITY

Originally, the SGC was included in the SUPERPAVE system in the hopes that the intermediate angle of 1.25° (between the French gyratory angle of 1° and the Texas gyratory angle of 6°) would provide an indication of constructability early in the compaction process and an indication of performance late in the compaction process (Harman et al., 2002). A parameter known as N-initial (N_{ini}) was developed to characterize mixes at low gyration levels, and a parameter known as N-maximum (N_{max}) was developed to characterize mixes at high gyration levels.

As seen in Figure 5.1, average SGC data from all Track mixes reveals quick consolidation early in the compaction process up until the point where the shear strength of the mixes peak (at around 50 gyrations), with consolidation at a more constant rate thereafter. Air voids in the latter region are thought to fall between field mat compaction and laboratory sample compaction, which should theoretically be representative of voids reduction under traffic (Harman et al., 2002). Note that on average the Sr_{max} value for Track mixes occurred near 50 Gyrations, which is generally considered undesirable (Dalton, 1999).

Of course, the problem with this form of analysis is that compaction temperature is a function of the grade of binder in the mix. Consequently, one would not expect any differences in SGC "performance" between the same aggregate blend mixed with unmodified and modified binder as long as they are compacted at equal viscosity temperatures. In reality, the modified blend would likely be more difficult to compact in the field and it would be more rut resistant under traffic (at the same performance temperature).

As seen in Figure 5.2, there does not appear to be any relationship between early rate of consolidation at low gyration levels in the SGC and mat density. Likewise, there does not appear to be a significant relationship between later rate of consolidation at higher gyration levels in the SGC and rutting performance. A model was initially conceived that would utilize early data to predict difficulty in compaction (true to the original goal of the SGC); however, as indicated in Figure 5.2 it was not possible to develop a reliable methodology. Figure 5.2 does show that field VTM appears to decrease as height change in early compaction increases, and that rutting appears to increase when height change in late compaction increases; however, both coefficients of determination are very low and it is not possible to infer a meaningful trend. This observation is consistent with the findings of other researchers (Bahia et al., 2003).

Track construction specifications required the contractor to vary their compaction methods to achieve a specified end result mat density. A method specification requiring the same compactive effort on every section may have been more suitable for modeling constructability with the SGC; however, achieving densities that met sponsors' expectations was a higher priority.

A modern variation to SGC mix design is to set the number of design gyrations equal to the point where significant changes in sample height no longer occur. This approach is commonly referred to as the "locking point" method of mix design. To facilitate this process, a standardized binder content must first be identified by specification for use in sample preparation. Specimens are compacted in the SGC and the record of sample height versus gyrations is recorded. The design level of gyrations is identified from the compaction record as the point where additional gyrations no longer induce significant changes in sample height (i.e., the sample height begins to repeat). Binder content is then optimized to produce a design level of air voids in the laboratory.

The great variety of mixes and materials on the 2000 Track presents an opportunity to compare different locking point determinations at (or near) optimum asphalt contents. As seen in Figure 5.3, there can be a substantial difference in "locking point" determinations depending on how the value is defined. In this figure, the number of gyrations until the height repeats once is plotted on the y-axis for each Track mix, while the corresponding levels until the height repeats two and three times is plotted on the x-axis. Mix design personnel and those involved in the development of "locking point" specifications should consider these differences when establishing a definition for their jurisdiction and design traffic level.

5.2 RUTTING PERFORMANCE

Laboratory performance tests are useful for screening potentially poor performing mixes in the design approval process, but the ultimate test in the value of a protocol is its suitability for modeling performance on the roadway. The 2000 Track provided an excellent opportunity for evaluating different tests because performance was monitored on a weekly basis, rather than simply at the end of traffic. As a prelude to the development of new models, it was first necessary to investigate simple correlations between laboratory testing and final measured rut depths.

As seen in Figure 5.4, there does not appear to be any correlation between Track rutting performance and gyratory compaction results in either the SGC or GTM. Simulative correlations, presented in Figure 5.5, offer more encouraging results. Coefficients of determination for the relationships between final Track rutting and laboratory rutting range from a low of 0.09 (indicating no relationship) to a high of 0.52. Likewise, coefficients of determination for fundamental testing protocols (shown in Figure 5.6) were found to range from 0.02 (indicating no relationship) to a high of 0.44.

There are several reasons why correlations for all three classes of tests may be so weak. The worst correlations were seen for results generated during gyratory compaction. This is not unexpected since higher compaction temperatures are used for mixes with stiffer binder grades, which effectively masks their performance enhancing effects during the compaction process. Fundamental testing produced better coefficients of determination. This outcome is logical because fundamental testing for all mixes was run at the same high performance temperature of 64°C, which would have reflected the performance of the stiffer binders. Fundamental testing did not correlate as well as simulative testing in part because of the relatively high coefficients of variation shown previously in Table 4.11.

Recalling that Figure 5.5 demonstrates a coefficient of determination of only 0.28 in relating final APA rut depth to final Track rut depth, note in Figure 5.7 that a stronger relationship exists between deformation over the last half of APA testing and total Track rutting. The coefficient of determination for this relationship improves to 0.49. A better correlation resulted when the effect of early APA deformation, in this case the deformation over the first 4000 load cycles, was removed in order to reduce the sensitivity of the test to more variable primary consolidation.

Early in the testing process, factors such as sample air voids, slight irregularities in properties on the loaded surface of test specimens, etc. can have a significant effect on the total rut depth that is measured at the end of the test. By focusing on the change that occurs over the second half of APA testing, laboratory characterization is limited to the more linear secondary consolidation. This observation supports the notion that a rutting performance model should utilize portions of laboratory performance testing individually (i.e., primary and secondary consolidation should be given separate consideration in rutting performance prediction).

By plotting these data on a smaller scale in Figure 5.8, a clear relationship between laboratory and field performance is observed. Additionally, utilizing different symbols to represent different binder performance grades for each mix in Figure 5.8 reveals that lower PG grades generally appear on the upper portion of the correlation and higher PG grades generally appear on the lower portion of the correlation. This is because all laboratory testing was conducted at the same performance temperature, and is further evidence that actual field temperature is critical in predicting field performance and must be a factor in the modeling process. Figure 5.8 also shows that rutting performance generally improves when higher PG binder grades are utilized.

The new modeling effort proposed herein should address the following shortcomings in traditional performance prediction:

1) Traffic applied when pavements are hot induce more rutting than traffic applied when pavements are cold – they should not be treated as if they are equal;
2) Rate of primary consolidation is affected by level of compaction in both the laboratory and the field;
3) Composite pavement structures cannot be effectively represented by the laboratory performance of the surface mix alone;
4) Underlying mixes that contribute to permanent deformation exist in a different stress and performance environment than surface mixes; and
5) Underlying mixes sometimes have lower PG binder grades, which complicates performance predictions.

5.2.1 Banding Traffic by Temperature

As demonstrated in Figure 5.9, each section of the Track survives in a performance environment that is unique to its aged color (heat retention) and surface texture (surface area). Rutting that occurred on the Track over the course of one week of traffic was induced by traffic loadings applied while each unique experimental mat cycled between hotter and cooler pavement temperatures. Truck passes on hotter laps induced more rutting than the same truck passes on cooler laps.

In consideration of this effect, the multi-depth temperature record for the entire 2-year traffic cycle was combined with the hourly ESAL record to produce a new record of traffic that was separated into different temperature bands. The design of the bands, a sample of which is shown in Figure 5.10, is similar to the approach used for the binder PG grading system. In the sample data used to construct Figure 5.10, section N3 was subjected to a total of 20,095 ESALs over the course of the week that ended on September 24, 2000. As the pavement temperature cycled over the course of each traffic day, these ESAL data are subdivided into temperature bands. Logical breakpoints for these bands were borrowed from the grading system that is already used to describe laboratory performance differences for asphalt binders.

Since ESALs applied when the pavement is hotter would be expected to induce more rutting, a rational method was needed to produce a factor that could be used to mathematically increase the significance of ESALs applied to hotter pavement surfaces. ESALs applied in each band could then be multiplied by the appropriate temperature factor for that band. This would be done to all applied ESALs, and the resulting products would be summed to produce an effective ESAL value that could be used as the traffic input for subsequent model predictions.

Hot pavements rut more than cool pavements under the same ESAL load because hot pavements are less resistant to permanent deformation. As a result of later Track research (Timm and Priest, 2006), it was possible to quantify the elastic component of pavement response at different temperatures under passing loads. A regression model to predict stiffness as a function of mix temperature was developed from data collected on the 2003 Track (Timm and Priest, 2006). The equation that was used to predict stiffness as a function of mid-depth temperature takes the form (Timm and Priest, 2006):

$$Stiffness_{Est} = \frac{2{,}793{,}344}{e^{(0.05976 \times {}^{\circ}C)}} \qquad \text{Equation 5.1}$$

Stiffness estimates are useful parameters because they give some indication of how much HMA will resist deformation at different temperatures; however, what is needed is an estimate of permanent deformation at different temperatures. Since the objective was to generate what is in effect a rutting weight factor to apply to ESALs applied when pavement surface temperatures fell within successively hotter temperature bands, the following equation was used to estimate compliance:

$$Compliance_{Est} = \frac{1}{Stiffness_{Est}} = \frac{1}{\left(2{,}793{,}344 \cdot e^{(-0.05976 \times {}^{\circ}C)}\right)} = \frac{e^{0.05976 \times {}^{\circ}C}}{2{,}793{,}344} \qquad \text{Equation 5.2}$$

Equation 5.2 was used to calculate a compliance value at the midpoint temperature of each PG band. Since rutting was not observed on the Track when ESALs were applied on pavements with surface temperatures cooler than 34°C (the low end of the 34°C to 40°C PG band), estimated compliance at 37°C (in the middle of the band) served as the reference point in the development of temperature factors for ESALs applied to hotter pavement surfaces. Compliance was estimated at the midpoint of all hotter PG bands, and each midpoint estimate was divided by compliance at 37°C (since it was the reference point).

Surface temperatures were used since they are the most straightforward values to obtain in practice (or to estimate for future predictions) on open roadways; however, the variation in temperature as a function of depth in pavement structures is an important factor that is considered in a later step. It should be noted that Equation 5.1 was developed using mid-depth temperatures; however, that is because it was derived empirically based on elastic layer analysis and it was important to use a value that represented the entire HMA layer as a homogeneous mass.

The following derivation was used to construct Table 5.1, which provides data for the construction of Figure 5.11 and the resulting model for ESAL weight factor at temperatures hotter than 37°C:

$$ESAL_{Wt\ Factor\ Actual} = \frac{Compliance_{Est\ @\ Hotter°C}}{Compliance_{Est\ @\ 37°C}} \quad \text{Equation 5.3}$$

$$ESAL_{Wt\ Factor\ Model} = \left(1.76\times10^{-5}\right)\cdot Temp_{Mid\ PG\ °C}{}^{3.01} \text{ (via Figure 5.11, > 34°C)} \quad \text{Equation 5.4}$$

$$ESAL_{Model} = \sum\left(ESAL_{Hot\ Banded\ >34°C} \times ESAL_{Wt\ Factor\ Model}\right) \quad \text{Equation 5.5}$$

Rather than simply using the total number of applied ESALs to predict performance, the load-temperature spectra approach described herein utilizes the sum of weighted hot ESALs for predictive purposes. Cold ESALs (applied when the pavement surface temperature is less than 34°C) are not considered at all (for rutting performance), and progressively higher weight factors are assigned for ESALs applied within hotter PG grading bands. The result is an $ESAL_{model}$ that is very different from the total ESALs applied. This methodology is known as the "load-temperature spectra" approach.

For example, assume that 100 ESALs are applied while the pavement surface temperature is 39°C and 100 ESALs are applied while the pavement surface temperature is 41°C. In this case, ESALs applied at 39°C fall into the band with a midpoint temperature of 37°C, for which Equation 5.4 produces an $ESAL_{wt\,factor\,model}$ value of 0.950. ESALs applied at 41°C fall into the band with a midpoint temperature of 43°C, for which Equation 5.4 produces an $ESAL_{wt\,factor\,model}$ value of 1.454. Although the simple numerical total ESAL count is 200, the application of Equation 5.5 results in a weighted total ESAL count of (0.950 x 100) + (1.454 x 100) = 240 for use in rutting performance prediction.

5.2.2 Scaling Primary Consolidation

All 3 classes of laboratory testing (testing during gyratory compaction, simulative testing and fundamental testing) exhibit final deformation values that are some combination of primary and secondary consolidation. Figure 5.12 is provided to illustrate this principal, and includes data from simulative and fundamental testing (the scale was not conducive for including compaction testing, but Figure 5.1 shows that the same type of consolidation curve is produced).

Primary consolidation observed in the laboratory during both simulative and fundamental testing can be significantly affected by the VTM of the test specimen. At the same binder content, laboratory samples with higher VTM values typically have greater levels of consolidation early in the testing process. The same effect can be expected when relating laboratory results (generated with samples at QC voids) to field conditions (expected to have much lower densities than QC samples, but the same asphalt content). For this reason, it is necessary to scale primary consolidation in the laboratory (in both simulative and fundamental testing) as a function of higher air voids when relating results to field performance:

$$Lab\ \mathrm{Pr}imary\ Consolidation_{Scaled} = Lab\ \mathrm{Pr}imary\ Consolidation_{Measured} \times \left(\frac{VTM_{Field}}{VTM_{Lab}} \right)$$

Equation 5.6

Simply put, laboratory primary consolidation is scaled up by the ratio of field voids to laboratory voids. This is a linear adjustment because literature indicates that primary consolidation is the result of volume change alone (Eisenmann and Hilmer, 1987). If deformation without volume change is expected, a multidimensional (third power) adjustment may be needed. It is not necessary to scale secondary consolidation in this manner because it is assumed secondary consolidation is not as sensitive to differences in air voids.

5.2.3 Temperature Shift Factor

Asphalt binders that are classified with the same PG grade can have actual high temperature failure grades that differ by as much as 6°C (the range of each PG band). For example, a PG77 and a PG81 are both assigned to the banded grade of PG76. In consideration of this phenomenon, the actual upper binder failure grade was chosen to serve as the basis of this experiment in order to better differentiate performance.

When asphalt binders are specified properly, in almost every case laboratory performance testing will be run at a temperature that is lower than the upper binder failure grade. Performance data that are generated by laboratory testing at the appropriate field PG grade would be less stiff if each sample could be run at its own unique binder failure grade. Since it is not practical to adjust the temperature for laboratory performance testing to exactly match higher binder failure grades, it was decided to develop a model parameter that reflects the necessary temperature shift.

Temperature shift (T_{shift}) is defined herein as the difference between the upper binder failure grade and laboratory test temperature. For example, by applying Equation 5.7 it is seen that a mix produced with binder having an upper temperature failure grade of 76°C and run in the laboratory at 64°C would have a T_{shift} value of 12°C.

$$T_{Shift} = PG_{Binder\ Upper\ Failure\ Temperature} - Lab\ Test\ Temperature \qquad \text{Equation 5.7}$$

5.2.4 Underlying Layer Considerations

Since pavement structures on roadways are usually combinations of different mixes, the effect of underlying mixes must be considered when predicting rutting performance. Underlying binder mixes can have completely different laboratory rutting performance than surface mixes, and may need to survive under completely different performance conditions; however, a rational method is needed with which a single number representing both layers can be produced. Rather than compute a simple numerical average for different mixes in a composite structure, a weighted average for underlying layers is necessary to account for cooler temperatures, lower stress levels, and potential differences in binder grade.

5.2.4.1 Cooler Temperatures

Although it is not necessary to scale secondary consolidation to account for differences in air voids between the laboratory and open roadways, it is necessary to scale both primary and secondary consolidation to reflect the cooler temperatures of underlying layers. Cooler temperatures will result in less plastic deformation and must be accounted for to maximize the effectiveness of subsequent rutting predictions. Track stiffness models were again utilized to construct an adjustment, but in this case stiffness predictions were combined with the SHRP temperature profile model to obtain a profile of estimated stiffness versus depth (Huber, 1994). The following derivation was used to construct Figure 5.13 and the resulting model component for reducing both primary and secondary laboratory consolidation as a function of the effect of generally cooler temperatures in underlying layers.

Equation 5.8 is used to estimate the temperature of underlying layers via known (in the case of the Track) or predicted (in the case of future use) pavement surface temperatures (Huber, 1994):

$$Temp_{Depth} = Temp_{Surface} \times \left(1 - \left(0.063 \cdot Depth\right) + \left(0.007 \cdot Depth^{2}\right) - \left(0.0004 \cdot Depth^{3}\right)\right)$$

Equation 5.8

With estimates for temperature at increasing depths, it was then possible to utilize Equation 5.1 to estimate stiffness as a function of depth. Stiffness describes resistance to deformation, but in order to predict rutting performance the tendency to yield is of greater interest; consequently, equation 5.2 was used to convert stiffness into compliance.

Equations 5.7, 5.1 and 5.2 were thus used to estimate compliance as a function of depth from the pavement surface. Since compliance at the surface has the greatest influence on composite rutting performance, surface compliance was chosen to serve as the devisor in computing a temperature depth factor ($T_{depth\ factor}$). This $T_{depth\ factor}$ parameter is simply the ratio of compliance at varying depths to compliance at the surface, which is shown as Equation 5.9:

$$T_{Depth\ Factor\ Actual} = \frac{Compliance_{Depth\ Temp\ Est}}{Compliance_{Surface\ Temp\ Est}}$$

Equation 5.9

Unique $T_{depth\,factor}$ values were computed at 0, 25, 50, 75 and 100 mm in order to produce Figure 5.13. To avoid the necessity of estimating temperatures, compliance and $T_{depth\,factor}$ values versus depth in practice, a second-degree polynomial curve was best fit to these data to produce Equation 5.10:

$$T_{Depth\ Factor\ Model} = 1 - (0.0082 \cdot Depth) + ((3.58 \times 10^{-5}) \cdot Depth^2) \qquad \text{Equation 5.10}$$

Equation 5.10 could then be used to apply a correction factor to data describing both primary and secondary consolidation to reflect the cooler temperatures of underlying layers as seen in Equation 5.11:

$$Lab\ Consolidation_{Adjusted} = Lab\ Consolidation(primary_{Scaled}\ \&\ secondary) \cdot T_{Depth\ Factor\ Model}$$

Equation 5.11

5.2.4.2 Lower Stress Levels

If a surface mix and the underlying binder mix exhibit different laboratory rutting properties, a weighted average must somehow be computed to serve as a composite performance indicator for model inclusion. A simple numerical average, even one that somehow considered thickness, would not necessarily have been mechanistically correct. This need applied to both primary and secondary consolidation.

To address this issue, the WESLEA computer program was used to compute pavement response within the Track's test sections. A depth of 100 mm was used as the basis of the analysis because no rutting was noted below this depth when the 2000 Track was trenched. Trenching after the end of trucking on the 2000 Track is illustrated in Figure 5.14, in which florescent string lines placed on all layer interfaces at the point on the Track with the greatest rut depth clearly revealed that deformation did not occur below 100 mm. The following derivation was used to construct Figure 5.15 and the resulting method for computing weighted averages of primary and secondary laboratory consolidation in multi-layered pavement systems.

The WESLEA computer program was used to estimate pavement response at 25 mm increments from the surface of the pavement to progressively deeper locations in the middle of the wheelpath. Results from the WESLEA analysis, including assumed layer properties for the Track's robust buildup, are included as Table 5.2. Since no tertiary flow was observed on any test sections, it was decided that normal vertical stress would be the focus of the investigation. If tertiary flow had been observed, shear stress may have been a more suitable parameter to utilize in this effort. Because there was no rutting below 100 mm, the effect of normal vertical stress on rutting performance had fully played out at a depth of 100 mm, meaning its effect had transitioned to zero. A rational method was needed to numerically weight the effect of normal vertical stress on rutting performance between depths of 0 and 100 mm.

Based on predicted responses from Table 5.2, the normal vertical stress at the bottom of each 25 mm increment was subtracted from the normal vertical stress at the surface of the pavement in order to compute the change in normal vertical stress due to increased depth. Next, the total change in normal vertical stress between the surface of the pavement and 100 mm was computed. The incremental change was then divided by the total change to produce the normal vertical stress multiplier shown as Equation 5.12:

$$Stress\ Multiplier_{Actual} = \frac{\left(Normal\ Vertical\ Stress_{Top\ of\ Layer} - Normal\ Vertical\ Stress_{100\ mm}\right)}{\left(Normal\ Vertical\ Stress_{Top\ of\ Pvmt} - Normal\ Vertical\ Stress_{100\ mm}\right)}$$

Equation 5.12

Various stress multiplier values were computed at 0, 25, 50, 75 and 100 mm in order to produce Figure 5.15. To avoid the necessity of predicting normal vertical stresses as a function of depth on a case-by-case basis in practice, a second-degree polynomial curve was best fit to these data that produced Equation 5.13:

$$Stress\ Multiplier_{Model} = 1.0083 + \left(0.001 \cdot Depth\right) - \left(0.0001 \cdot Depth^{2}\right)$$ Equation 5.13

Equation 5.13 could then be used as a weighting factor to compute a weighted average of both primary and secondary consolidation that reflected the lower stress levels of underlying layers as seen in Equation 5.14:

$$Lab\ Consolidation_{\mathrm{Pr}imary\ \&\ Secondary_{Composite}} = \frac{Lab_{Upper} + \left(Lab_{Lower} \cdot Multiplier_{Model}\right)}{\left(1 + multiplier_{Model}\right)}$$

Equation 5.14

5.2.4.3 Binder Grade Differences

It is also necessary to compute a weighted average T_{shift} for multi-layered pavement systems in which upper and lower layers contain different binders. A single T_{shift} value is needed to represent the composite pavement structure (down to a depth of 100 mm), which is obtained via Equation 5.15 through a similar weighting method based on the aforementioned stress multiplier:

$$T_{shift_{composite}} = \frac{\left(T_{shift_{Upper\ Layer}} + \left(T_{shift_{LowerLayer}} \cdot Multiplier_{Model}\right)\right)}{\left(1 + Multiplier_{Model}\right)} \qquad \text{Equation 5.15}$$

5.2.4.4 Composite Lab Performance

In summary, it was first necessary to mathematically scale primary consolidation measured in the laboratory to match the level of air voids constructed in the field. This was done by multiplying primary consolidation by field VTM then dividing by laboratory VTM, and was necessary for both upper and lower layers. It was then necessary to scale down both the primary and secondary consolidation measured in the laboratory for the underlying layer, since it performs in a cooler environment. A second-degree polynomial equation was developed for this purpose. The difference in temperature between the upper binder failure grade and laboratory test temperature was then computed, which has been referred to herein as the temperature shift factor. Lastly, it was necessary to combine lab performance data for binder and surface layers using a rational, stress-based weighting method (using another second-degree polynomial equation) to produce a single composite parameter each for primary consolidation, secondary consolidation and temperature shift factor for model inclusion.

5.2.5 Predicting Performance for a Specific Time Interval

Since the goal of the process was to predict rutting from traffic via laboratory performance, the basic predictive model was built upon Equation 5.5 and took the following form:

$$dRut = ESAL_{Model} \cdot X_1 \cdot X_2, \; where \qquad \text{Equation 5.16}$$

dRut = *change in rutting for the time interval in which* $ESAL_{Model}$ *was applied*

X_1 = *laboratory performance correlation for a specific time interval*

X_2 = *age factor that serves to explain data in all other time intervals*

By solving Equation 5.16 for X_1, it was seen that the function to relate "load-temperature spectra" to observed incremental rutting performance took the following form:

$$X_1 = \frac{dRut}{ESAL_{Model} \cdot X_2} \qquad \text{Equation 5.17}$$

The parameter X_2 was expected to encompass an age effect that would change with time. In order to identify the effect of age in Equation 5.17, it was decided that a trial and error investigation would be conducted to search for a time interval within the two-year life of Track pavements when X_2 (the age effect) was equal to 1. In any time interval when X_2 was found to be equal to 1, it would be possible to solve for X_1 directly. This simplest form of X_1 would then be solely a regression function of laboratory performance for the composite pavement structure.

By trial and error, it was found that the best regression correlation was for the time interval that spanned the summer of 2001 (when the midpoint age was 301 days). In effect, this implies that translation from the laboratory to the field was most applicable to the Track at that time. Since both *dRut* and $ESAL_{model}$ were known for the Track during the summer of 2001 (and X_2 was equal to 1), it was possible to solve for the X_1 term for each unique experimental section. Computed X_1 values for each test section were then regressed with corrected, composite values for primary consolidation, secondary consolidation and temperature shift factors (using Equations 5.6 through 5.15 as described above) in the following manner:

$$X_{1_{Model}} = a + (b \cdot x_1) + (c \cdot x_2) + (d \cdot x_3),\ where \qquad \text{Equation 5.18}$$

$x_1 = lab\ primary\ consolidation_{scaled,\ adjusted,\ composite}$

$x_2 = T_{shift_{composite}}$

$x_3 = lab\ secondary\ consolidation_{adjusted,\ composite}$

The *a*, *b*, *c* and *d* terms in Equation 5.18 are regression constants unique for each laboratory testing protocol. The resulting summer 2001 prediction completed for APA testing is plotted in Figure 5.16, which reveals a coefficient of determination of 0.77.

This was a good correlation and an important finding, since the model would be constructed by translating the summer 2001 correlation to other time increments. Details for the summer 2001 regression relating to all laboratory test protocols is included in the model summary provided in Table 5.3.

5.2.6 Age Factor Development

The next logical step in the process required providing a mechanism for adjusting the summer 2001 correlation to match performance at all other times. In effect, this means solving for the age-dependent term X_2, which was found to be equal to 1 for the summer of 2001 but should not be equal to 1 at all other ages. Since HMA is expected to stiffen as it ages, the age-dependent term X_2 would be expected to be greater than 1 for all ages younger than the summer of 2001 and less than 1 for all older ages. The age-dependent X_2 term could thus be represented by the following expression:

$$X_{2_{actual}} = \frac{dRut}{ESAL_{Model} \cdot X_{1_{Model}}} = age\ factor \qquad \text{Equation 5.19}$$

Equation 5.19 was then used to compute X_2 values for all other times, which were plotted for each experimental section in all time increments. It was found via Figure 5.17 that the best fit for the data appeared to be a power function taking the form:

$$X_{2_{Model}} = f \cdot age^{g} \quad for\ a\ power\ curve\ fit, then \qquad \text{Equation 5.20}$$

$$f = \frac{1}{301^g} \quad at\ 301\ days,\ which\ must\ be\ true\ for\ all\ ages,\ and \qquad \text{Equation 5.21}$$

$$X_{2_{Model}} = \frac{age^g}{301^g} = \left(\frac{age}{301}\right)^g \quad by\ substitution\ for\ all\ ages \qquad \text{Equation 5.22}$$

The terms f and g are constants that are unique for each laboratory testing protocol. Actual X_2 values at early ages were then used to solve for the g parameter in Equation 5.22. It was decided that the first performance week would be used to solve for the g term in order to contrast rutting performance after approximately one year with the youngest age possible (i.e., after three days at the midpoint of the first performance week):

$$X_{2_{actual\ at\ 3\ days}} = \frac{age^g}{301^g} = \frac{3^g}{301^g} = \left(\frac{3}{301}\right)^g = 0.01^g \qquad \text{Equation 5.23}$$

It was then possible to solve for the g parameter in the following manner:

$$g_{actual} = \log_{0.01}\left(X_{2_{actual\ at\ 3\ days}}\right) = \frac{\ln\left(X_{2_{actual\ at\ 3\ days}}\right)}{\ln(0.01)} \qquad \text{Equation 5.24}$$

Since the purpose of the exercise was to develop a model that could be run only using information that was available at the time of construction, it was then necessary to model the g term. This is because measured values for *dRut* made it possible to solve for

g at the Track; however, this will not be possible on actual projects because change in rutting will not be known. A unique value for g was calculated using Equation 5.24 for every experimental section on the Track's tangents. A regression model for the g term was then determined by trial and error using either the natural log of primary consolidation or the temperature shift parameter, whichever produced the best coefficient of determination. The final model parameter g could then be identified using the following equation:

$$g_{Model} = h + (i \cdot x_4), \textit{ where} \qquad \text{Equation 5.25}$$

$$x_4 = \ln(\textit{lab primary consolidation}_{\textit{scaled, adjusted, composite}}) \textit{ or } T_{\textit{shift}_{\textit{composite}}}$$

Equation 5.21 can then be used to compute f. Since g and f are effectively controlled by the change in rutting rate between very early performance and performance in the summer of 2001, it makes intuitive sense that primary consolidation would be an important factor. Secondary consolidation would be expected to have a less significant effect on early rutting rate.

5.2.7 Load-Temperature Spectra Model

With the approach defined and all terms identified, the final load-temperature spectra model for predicting rutting performance on the 2000 NCAT Pavement Test Track combines Equations 5.4, 5.5, 5.16, 5.18 and 5.22 to take the following form:

$$Rut = \sum \left(dRut \left(at\ 1 \ \ and\ 2\ weeks\ then\ monthly\ thereafter \right) \right), or$$

$$Rut = \sum_{age=3}^{age=n} \left(ESAL_{Model} \cdot X_1 \cdot X_2 \right), which\ with\ all\ terms\ shown\ becomes$$

$$Rut = \sum_{age=3}^{age=n} \left(\begin{array}{l} \sum_{T_{mid\ PG^\circ C=37}}^{T_{mid\ PG^\circ C=Max}} \left(ESALs_{Hot\ Banded>34^\circ C} \cdot \left(\left(1.76 \times 10^{-5} \right) \cdot Temp_{mid\ PG^\circ C}{}^{3.01} \right) \right) \times \\ \left(\begin{array}{l} a + \left(b \cdot LabConsolidation_{primary\ composite} \right) + \left(c \cdot T_{shift_{composite}} \right) + \\ \left(d \cdot LabConsolidation_{\sec ondary\ composite} \right) \end{array} \right) \times \\ \left(\frac{age}{301} \right)^{g_{Model}} \end{array} \right) \qquad \text{Equation 5.26}$$

Every test protocol run for the 2000 Track and shown in Tables 4.9, 4.10 and 4.11 was processed using this methodology. Primary consolidation in the laboratory was scaled to correct for lab-to-field VTM differences using Equation 5.6 and temperature adjusted for depth of placement using Equation 5.11. Secondary consolidation in the laboratory did not require scaling for VTM differences and was simply temperature adjusted for depth of placement using Equation 5.11. A single value for laboratory primary consolidation, laboratory secondary consolidation and temperature shift factor was produced for multi-layered pavement structures by weighting the terms based on differences in anticipated stress levels using Equations 5.14 and 5.15. The terms resulting from this process were then regressed with performance for the summer of 2001 to identify the coefficients necessary for Equation 5.18. Either primary consolidation or temperature shift factor were then regressed with the result of Equation 5.24 to identify the coefficients necessary for the h_{model} product of Equation 5.25.

This process was repeated for each protocol and predictions were generated for all time increments using actual load-temperature spectra model inputs. Final Track rutting predictions were simply the sum of all the individual incremental predictions. In order to evaluate the suitability of each individual model, the upper binder failure grade alone was utilized to generate a control model prediction. Coefficients of determination from each model were used to rank them from least effective (lower coefficients) to most effective (higher coefficients). Results from this effort are presented in Table 5.3, in which it is seen the control model ranked 27th out of 32 total predictions. The only models to rank lower (i.e., lower coefficients of determination) than the control were GSI_{air}, $GSI_{air\ c\text{-}t}$, Dynamic $Modulus_{10\ Hz}$, VTM and SST_{RSCH}. The highest coefficient of determination (0.639) was achieved using data produced from RLWT testing. The coefficient of determination for predictions based on RLCC testing was 0.636, while the coefficient of determination resulting from APA data was 0.552.

In Table 5.3, it is seen that results from the RLWT, RLCC, SGC (Sr_{max} as well as rate of height change), and rate of change in the GTM with the air cell all produce model predictions that exhibit coefficients of determination better than 0.5. This can be interpreted to mean that the model for these protocols explains more than half the variability observed in total rut depth on the Track.

Model predictions for final rut depths in the APA are shown as an example in Figure 5.18. A coefficient of determination of 0.552 was observed using all data points. This appears to be a reasonable predictive effort, with only three points (representing sections N5, N13 and S5) lying outside of a tightly grouped band (circled in Figure 5.18).

The "load-temperature spectra" approach described in this chapter appears to compare well with other correlation efforts described in the literature. For example, a comprehensive modeling effort was performed as part of the calibration effort for AASHTO's Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. Data from the 88-section testing program is shown in Figure 5.19, in which it is seen that final rut depths (with plastic deformation contributions from the base as well as the subgrade and no outlier exclusions) were predicted with a coefficient of determination of 0.09 (ARA, 2004). Figure 5.19 illustrates that Track rut depths were generally much less than those used to validate the rutting model in the MEPDG study, which makes it impossible to do a direct comparison of model effectiveness.

The best correlations with total Track rutting from testing during gyratory compaction, simulative testing and fundamental testing are presented in Figures 5.20, 5.21 and 5.22, respectively. Residuals from the model used to construct Figure 5.21 were computed and plotted as a function of cumulative probability distribution in Figure 5.23 in order to check the adequacy of the approach. With the exception of three outliers near the ends of both tails, the points were observed to fall along a normally distributed straight line. Residuals were also plotted against predicted values on a secondary axis in Figure 5.24. Because there does not appear to be a relationship between variance and magnitude, nonconstant variance is not likely. This would be the case if error in the experiment was a constant percentage of the size of the observation. Since the plot seemed to be structureless, there was no reason to suspect the residuals were related to any other variable. Based on this evidence, it was concluded that the model adequately described the observations.

5.2.8 Practical Application

This methodology should be useful to owners who are responsible for managing infrastructure networks as well as to contractors who are engaged in warranty work. In both cases, it is likely that some critical rut depth exists that will require corrective action. In order to predict rutting over the life of a project, it is first necessary to determine the 24-hour distribution over which daily ESALs will be applied. For each day of traffic, anticipated hourly pavement surface temperatures are utilized to group ESALs into temperature bands as described in section 5.2.1. Growth factors can be used to inflate ESAL values for future time increments.

Coefficients from Table 5.3 are then selected based on the type of protocol utilized. Assumptions must be made regarding field VTM; otherwise, the only other input necessary is the depth to the second layer. After the first four weeks of traffic, incremental rutting is computed for each month of service. Finally, total rutting is simply calculated as the sum of all incremental rutting.

In order to improve predictive outcomes, rutting performance models should eventually be validated using QC samples produced during the construction of the 2003 Track. Additionally, predictions should be extended for longer periods of time by continuing to monitor all sections that were left in place and subjected to another round of truck traffic on the 2003 Track. In this manner, the age adjustment factor X_2 could be refined to better predict long-term rutting performance. By continuing to study these older sections under additional truck traffic, it may be possible to encompass durability using select laboratory testing protocols. With the completion of the 2003 Track, these data are now available; however, a significant amount of effort will be required to construct the necessary data tables.

TABLE 5.1 Construction of Temperature Factor for Weighting Hot ESALs

Lower and Upper Temperatures of Bands (°C)	34 to 40	40 to 46	46 to 52	52 to 58	58 to 64	64 to 70
Midpoint of Temperature Bands (°C)	37	43	49	55	61	67
$\text{Stiffness}_{est} = 2{,}793{,}344 / e^{(0.05976 \times {}^{\circ}C)}$	306,089	213,859	149,419	104,396	72,940	50,962
$\text{Compliance} = 1 / \text{Stiffness} = e^{(0.05976 \times {}^{\circ}C)} / 2{,}793{,}344$	3.27E-06	4.68E-06	6.69E-06	9.58E-06	1.37E-05	1.96E-05
Multiplier = Band Compliance / Compliance @ 37°C	1.000	1.431	2.049	2.932	4.196	6.006
$\text{Modeled Multiplier} = 1.76\text{E-}5 \times T_{mid\ PG\ {}^{\circ}C}^{\ 3.01}$	0.950	1.454	2.139	3.044	4.213	5.692

TABLE 5.2 Elastic Response Predictions from WESLEA Analysis

WESLEA for Windows - Simulation Output

File: C:\Buzz\PhD Stuff\Weslea analysis of Track buildup.xls
Date: Sat Nov 27 22:33:22 2004

STRUCTURAL INFORMATION

Layer	Modulus * (psi)	Poisson	Height (in)	Slip
1	500003	0.35	4	1
2	750004	0.35	6	1
3	500003	0.35	9	1
4	29994	0.4	10	0
5	8006.1	0.45	Infinite	

* Modulus values assumed as in early analyses of Track structure

LOADING CONFIGURATION

Axle Ty| Single

Tire#	X (in)	Y (in)	Load (lb)	Pressure (psi)
1	0	0	5000	100
2	13.5	0	5000	100

PREDICTED PAVEMENT LIFE

Non-standard locations were selected.
Empirical transfer functions are not applicable.

ENGINEERING RESPONSES

		Coordinates (in)			Normal Stress (psi)			Normal MicroStrain			Displacement (milli-in)			Shear Stress (psi)		
Loc#	Layer	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	YZ	XZ	XY
1	1	0	0	0	95.51	98.7	100	51.92	60.56	64.05	0.21	0	9.61	0	0	0
2	1	0	0	1	66.39	68.46	99.25	15.39	20.96	104.11	0.18	0	9.52	0	-0.62	0
3	1	0	0	2	44.72	45.81	93.07	-7.78	-4.82	122.76	0.14	0	9.41	0	-1.25	0
4	1	0	0	3	33.11	33.45	81.53	-14.27	-13.34	116.47	0.12	0	9.29	0	-1.87	0
5	1	0	0	4	30.01	29.84	67.68	-8.24	-8.7	93.46	0.09	0	9.18	0	-2.44	0
6	2	0	0	5	18.6	17.39	54.16	-8.6	-10.77	55.42	0.06	0	9.12	0	-3.14	0
7	2	0	0	6	12.65	10.74	42.84	-8.14	-11.58	46.21	0.04	0	9.07	0	-3.7	0
8	2	0	0	7	7.7	5.27	33.74	-7.94	-12.3	38.93	0.01	0	9.03	0	-4.09	0
9	2	0	0	8	3.08	0.26	26.52	-8.39	-13.47	33.8	-0.01	0	8.99	0	-4.31	0
10	2	0	0	9	-1.61	-4.79	20.86	-9.64	-15.37	30.8	-0.03	0	8.96	0	-4.35	0
11	3	0	0	10	-1.5	-3.87	16.6	-11.9	-18.31	36.94	-0.05	0	8.93	0	-4.24	0
12	3	0	0	11	-3.54	-5.75	13.36	-12.4	-18.38	33.21	-0.07	0	8.89	0	-4.12	0
13	3	0	0	12	-5.65	-7.72	10.67	-13.37	-18.96	30.71	-0.08	0	8.86	0	-3.93	0
14	3	0	0	13	-7.85	-9.81	8.43	-14.74	-20.03	29.23	-0.1	0	8.83	0	-3.68	0
15	3	0	0	14	-10.16	-12.06	6.54	-16.46	-21.58	28.63	-0.11	0	8.8	0	-3.34	0
16	3	0	0	15	-12.6	-14.5	4.96	-18.52	-23.64	28.88	-0.13	0	8.77	0	-2.94	0
17	3	0	0	16	-15.19	-17.17	3.66	-20.93	-26.27	29.97	-0.15	0	8.74	0	-2.45	0
18	3	0	0	17	-17.99	-20.16	2.64	-23.72	-29.57	31.99	-0.17	0	8.71	0	-1.87	0
19	3	0	0	18	-21.04	-23.52	1.94	-26.98	-33.66	35.06	-0.19	0	8.68	0	-1.19	0
20	3	0	0	19	-24.41	-27.36	1.59	-30.79	-38.75	39.43	-0.22	0	8.64	0	-0.38	0

TABLE 5.3 Summary of Model Developed for Each Test Method

		Laboratory	Constant		Primary or Only		Tshift		Secondary		Time Effect Prediction			Fit Statistics			Final	R^2
A Col	B Col	Protocol	a value	a prob	b value	b prob	c value	c prob	d value	d prob	g key	g int	g slope	Sum01 R^2	g R^2	dRut R^2	Model R^2	Rank
50		APA_{8k}	8.3E-07	0.02	2.0E-07	0.00	-8.3E-08	0.00	NA	NA	ln(prim)	-0.870	0.227	0.74	0.31	0.65	0.54	8
49	51	APA_{8k-4k}	6.0E-07	0.09	1.3E-07	0.06	-6.8E-08	0.00	1.4E-06	0.02	ln(prim)	-0.887	0.263	0.77	0.32	0.66	0.55	7
55		$Hamburg_{8k}$	1.0E-06	0.03	1.2E-07	0.02	-9.3E-08	0.00	NA	NA	ln(prim)	-1.116	0.314	0.64	0.40	0.60	0.47	19
54	56	$Hamburg_{8k-4k}$	9.6E-07	0.04	1.6E-07	0.01	-9.1E-08	0.00	-1.6E-07	0.67	ln(prim)	-1.327	0.458	0.66	0.42	0.60	0.47	25
60		$Rutmeter_{8k}$	6.2E-07	0.06	2.9E-07	0.00	-6.8E-08	0.00	NA	NA	ln(prim)	-0.822	0.241	0.78	0.38	0.68	0.64	1
59	61	$Rutmeter_{8k-4k}$	6.1E-07	0.10	2.6E-07	0.08	-6.5E-08	0.00	7.8E-07	0.10	ln(prim)	-0.893	0.359	0.79	0.49	0.68	0.64	2
65		$Wessex_{8k}$	1.5E-06	0.00	8.7E-08	0.12	-1.0E-07	0.00	NA	NA	Tcomp	-0.343	-0.026	0.60	0.24	0.60	0.50	11
64	66	$Wessex_{8k-4k}$	1.4E-06	0.00	6.8E-08	0.46	-9.9E-08	0.00	6.2E-07	0.64	ln(prim)	-0.734	0.137	0.60	0.12	0.60	0.49	13
74		Dyn Mod_{10}	1.8E-06	0.00	5.5E-07	0.74	-1.2E-07	0.00	NA	NA	Tcomp	-0.392	-0.019	0.55	0.24	0.58	0.47	17
75		Dyn Mod_5	1.9E-06	0.00	5.0E-07	0.81	-1.2E-07	0.00	NA	NA	Tcomp	-0.391	-0.019	0.55	0.25	0.58	0.47	23
76		Dyn Mod_1	1.8E-06	0.00	1.2E-06	0.71	-1.2E-07	0.00	NA	NA	Tcomp	-0.394	-0.019	0.55	0.25	0.58	0.47	23
77		Dyn $Mod_{0.5}$	1.8E-06	0.00	1.5E-06	0.70	-1.2E-07	0.00	NA	NA	Tcomp	-0.394	-0.018	0.56	0.25	0.58	0.47	25
78		Dyn $Mod_{0.1}$	1.7E-06	0.00	2.4E-06	0.61	-1.2E-07	0.00	NA	NA	Tcomp	-0.397	-0.018	0.56	0.25	0.58	0.47	20
74	78	Dyn $Mod_{10-0.1}$	1.8E-06	0.00	-1.1E-06	0.77	-1.2E-07	0.00	5.4E-06	0.64	Tcomp	-0.398	-0.018	0.56	0.27	0.58	0.46	30
82		$PSPA_{mid}$	1.4E-06	0.00	5.9E-10	0.07	-1.0E-07	0.00	NA	NA	Tcomp	-0.411	-0.018	0.61	0.23	0.59	0.47	20
83		$PSPA_{top}$	1.2E-06	0.00	6.9E-10	0.02	-1.0E-07	0.00	NA	NA	Tcomp	-0.399	-0.020	0.64	0.22	0.61	0.49	14
79		SST_{RSCH}	1.2E-06	0.01	3.2E-07	0.05	-8.7E-08	0.00	NA	NA	Tcomp	-0.420	-0.016	0.62	0.17	0.59	0.44	32
72		Triaxial U_{10k}	1.3E-06	0.00	9.1E-12	0.01	-8.3E-08	0.00	NA	NA	ln(prim)	-1.615	0.103	0.66	0.24	0.64	0.61	4
73		$Triaxial_{slope}$	1.4E-06	0.00	9.0E-08	0.02	-8.3E-08	0.00	NA	NA	ln(prim)	-0.613	0.082	0.66	0.26	0.64	0.60	5
71	73	$Triaxial_{inter-slope}$	1.0E-06	0.01	4.2E-11	0.08	-9.5E-08	0.00	7.6E-08	0.03	Tcomp	-0.432	-0.015	0.70	0.18	0.66	0.64	2
80		WS_p	1.3E-06	0.01	5.1E-10	0.16	-1.0E-07	0.00	NA	NA	Tcomp	-0.411	-0.017	0.59	0.26	0.59	0.47	17
81		WS_t	1.2E-06	0.01	6.4E-10	0.09	-9.7E-08	0.00	NA	NA	Tcomp	-0.403	-0.019	0.61	0.28	0.59	0.47	16
127		GSI_{air}	8.6E-07	0.28	1.0E-06	0.16	-1.2E-07	0.00	NA	NA	Tcomp	-0.394	-0.019	0.59	0.31	0.59	0.46	28
118	127	$GSI_{air\ c-t}$	8.2E-07	0.55	5.8E-08	0.97	-1.2E-07	0.00	1.0E-06	0.18	Tcomp	-0.394	-0.019	0.59	0.31	0.59	0.46	29
136		GSI_{cd}	2.3E-06	0.10	-3.6E-07	0.77	-1.2E-07	0.00	NA	NA	Tcomp	-0.386	-0.020	0.55	0.24	0.58	0.47	20
135		$GTM_{air\ rate}$	1.5E-06	0.00	2.3E-05	0.09	-1.1E-07	0.00	NA	NA	Tcomp	-0.419	-0.015	0.61	0.23	0.60	0.48	15
117	135	$GTM_{c\ zero\ -\ air\ rate}$	1.8E-07	0.84	4.0E-09	0.09	-1.3E-07	0.00	1.0E-04	0.04	Tcomp	-0.337	-0.028	0.65	0.22	0.62	0.51	9
116		$N\text{-}Sr_{max}$	2.2E-06	0.00	-2.4E-09	0.05	-1.2E-07	0.00	NA	NA	Tcomp	-0.399	-0.018	0.62	0.27	0.61	0.50	12
97	98	$SGC_{0.50-75}$	1.1E-06	0.23	-2.7E-08	0.73	-1.1E-07	0.00	9.0E-07	0.23	Tcomp	-0.421	-0.016	0.67	0.22	0.61	0.51	9
115		Sr_{max}	3.0E-06	0.00	-1.4E-06	0.01	-1.1E-07	0.00	NA	NA	Tcomp	-0.419	-0.016	0.67	0.23	0.63	0.59	6
35		VTM	2.7E-06	0.00	-2.1E-07	0.14	-1.2E-07	0.00	NA	NA	Tcomp	-0.411	-0.017	0.59	0.24	0.59	0.45	31
dT		**control model**	**1.9E-06**	**0.00**	**-1.2E-07**	**0.00**	NA	NA	NA	NA	**Tcomp**	**-0.385**	**-0.020**	**0.55**	**0.26**	**0.58**	**0.46**	27

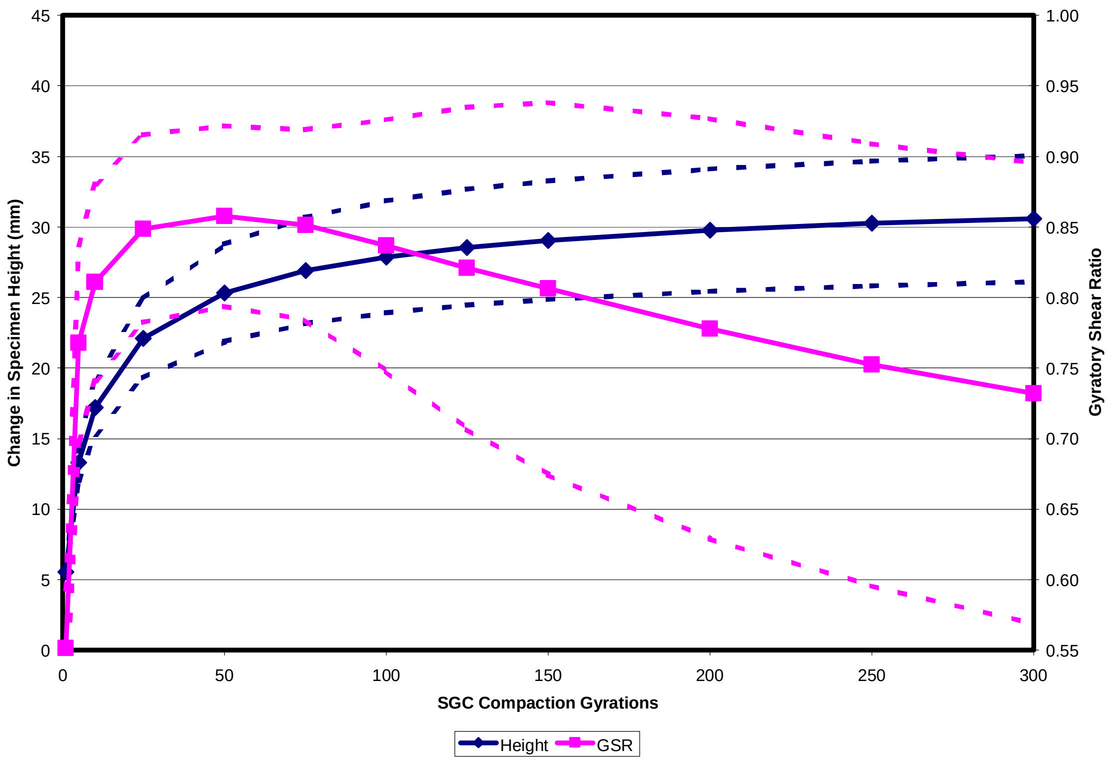

FIGURE 5.1 Plot of Average Height Change and Average Shear Strength During Gyratory Compaction of All Track Tangent Mixes (Shown with ± 1 Standard Deviation).

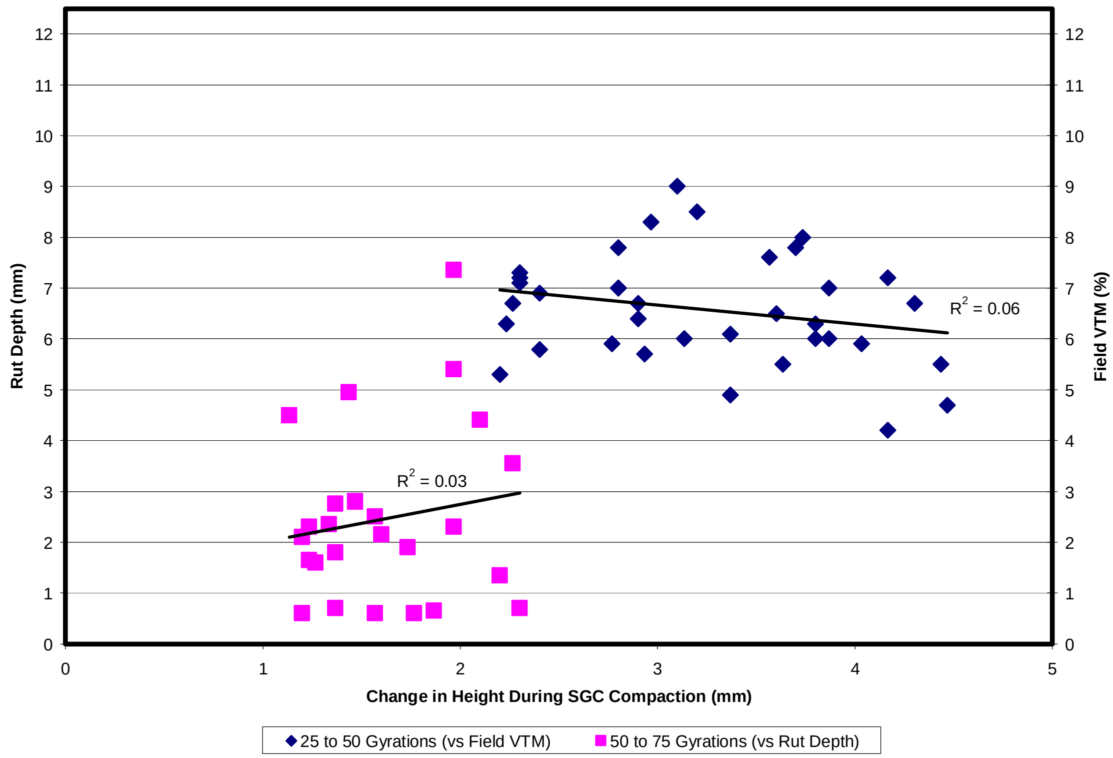

FIGURE 5.2 Inferring Mat Constructability and Performance from SGC Data.

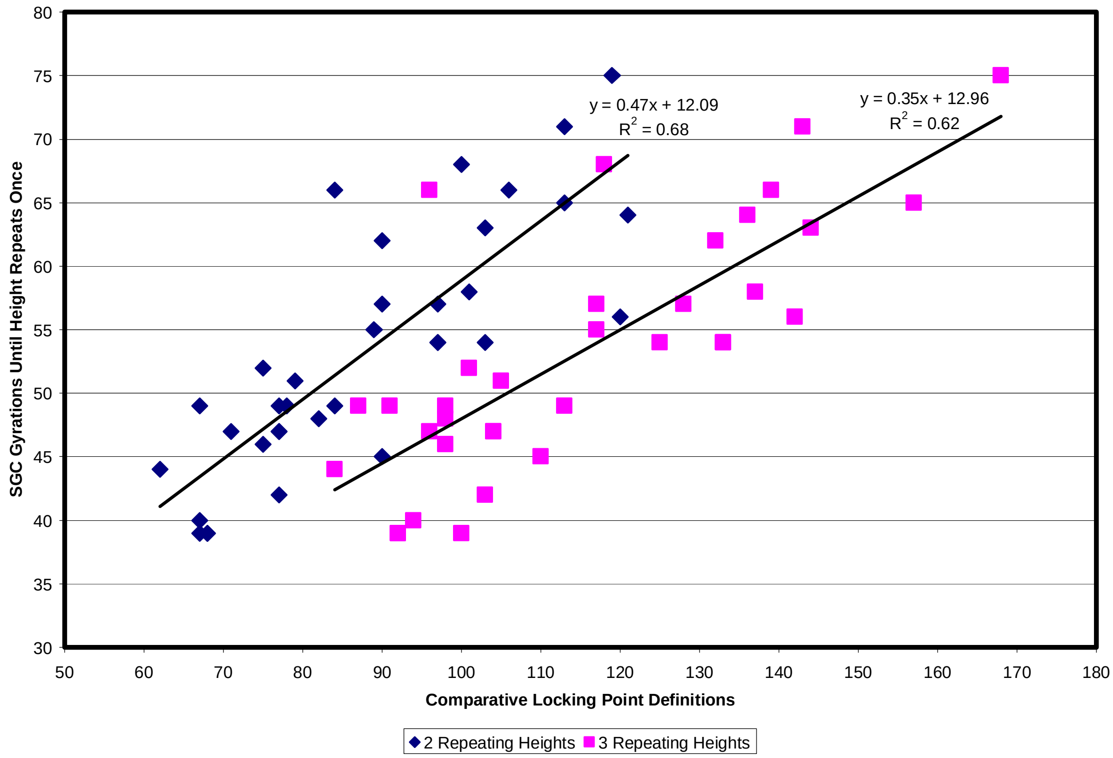

FIGURE 5.3 Comparison of Locking Point Methodologies Using All Track Mixes.

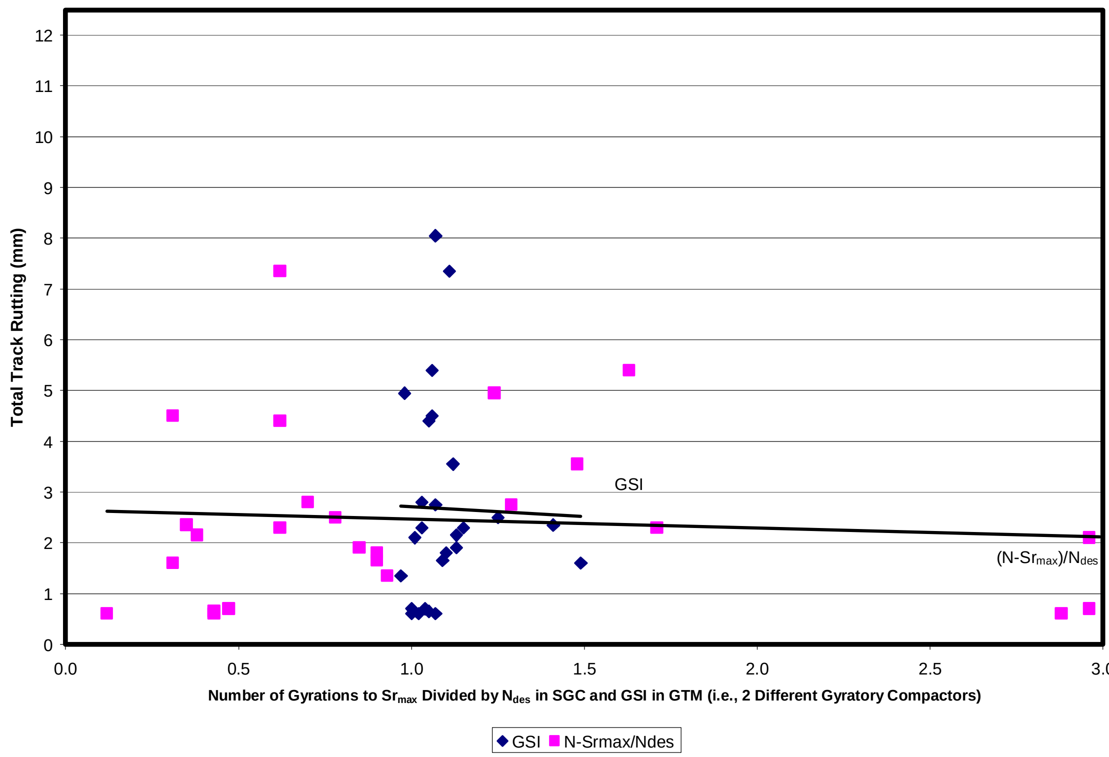

FIGURE 5.4 Correlation of Results Generated During Gyratory Compaction with Rutting Performance.

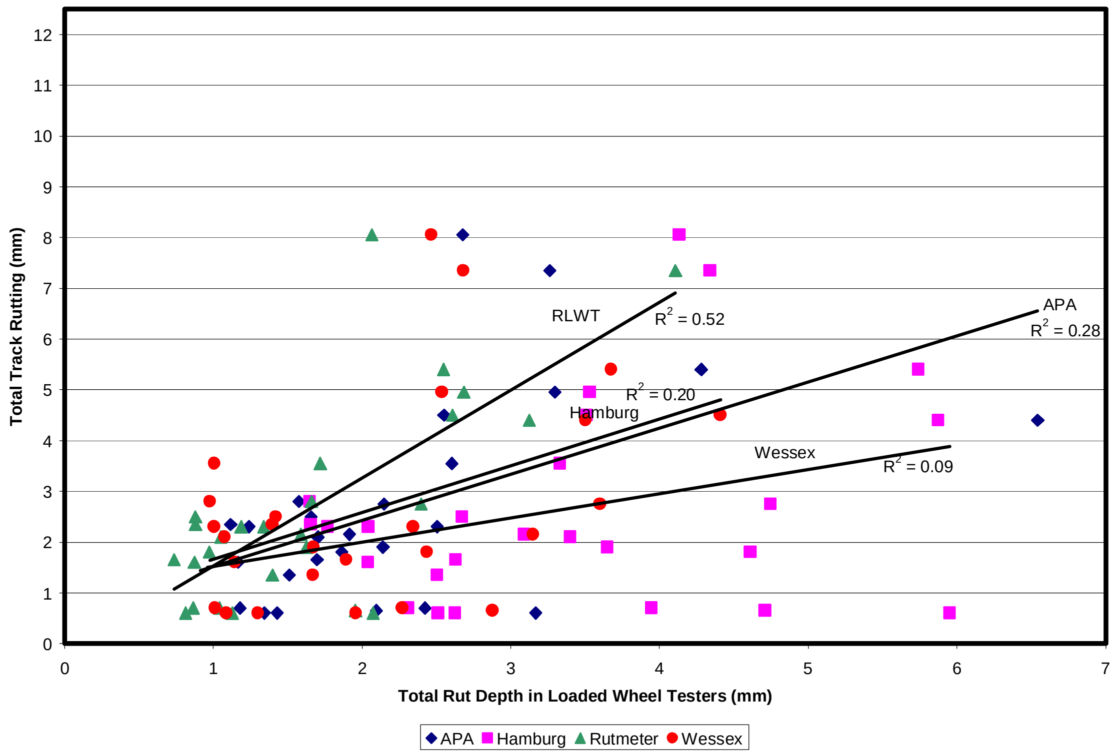

FIGURE 5.5 Correlation of Simulative Results with Rutting Performance.

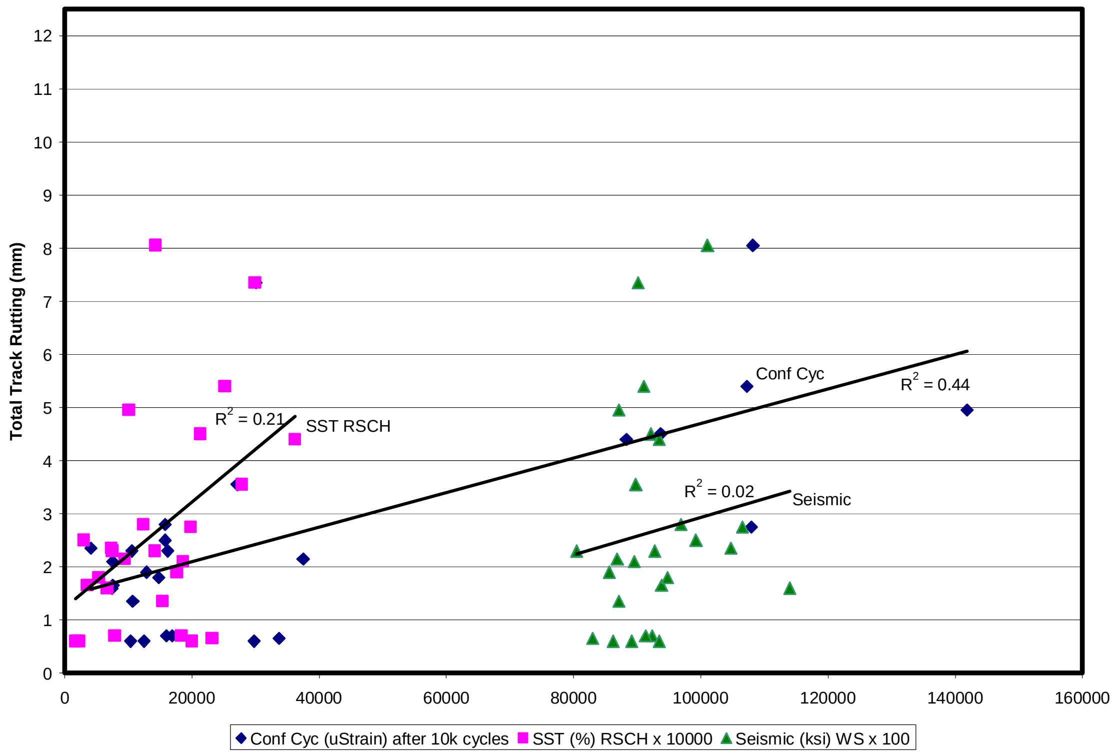

FIGURE 5.6 Correlation of Fundamental Results with Rutting Performance.

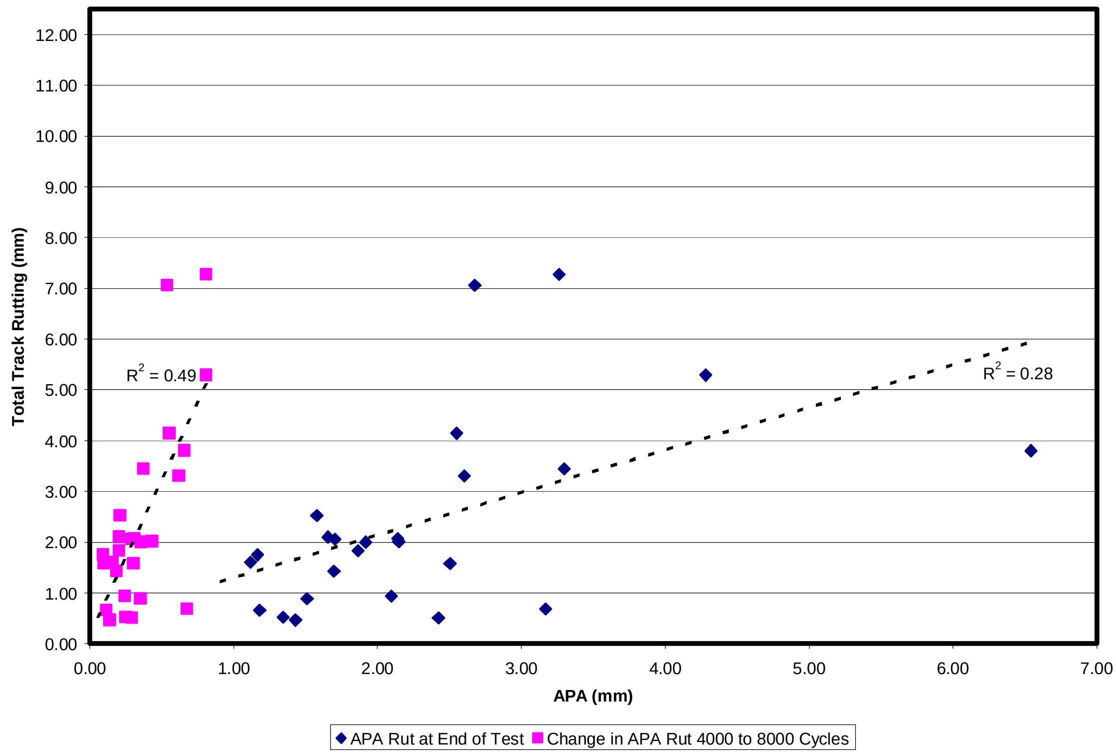

FIGURE 5.7 Development of Devised Method Results Using APA and Total Track Rutting as Example.

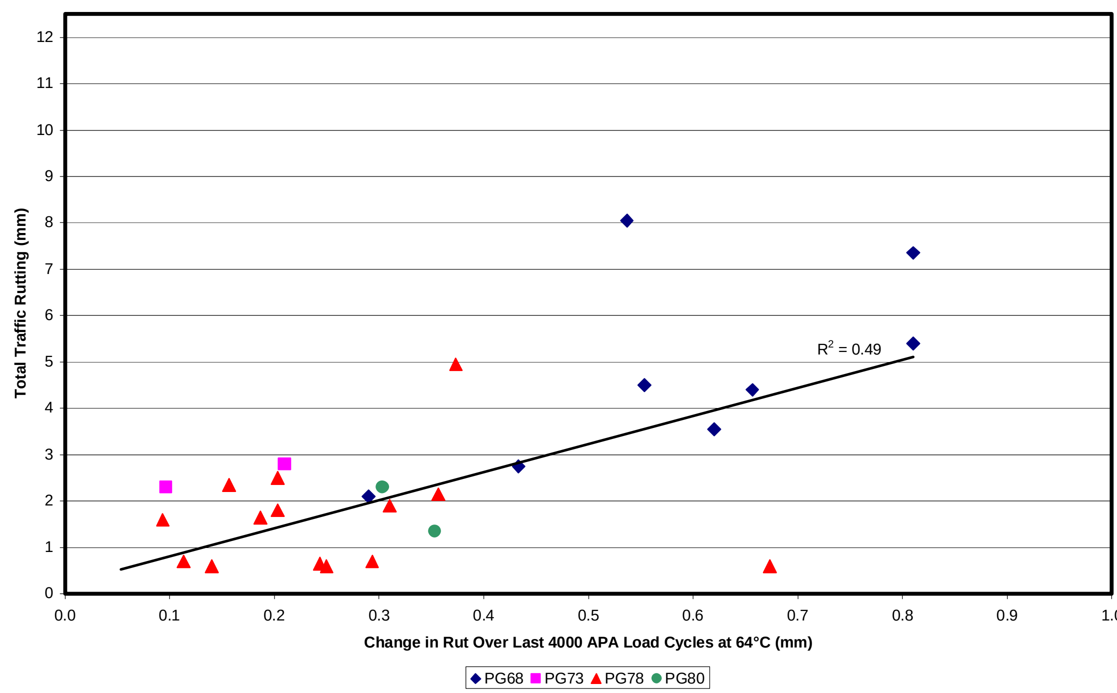

FIGURE 5.8 Plot of APA Slope versus Total Track Rutting for Different Binder Grades.

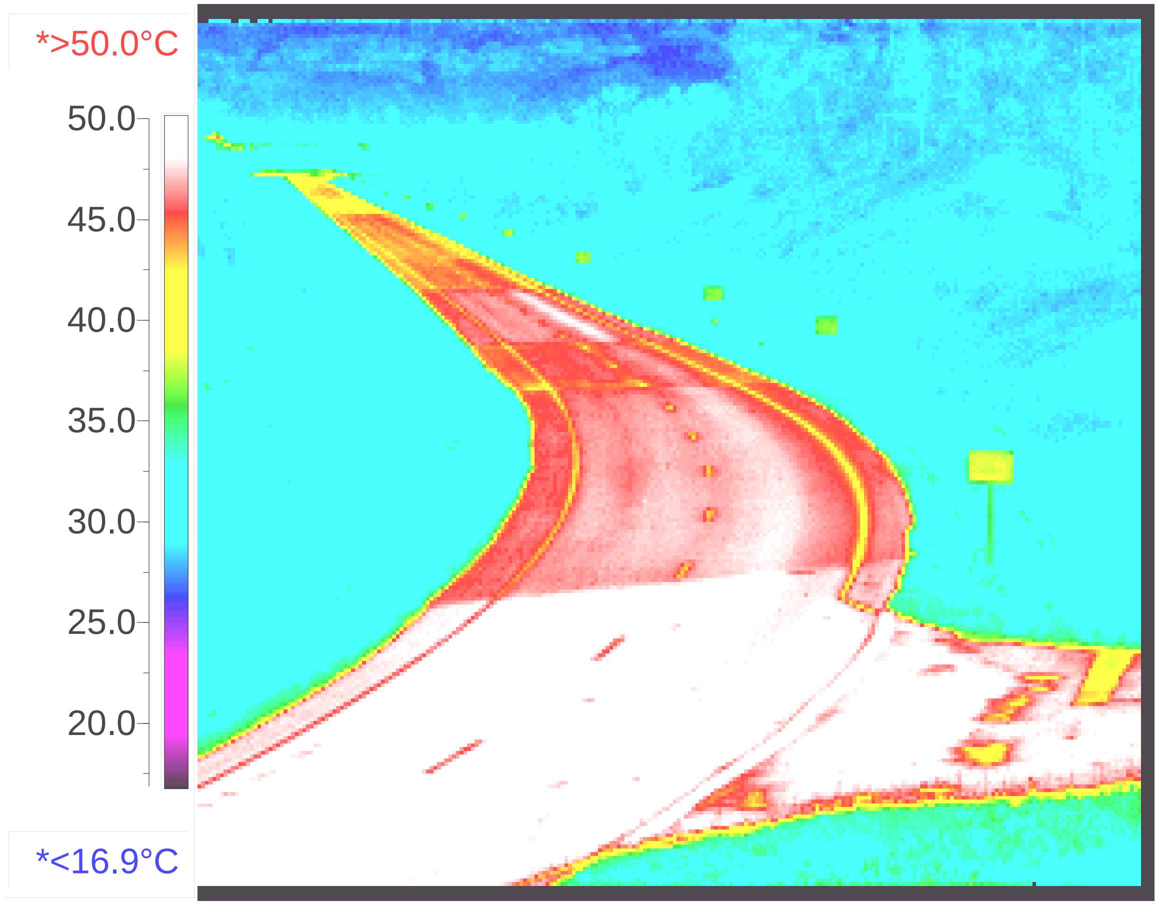

FIGURE 5.9 Infrared Image of Track's South Tangent Showing Unique Temperature Environment for Each Section.

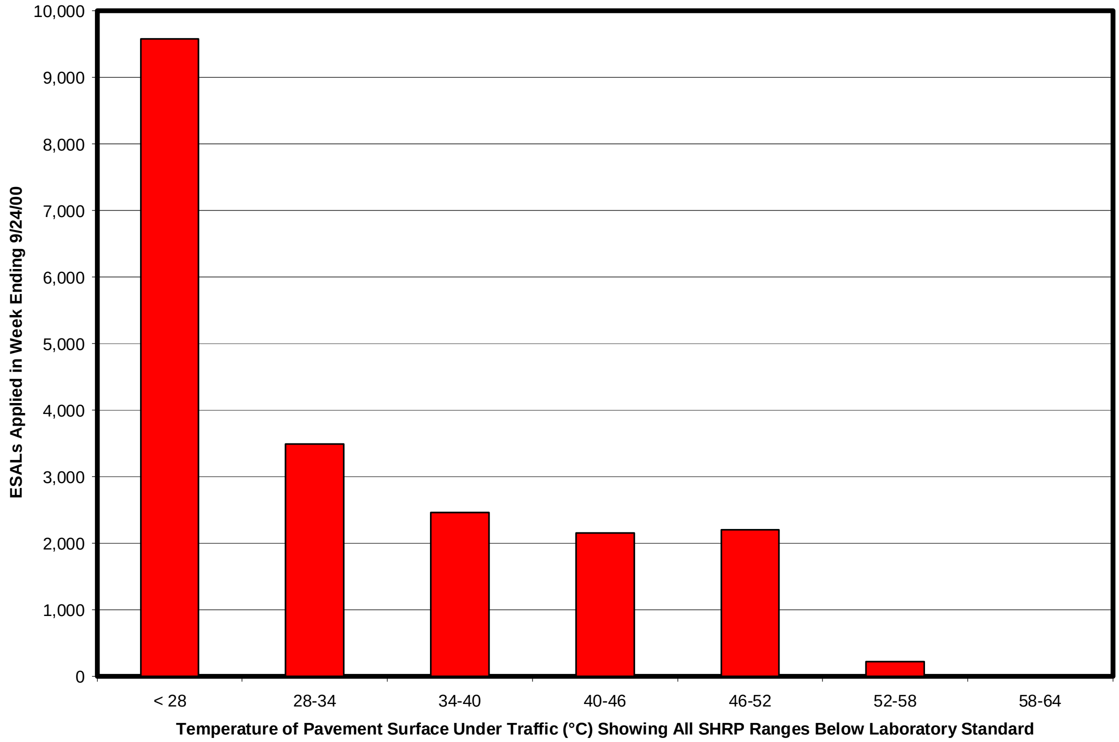

FIGURE 5.10 Sample Distribution of Traffic Over Changing Temperatures.

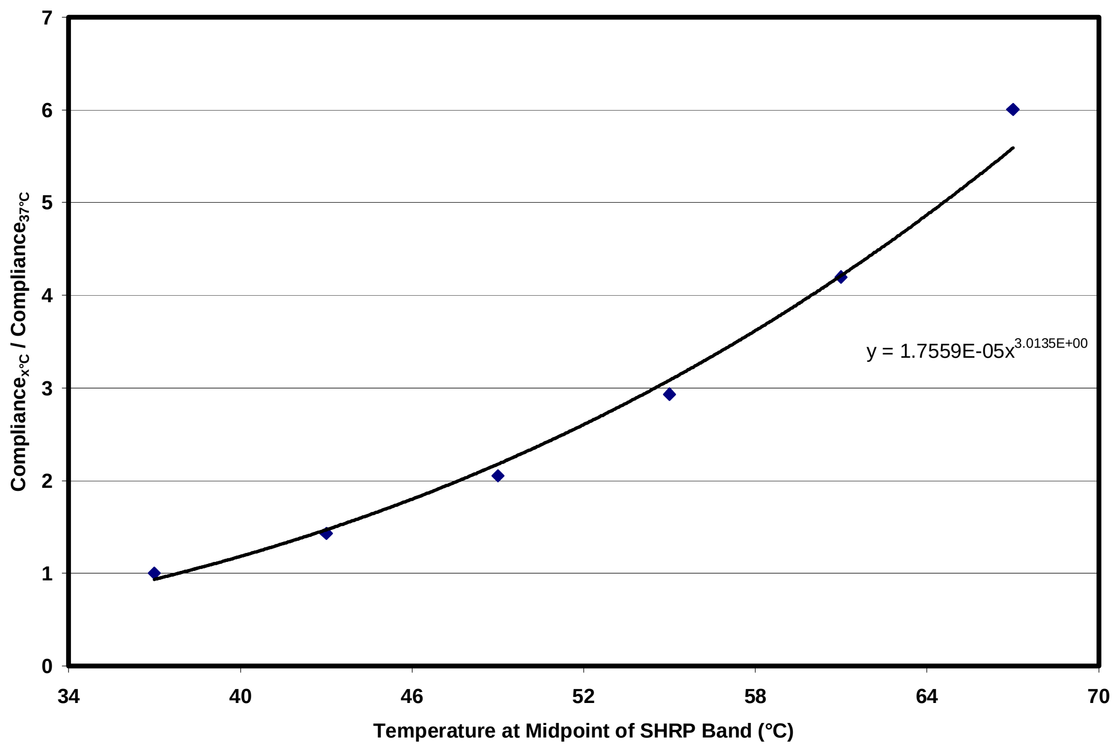

FIGURE 5.11 Utilization of HMA Compliance to Obtain a Weight Factor that Characterizes Rutting Damage from ESALs Applied at Different Temperatures.

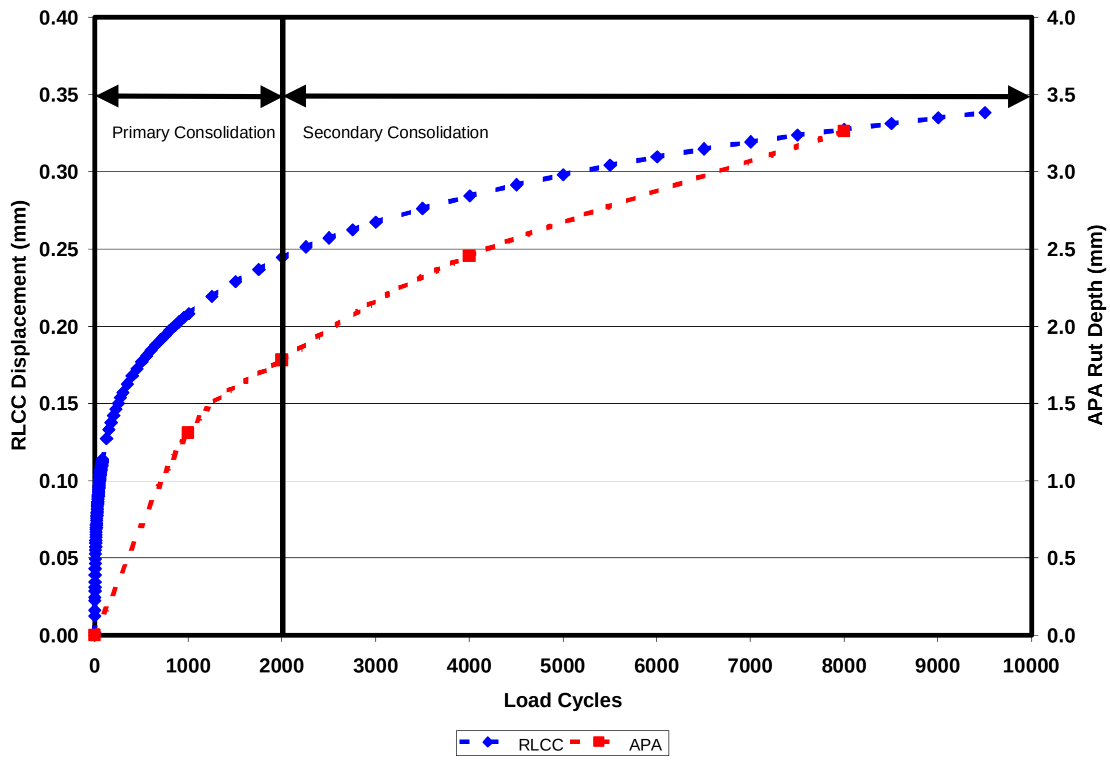

FIGURE 5.12 Sample Plot from Section N3 of APA and Triaxial Testing Illustrating Generalized Primary and Secondary Consolidation.

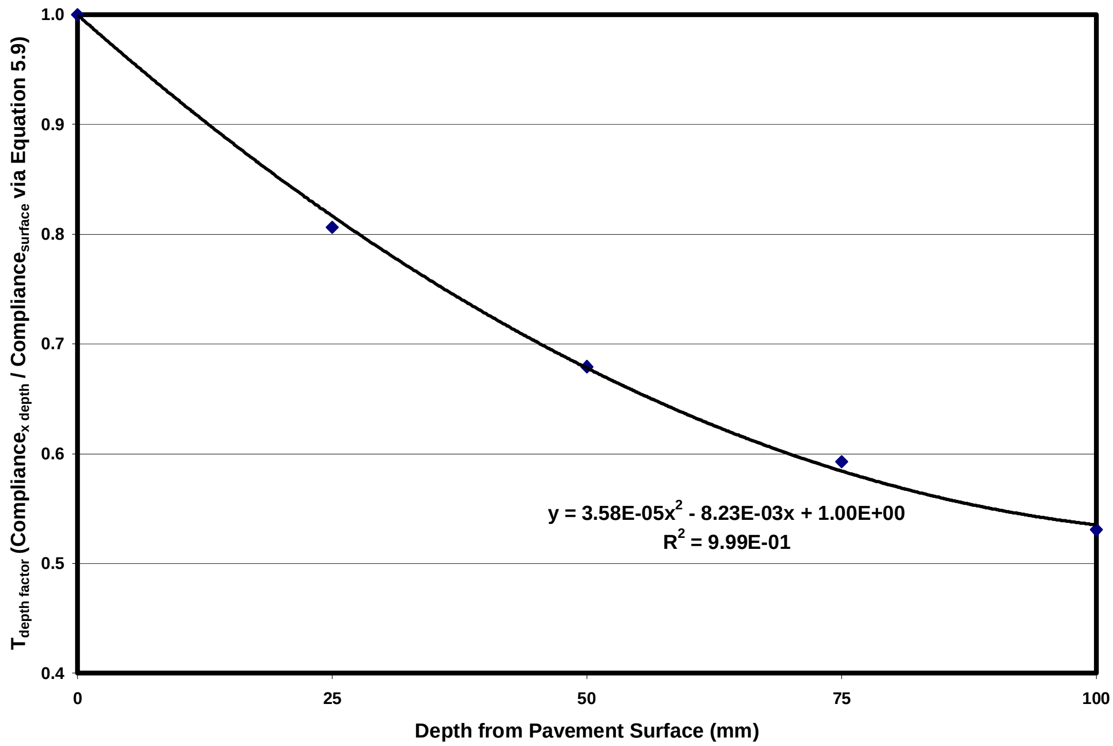

FIGURE 5.13 Method for Adjusting Laboratory Performance as a Function of Reduced Temperature of Underlying Layer.

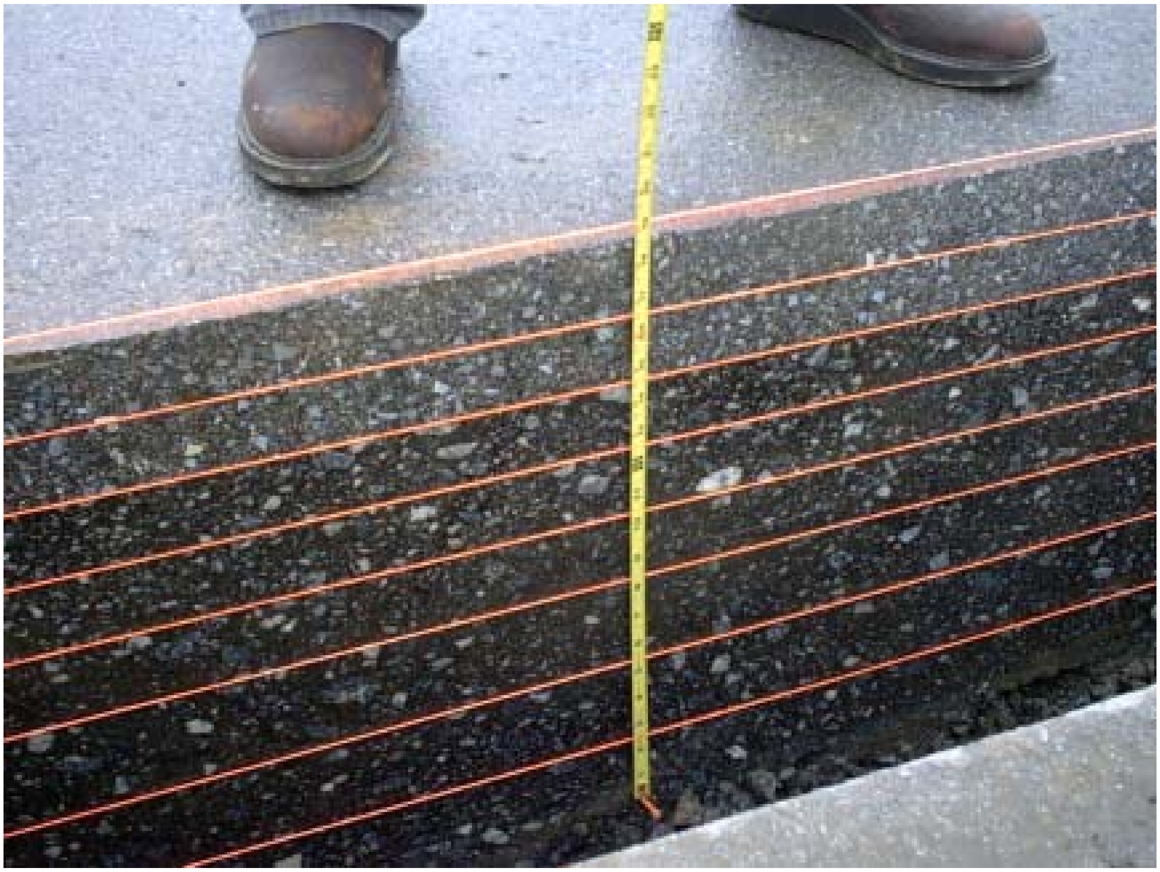

FIGURE 5.14 Trenching to Verify that Deformation Was Limited to the Top 100 mm.

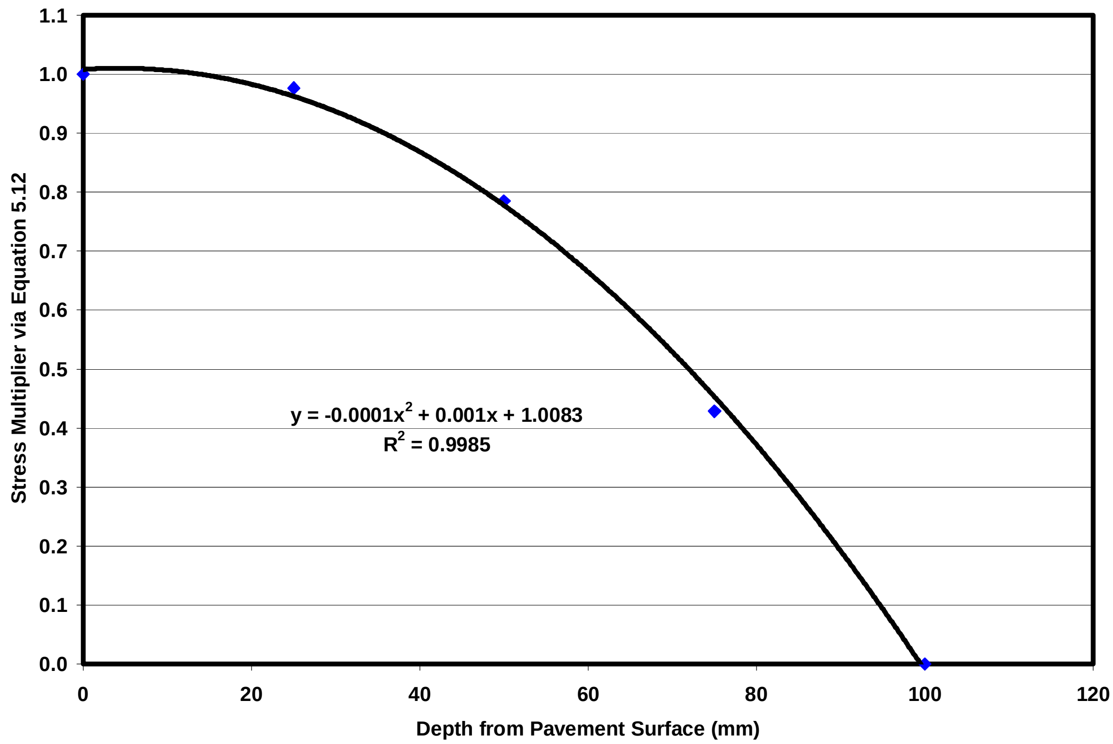

FIGURE 5.15 Weight Factor for Averaging Laboratory Performance Parameters Based on Top Depth of Underlying Layer.

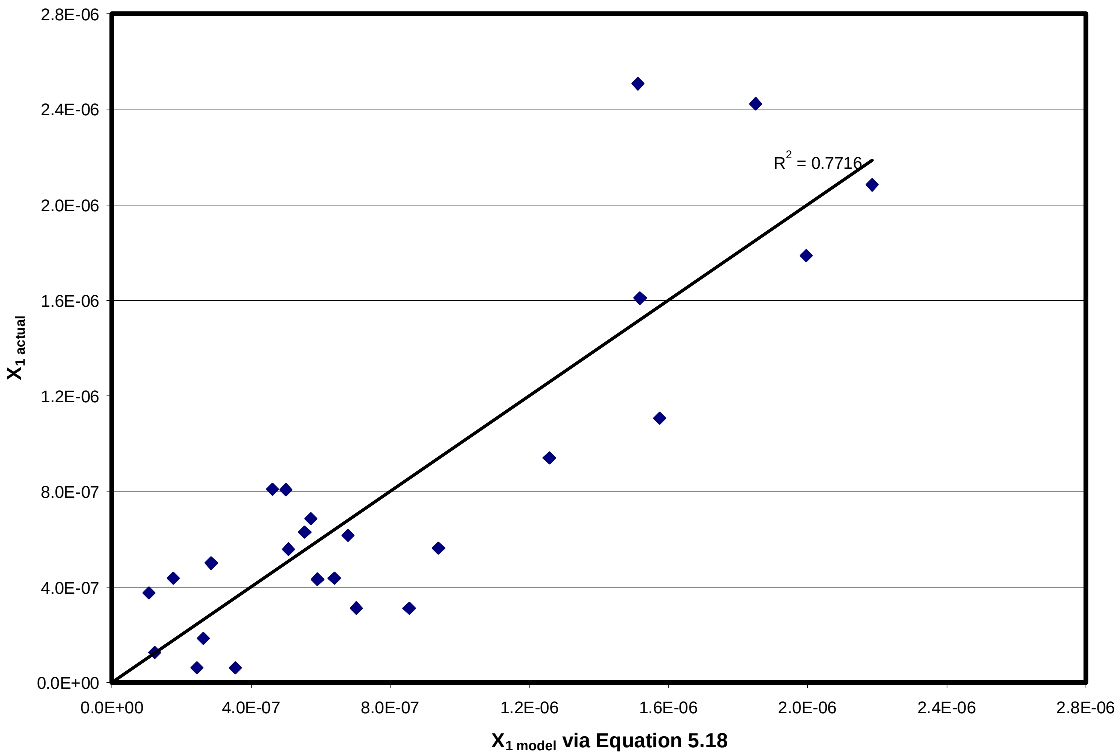

FIGURE 5.16 Final Model to Predict Summer 2001 Performance from APA Data.

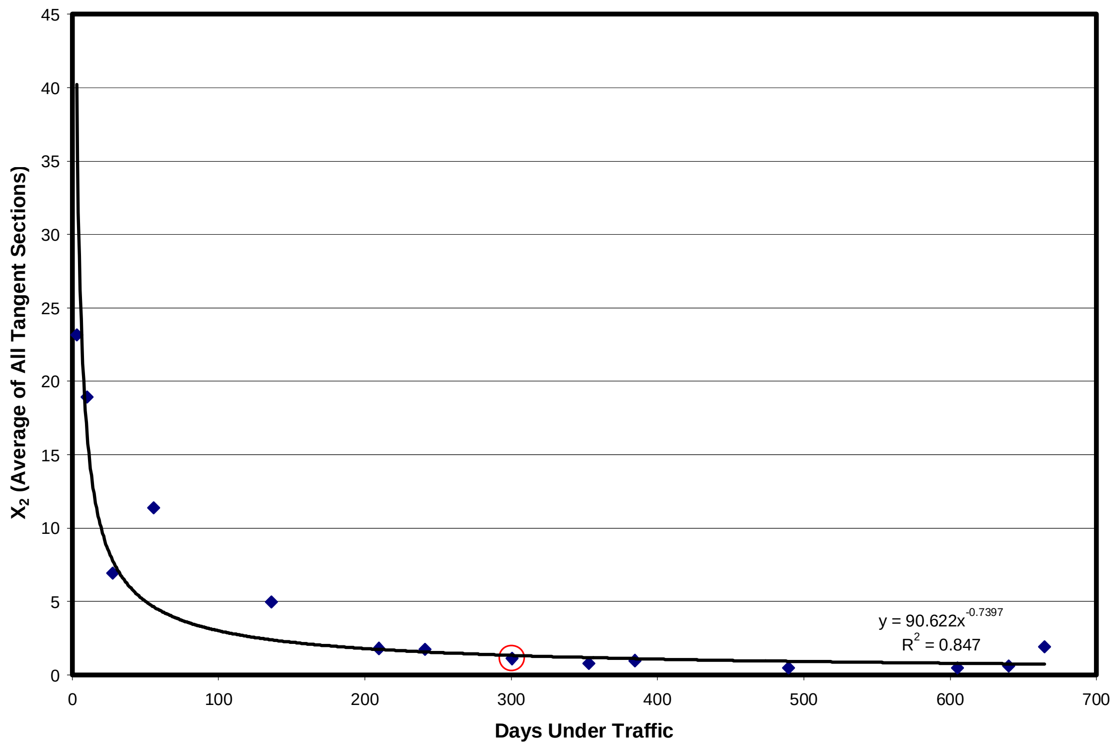

FIGURE 5.17 Effect of Aging on Lab-to-Field Correlation with APA.

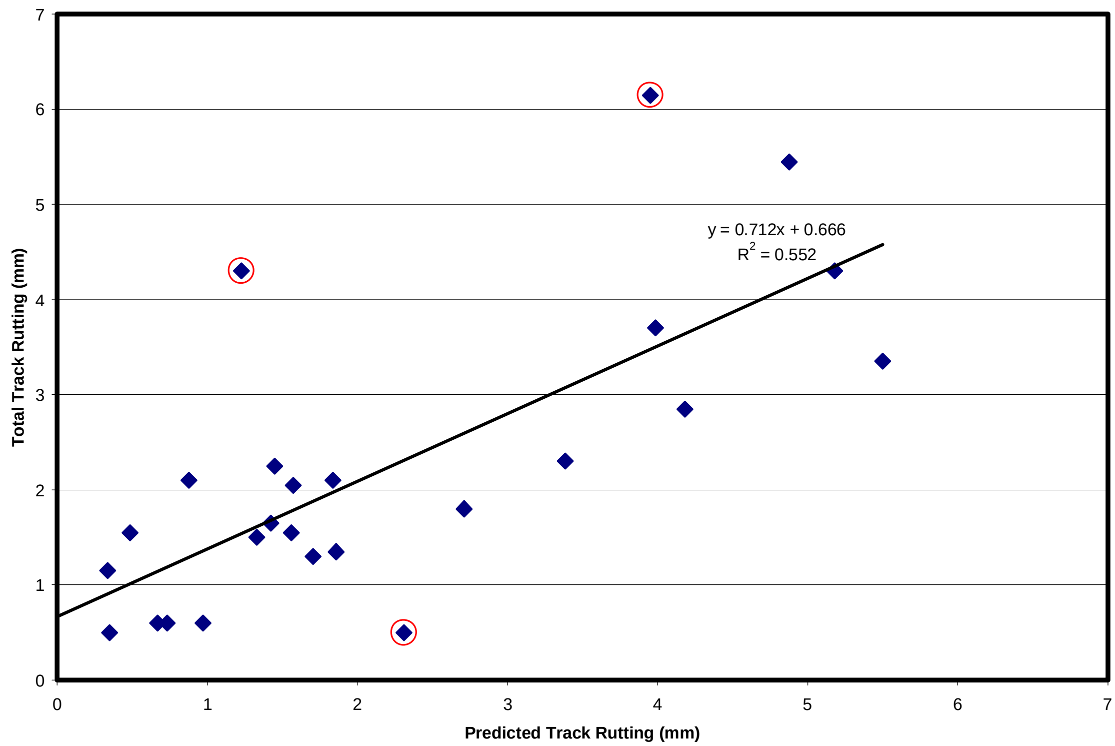

FIGURE 5.18 Final Model to Predict Track Rutting Performance of all 26 Tangent Test Sections Using Data Generated During Laboratory APA Testing.

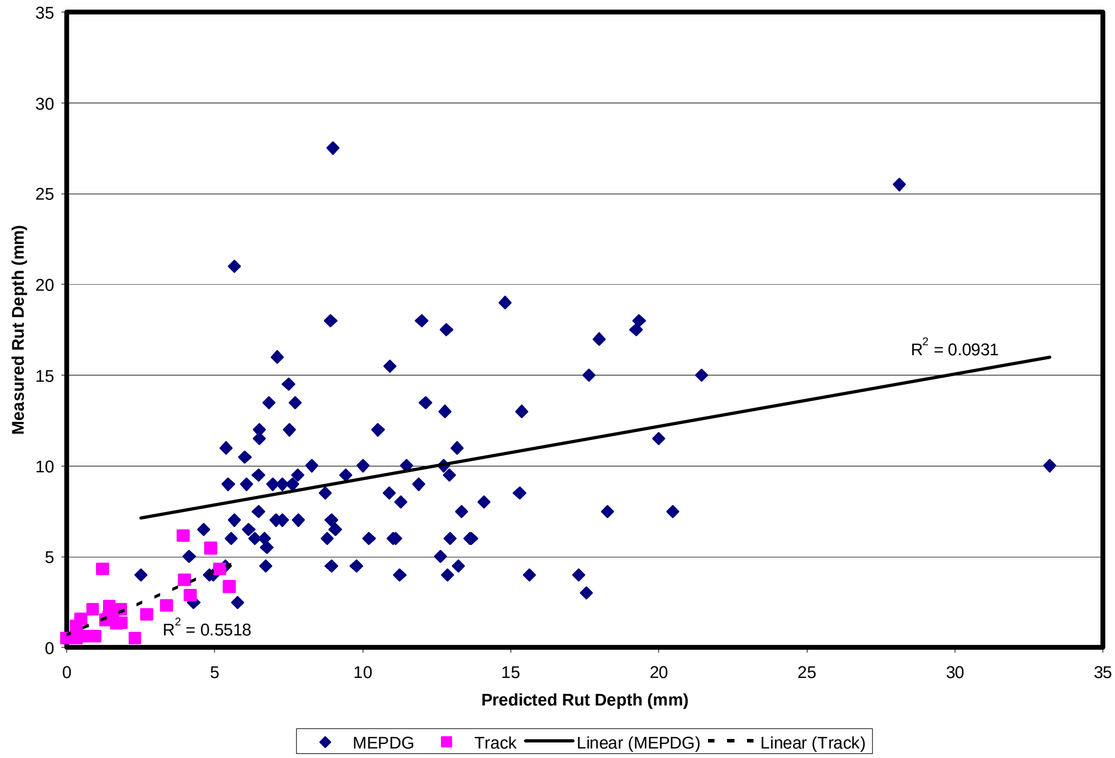

FIGURE 5.19 NCAT Model Predictions Compared to AASHTO's Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (ARA, 2004).

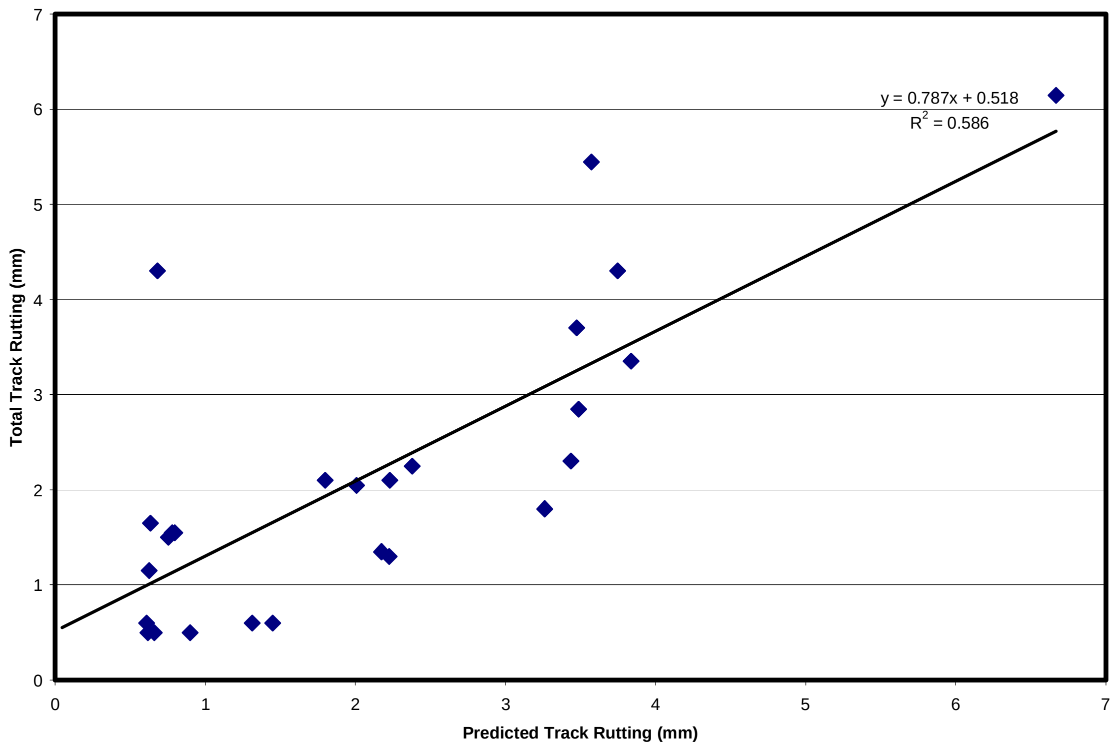

FIGURE 5.20 Compaction Model (via Equation 5.26) with Highest Coefficient of Determination (Sr_{max} in SGC).

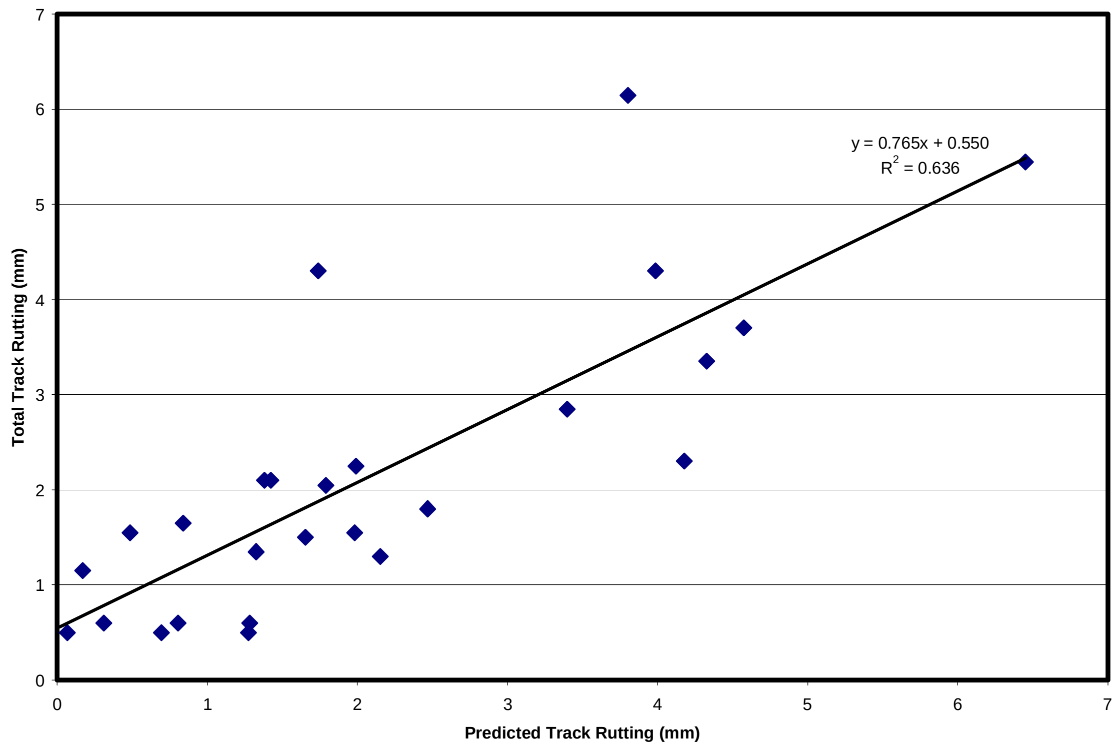

FIGURE 5.21 Simulative Model (via Equation 5.26) with Highest Coefficient of Determination (RLWT).

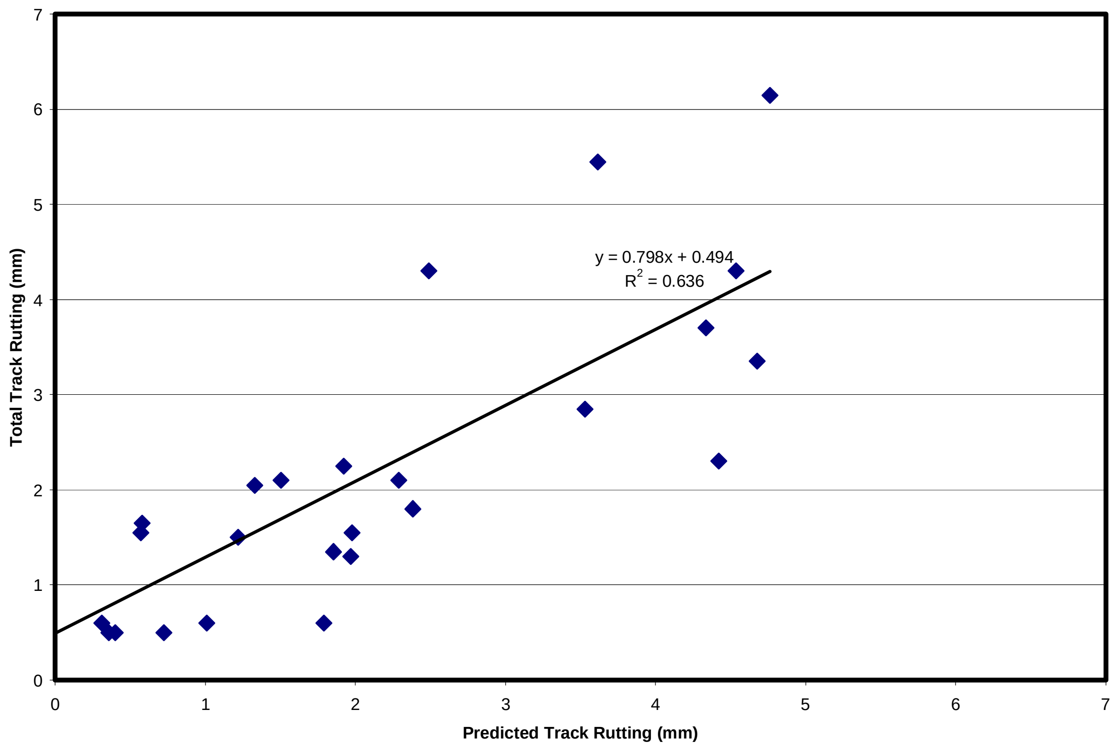

FIGURE 5.22 Fundamental Model (via Equation 5.26) with Highest Coefficient of Determination (Triaxial RLCC).

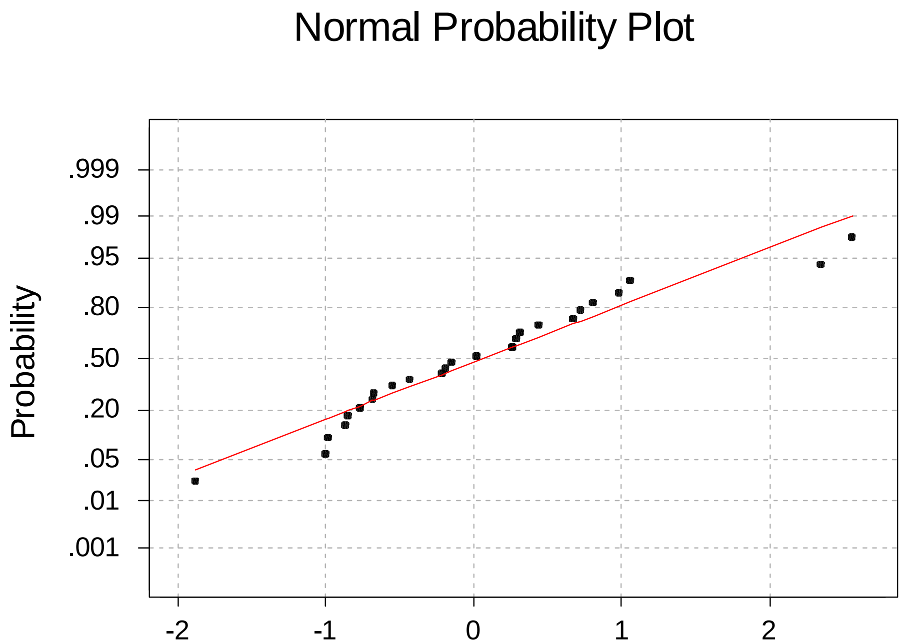

FIGURE 5.23 Residuals from Figure 5.21 Plotted with Cumulative Normal Distribution.

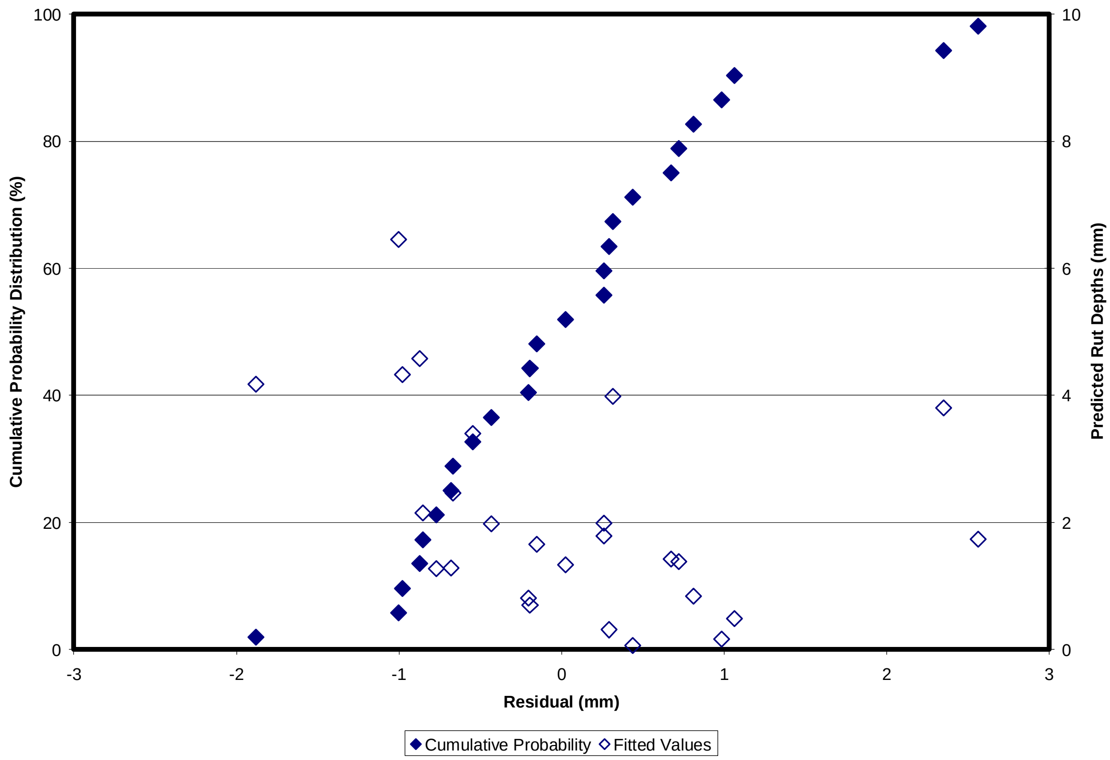

FIGURE 5.24 Residuals from Figure 5.21 Plotted with Model Predictions.

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

6.1 OBSERVATIONS

In consideration of field and laboratory data collected before, during and after traffic, the following observations are summarized as follows:

1. Little rutting was noted on the 2000 Track and all mixes (most of which were designed to support the intended traffic of 10 million ESALs) performed in a satisfactory manner. The section with the deepest rut only averaged 7.3 mm, and the section with the least rut only averaged 0.5 mm;
2. Virgin mixes designed both above ("fine-graded mixes") and below ("coarse-graded mixes") the maximum density line performed in a satisfactory manner. Although a very slight statistically significant performance advantage was generally observed with coarse-graded mixes, the differences were small enough to conclude equal performance in practical terms. The sole gradation study in which both the coarse-graded and fine-graded comparison mixes contained RAP resulted in better performance with the fine-graded mix (with statistical significance), although the use of RAP in these mixes did not necessarily have any effect on the outcome of the experiment;

3. Increasing the asphalt content by ½ percent to improve durability and compactability in mixes produced with 100 percent crushed stone and modified (PG76) binder did not negatively affect rutting performance. The same adjustment in mixes produced with 100 percent crushed stone and unmodified (PG67-22) binder had a statistically significant negative impact on rutting performance. In the latter case, the difference in rutting was great enough that increasing the asphalt content in mixes containing unmodified binder should be done with caution;
4. No statistical difference in rutting performance was noted between stone mixes produced with SBR-modified binder and SBS-modified binder of the same performance grade (PG76-22);
5. No statistical difference in rutting performance was noted between comparison sections that contained crushed granite, with one produced as an SMA and the other produced as a SUPERPAVE mix;
6. No statistical difference in rutting performance was noted between a section produced with high LA abrasion loss material and a section produced with conventional aggregates;
7. No statistical difference in rutting performance was noted between comparison mixes produced with 9.5 mm specifications and mixes produced with 12.5 mm specifications; and
8. No statistical difference in rutting performance was noted in dual studies in which gravel mixes were compared to mixes blended with limestone in

the hopes of improving quality. In one case, the gravel mix appeared to be so stiff that premature cracking might be an issue.

6.2 CONCLUSIONS

In consideration of the analyses presented in Chapters 4 and 5, the following conclusions can be drawn regarding the relationships between Track field performance and laboratory performance:

1. It is not possible to reliably predict field VTM from laboratory test data;
2. Transverse profiles best relate to wire line rut depth measurements when a 6-point method is utilized to account for the peaks on either side of the rutted wheelpath in addition to the valley in the middle of the wheelpath. Three-point methods that ignore the effect of the peaks on the outside edges of the wheelpaths do not relate well to wire line rut depths;
3. Locking point definitions have a big impact on final design gyrations. Significant differences exist between one-, two- and three-repeat scenarios;
4. Based on a limited dataset, lateral pressure in very early compaction can be a good indicator of rutting performance;
5. Although a conversion factor is required, laboratory rutting induced by other loaded wheel testers can be used to estimate rutting in the APA;

6. The relationship between induced rutting and traffic is optimized when traffic applied on a hot pavement surface is differentiated from traffic applied on a cool pavement surface. The temperature below which rutting no longer occurs is 34°C. Weight factors emphasizing the rutting effect of progressively hotter ESALs have been developed as a function of estimated compliance, with temperature banding suggested based on the PG binder system;
7. Secondary consolidation in the laboratory relates better to field rutting performance than total consolidation in the laboratory (as the sum of both primary and secondary consolidation);
8. Laboratory performance data can be used to predict field rutting performance after about one year of service. A regression equation was developed for this purpose that produced coefficients of determination that ranged from 0.55 (using only temperature weighted traffic and the difference between the laboratory test temperature and PG binder grade) to 0.79 (using temperature weighted traffic, primary and secondary consolidation in a LWT, and the difference between the laboratory test temperature and PG binder grade). A separate model was developed for every laboratory test method included in the study;
9. Lab-to-Field performance correlations after about a year of service can be age-adjusted in order to predict rutting at other times;

10. Models were developed (shown in Table 5.3) that predict rutting performance at the end of the two-year traffic cycle on the 2000 Track with coefficients of determination that range between 0.45 and 0.64. Models that produced the highest coefficients of determination for testing during gyratory compaction, simulative testing and fundamental testing are shown in Figure 5.20, 5.21 and 5.22, respectively;

11. As seen in Table 5.3, model performance predictions based on results from RLWT testing, triaxial (RLCC) testing, Sr_{max} measurements and rate of height change in the SGC, and rate of height change during GTM compaction with the air cell accounted for more than half of the variation in measured Track rutting values (i.e., coefficients of determination were greater than 0.5 for models based on results from these tests); and

12. On average the "Load-Temperature Spectra" method of traffic accumulation produced realistic estimates of performance. It is possible that higher coefficients of determination could be achieved by testing more samples in the laboratory or by optimizing the level of compaction in laboratory specimens.

6.3 RECOMMENDATIONS

In consideration of the aforementioned observations and conclusions, the following recommendations are made regarding future practice in hot-mix asphalt design and selection:

1. Bump modified asphalt contents by ½ percent (or reduce design gyration levels to produce the same effect) to enhance mix durability when N_{des} is greater than or equal to 100. Load-temperature spectra predictions should be used in conjunction with laboratory testing to evaluate potential asphalt content increases when N_{des} is less than 100, which is the case with many mixes now that N_{des} levels are being reduced;
2. Consider mixes designed on the fine side of the maximum density line in cases where a tighter mat is desirable, such as when permeability is an issue, and expect good performance. Load-temperature spectra predictions should be used in conjunction with laboratory testing to evaluate potential changes in aggregate blending;
3. SBR and SBS modifiers can be used interchangeably with the same aggregate blend with similar rutting performance; however, mix durability was not within the scope of the 2000 Track;
4. Consider blending stockpiles into 9.5 mm NMAS designs as a suitable alternative to blending them into 12.5 mm NMAS designs (using the same stockpiles at different proportions) with the expectation of no statistical difference in rutting performance. Blends will not be interchangeable in all cases, and laboratory performance testing should be used to verify expected comparability. Other performance requirements (such as surface friction, noise, etc.) or economy may override rutting comparability;

5. In order to better predict rutting performance, ESALs should be broken into temperature bands and weighted for hotter temperatures;
6. Rutting should be estimated with performance models that are provided for many different types of tests. The recommended method for rutting performance modeling is provided as Equation 5.26;
7. Rutting performance models should be validated using QC samples produced during the construction of the 2003 Track. Samples compacted during the reconstruction of the Track in 2003 should be subjected to various test protocols and utilized to generate rutting predictions that can be compared to the historical record;
8. Rutting performance models should be improved and extended for longer periods of time by utilizing performance data from sections that were left in place and subjected to another round of truck traffic on the 2003 Track. This will necessitate again combining the environmental, traffic and field performance records; and
9. Track section comparisons should be extended into future cycles of truck traffic in order to consider durability of experimental pavements.

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APPENDICES

APPENDIX A

CONSTRUCTION SPECIFICATION

ALDOT SPECIAL PROVISION NUMBER 3347 R

Test Section Bituminous Concrete Pavement for Projects Numbered 99-700-000-000-901 and 99-700-000-000-902 in Lee County, Alabama Dated June 16, 1999. Alabama Standard Specifications, 1992 Edition, are hereby amended by the addition of a NEW SECTION 412 as follows:

412.01 DESCRIPTION

The Contractor shall pave test sections of the National Center for Asphalt Technology (NCAT) test track in accordance with the requirements given in this Section and the requirements shown on the Plans. Several State Department of Transportation agencies will provide materials and paving mix designs to be used in the construction of test sections on the test track.

412.02 MATERIALS

The Contractor will be furnished with all materials (asphalt, aggregate, mineral filler, lime, fibers, etc.) for the paving of the test sections.

412.03 ASPHALT MIXES

412.03(a) General

The Contractor shall produce the mixes in accordance with the mix design requirements furnished with the asphalt mix materials.

412.03(b) Design Mixes

The job mix formula (JMF) will be provided to the Contractor. The Contractor shall control the gradation and asphalt content within the allowable tolerances. The initial setting of the controls for all materials shall be those amounts shown on the job-mix formula. The Contractor shall make changes as necessary in order that the mixture will run as close as practical to the percentage designated on the job-mix formula. [See Subarticle 412.03 (e)].

412.03(c) Trial Mixes

The contractor shall produce at least twenty tons of trial mix for each test section. The trial mix shall be placed by spreader and compacted to construct a vehicle parking area in the vicinity of the plant as directed by the Engineer. The wasting of trial mixes will be paid for as Wasting Trial Mixes.

The Contractor shall make adjustments to the production mixes for the test sections based on the results obtained from the testing of the trial mixes.

412.03(d) Layout of Test Sections

Each test section will be approximately four inches thick and shall be placed in two layers. Some test sections will be such that the bottom layer will be of a different job mix formula than the top layer. All test sections are to be approximately 200 feet in length, unless otherwise directed by the Engineer. The actual construction of each test section on the track surface shall be in a continuous manner with no breaks in laydown. Once construction begins on the test sections, the contractor shall continuously and diligently pursue the completion of the work.

412.03(e) Quality Assurance

412.03(e)1. Acceptance Procedures

All materials will be evaluated for acceptance through the National Center for Asphalt Technology (NCAT). NCAT will participate in determining the acceptability of the construction and materials incorporated therein. NCAT will advise the Engineer of the point in production (the production time or tonnage) for sampling at the plant (for mixture testing) and the locations for sampling and testing on the roadway (for mat density testing).

412.03(e)1.a. Section Comprised of the Same Mix for the Bottom and Top Layers

A LOT is defined as the production of the design job mix formula for an individual test section. Each lane of mix of each layer will be defined as a sublot, thus yielding a total of four sublots per LOT.

In-place density measurements will be taken at three sampling locations for each sublot. Each sublot of each LOT shall have an average compacted density of no less than ninety four percent of TMD (Theoretical Maximum Density). The mean absolute deviation of the density tests for each sublot shall not exceed 1.2 percent from 94 percent of TMD.

One sample of produced mix will be taken for each of the sublots and asphalt content and gradation determined. The mean absolute deviation of the asphalt content from the JMF for each LOT shall not be more than 0.3 percent based upon four samples per LOT (One random sample per each sublot).

The mean absolute deviation of the gradation for each of the coarse aggregate (aggregate retained on the Number 4 sieve and larger) from the JMF shall not be more than three percent based upon four samples per LOT (One random sample per each sublot).

The mean absolute deviation of the fine aggregate from the JMF shall not be more than two percent based upon four samples per LOT (One random sample per each sublot).

The mean absolute deviation of the filler (material passing the Number 200 sieve) from the JMF shall not be more than one percent based upon four samples per LOT (One random sample per each sublot).

412.03(e)1.b. Section Comprised of Different Mixes in the Bottom and Top Layers

A LOT is defined as the production of the design job mix formula for each layer of the test section. The LOT will be divided into two sublots, with each sublot being one lane.

In-place density measurements will be taken at three sampling locations for each sublot. Each sublot of each lot shall have an average compacted density of no less than ninety four percent of TMD. The mean absolute deviation of the density tests for each sublot shall not exceed 1.2 percent from 94 percent of TMD.

One sample of produced mix will be taken for each of the sublots and asphalt content and gradation determined. The mean absolute deviation of the asphalt content from the JMF for each LOT shall not be more than 0.3 percent based upon two samples per LOT (One random sample per each sublot).

The mean absolute deviation of the gradation for each of the coarse aggregate (aggregate retained on the Number 4 sieve and larger) from the JMF shall not be more than three percent based upon two samples per LOT (One random sample per each sublot).

The mean absolute deviation of the fine aggregate from the JMF shall not be more than two percent based upon two samples per LOT (One random sample per each sublot).

The mean absolute deviation of the filler (material passing the Number 200 sieve) from the JMF shall not be more than one percent based upon two samples per LOT (One random sample per each sublot).

412.03(e)2. Acceptance or Rejection

The decision of the Engineer will be final as to the acceptance or rejection of each sublot. Rejected sublots shall be removed at no cost to the Department and replaced at the contract unit bid price.

412.04 CONSTRUCTION REQUIREMENTS

412.04(a) General

In general, the choice of equipment will be left to the Contractor and it shall be his responsibility to provide proper sized and amounts of equipment that will produce, deliver to the roadbed, spread, and compact the plant mixed material in sufficient quantities for the continuous movement of the spreaders under normal operation conditions.

The mixing plant, hauling, spreading and compaction equipment shall meet the requirements listed below; however, other equipment that will produce equally satisfactory results, such as electronically or automatically controlled devices of proven performance, will be considered for use in lieu thereof.

The Contractor shall secure approval of all equipment prior to beginning work. Any equipment found unsatisfactory shall be promptly replaced or supplemented.

412.04(b) Sequence of Construction

The Contractor shall construct the test sections in accordance with the Sequence of Construction shown on the Plans. Low production rates should be expected due to testing, wasting trial mixes etc.

412.04(c) Mixing Plant

412.04(c)1. General

The Contractor shall provide a hot mix asphalt mixing plant on site for the production of the hot mix asphalt for this project. An area for the plant is shown on the Plans and has been cleared and leveled under a previous contract.

412.04(c)2. Plant Type

The mixing plant shall be either a drum mix or a batch type plant. Mixing plants shall comply with the requirements of AASHTO M 156 as modified by BMTP-324, Mixing Plant Requirements for Hot-Mixed, Hot-Laid Asphalt Paving Mixtures. The plant shall be capable of operating at a production rate as low as one hundred and fifty tons per hour and as high as required to successfully complete the work.

412.04(c)3. Scales

A digital recorder shall be installed as part of the platform truck scales. The recorder shall produce a printed digital record on a ticket of the gross and tare weights of the delivery trucks along with a time and date print for each ticket. Provisions shall be made so that scales may not be manually manipulated during the printing process, and so interlocked as to allow printing only when the scale has come to rest. The scales and recorder shall be of sufficient capacity and size to accurately weigh the heaviest loaded truck or tractor trailers that are used for the delivery of the hot mix asphalt from that plant.

In lieu of plant and truck scales, the Contractor may provide either (1) an approved automatic printer system which will print the weights of the material delivered (evidenced by a weight ticket for each load), provided the system is used in conjunction with an approved automatic batching and control system, or (2) an electronic load cell weigh system with associated computer hardware and automated printing system.

The Contractor may provide a "weigh batcher" system utilizing a weigh hopper equipped with load cells that determine the net amount of mix delivered from the weigh hopper. An automated weight printing system shall be provided to accurately print the weight of material delivered, the time and the date for each ticket.

All scales which weigh the mix for pay purposes shall meet the requirements of Subarticle 109.01(h).

412.04(c)4. Plant Configuration and Storage Requirements

The asphalt plant shall be capable of uniformly adding up to ten percent commercial mineral filler (in addition to the silo for hydrated lime) and up to 0.4 percent mineral or cellulose fiber. A silo capable of storing at least seventy five tons of hot mix asphalt shall be available.

The plant shall have the capability of metering and proportioning all or any part of the collected fines back into the mixture.

At least two asphalt binder storage tanks shall be provided. This may consist of one tank with two storage compartments.

The plant shall have at least five aggregate cold storage bins.

412.04(d) Hauling Equipment

Trucks used for hauling hot mix asphalt mixtures shall have tight, clean, smooth metal beds which have been thinly coated with a minimum amount of paraffin oil, lime solution or other approved material to prevent the mixture from adhering to the beds. The use of gasoline, kerosene or other volatile material is prohibited. Each truck shall be equipped with a cover of canvas or other suitable material of such size as to protect the mixture from adverse conditions. Each truck shall have a hole in the side of the body, approximately 5/16" in diameter and suitably placed, to allow for temperature measurement of the asphalt mix. When the air temperature is below 60 ºF, or hauling time exceeds 30 minutes, or threatening weather exists, no mixture shall leave the plant unless it is covered entirely and the cover securely fastened. Reference is made to Article 105.12 concerning load limitations on hauling equipment.

412.04(e) Mix Transfer Equipment

The asphalt mix shall be delivered to the spreader for the placement of the test sections by transfer equipment that is capable of remixing the material prior to the materials being placed in the spreader.

412.04(f) Spreaders

At least two hot mix asphalt spreaders will be required during the construction of the test sections. One spreader will be required for the placement of the test sections and one will be required for the placement of the wasted trial mixes.

Hot mix asphalt spreaders shall be self-contained and of sufficient size, power and stability to receive, distribute, and strike off the asphalt material at rates and widths consistent with the specified typical section requirements and details shown on the plans.

All hot mix asphalt spreaders used for mainline paving, including shoulders and interchange ramps, shall be operated with a full width vibratory, or other compactive type, screed. The augers used to move the material across the width of the screed shall extend within one foot of the edge of the screed. It will be permissible to use a hydraulically extendable strikeoff for paving turnouts and short sections of pavement including variable width sections, and crossovers.

When laying mixtures, the spreader shall be capable of being operated at forward speeds consistent with satisfactory laying of the mixture, providing a finished surface of the required evenness and texture without tearing, gouging or shoving of the mixture.

All hot mix asphalt spreaders shall be operated with automatic grade and slope controls unless otherwise directed by the Engineer. Equipment operating together shall have the same type controls. The automatic controls may operate either from control grade wires or ski; however, when a ski is used the spreader shall have a ski of not less than 30 feet in length. Both grade and slope controls shall be in good working order at all times. In the event of a malfunction of the automatic control system, the spreading operation shall be discontinued after one hour until the equipment is repaired and restored to first class working order.

412.04(g) Compaction

At least three types of rollers shall be available for compaction. These rollers shall be a vibratory steel wheel roller, a rubber tire roller and a static steel wheel roller.

The vibratory steel wheel roller shall be in good condition and shall weigh at least ten tons.

The rubber tire roller shall be in good condition and shall weigh at least fifteen tons. The tires shall be capable of being inflated to at least 90 psi. All tires shall be in good condition.

The static steel wheel roller shall be in good condition and shall weigh at least eight tons.

All test sections shall be compacted to a density of ninety four percent of TMD. The mean absolute deviation shall not exceed 1.2 percent from 94 percent of the TMD as described in Subarticle 412.03(e).

412.05 LAYER, SURFACE AND EDGE REQUIREMENTS

412.05(a) Preparation of Mixtures

412.05(a)1. Liquid Asphalt Binder

The liquid asphalt binder material shall be heated in a manner that insures the even heating of the entire mass under efficient and positive control at all times. Any liquid asphalt binder material which, in the opinion of the Engineer, has been damaged shall be rejected.

412.05(a)2. Aggregate

412.05(a)2.a. Aggregate Used for Batch Mixing Operations

All aggregates shall be dried so that the moisture content of the hot mix asphalt at the point of sampling is less than 0.2 percent by weight in accordance with BMTP-130. The temperature of the aggregate at the dryer shall not exceed 600 ºF.

When more than two ingredients enter into the composition of the mineral aggregate, they shall be combined as directed.

The aggregate, immediately after being heated, shall be screened into three or more sizes and conveyed into separate bins, ready for batching and mixing with liquid asphalt binder material. However, for mixes using aggregate of one-half inch maximum size, the number of bins may be reduced to two.

412.05(a)2.b. Aggregates for Dryer Drum Mixing Operations

Maintenance of a uniform aggregate gradation is essential for a dryer drum operation, hence, caution and care shall be exercised in stockpiling of materials to avoid segregation.

412.05(a)3. Mixing

412.05(a)3.a. General

The temperature range of mixing shall not exceed the temperature shown on the approved job-mix formula.

412.05(a)3.b. Batch Mixing

The dried mineral aggregate, and measured mineral filler when used, prepared as prescribed above, shall be combined in uniform batches by weighing and conveying into the mixer the proportionate amounts of each aggregate required to meet the job-mix formula. The largest size aggregate shall be introduced first, then smaller sizes progressively, with mineral filler last, or all mineral components may be added simultaneously. The mineral filler shall be added directly into the weigh hopper. The mineral components shall be thoroughly mixed. The required quantity of liquid asphalt binder material for each batch shall be measured by weight using scales or a liquid asphalt binder material metering device attached to the liquid asphalt binder material bucket.

After the mineral components have been mixed, the liquid asphalt binder material shall be added and the mixing continued for a period of at least 45 seconds, or longer if necessary to produce a homogeneous mixture. However, if a check by ASTM D 2489 (Ross Method) shows that 95 % plus coating is obtained, a shorter mixing time will suffice. The Engineer may then give written permission for a change. Each batch must be kept separate throughout the weighing and mixing operations.

The mixture shall be uniform in composition, free from lumps or balls of material containing an excess quantity of asphalt, or from pockets deficient in asphalt.

412.05(a)3.c. Dryer-Drum Mixing

Components shall be proportioned by weight as noted hereinbefore in Item 410.03(a)1 for this method of mixing. Amounts of aggregate and liquid asphalt binder material entering the mixer, and the rate of travel through the mixer, shall be so coordinated that a uniform mixture of specified gradation and liquid asphalt binder content will be produced. An anti-stripping agent may be required to insure adequate coating of the aggregates if so directed by the Engineer.

412.05(a)4. Mineral or Cellulose Fiber

Mineral or cellulose fiber shall be added to the mix in a manner that insures complete blending of the fiber with the aggregates and liquid asphalt binder.

412.05(a)4.a. Batch Plant

In a batch plant, the fiber shall be added into the weigh hopper simultaneously with the hot aggregates. Dry mixing time shall be increased at least 5 seconds to insure adequate blending. Wet mixing time will be increased at least 5 seconds for cellulose fibers and up to 5 seconds for mineral fibers.

412.05(a)4.b. Drum Plant

In a drum plant, a separate fiber feeding system shall be used to accurately and uniformly meter the fiber into the mix. If there is any evidence of fiber in the bag-house or wet-washer fines, the liquid asphalt binder line and/or the fiber line shall be relocated so that the fiber is captured by the liquid asphalt binder spray and incorporated into the mix. If there is any evidence of clumps of fibers or pellets at the discharge chute, the contractor shall increase the mixing time and/or intensity. This may entail extending the liquid asphalt binder and fiber feeding line further into the drum.

412.05(b) Placement of the Mixtures

412.05(b)1. Spreading

Spreading of the hot mix asphalt mixture shall be performed by equipment meeting the requirements of Subarticle 412.04(f), except as noted hereinafter in this Item. Approved specialized equipment may be employed to spread the hot mix asphalt material where standard full scale equipment is impractical due to size and irregularity of the area to be paved.

For hot mix asphalt pavement test track layers, spreading operations shall be so correlated with plant and hauling equipment that the spreading operation, once begun, shall proceed at a speed as uniform and continuous as practical. The continual forward movement of the spreader requires the use of hauling vehicles capable of supplying the spreader with hot mix asphalt material while the spreader is in motion. Repetitive interruptions or stopping of the spreader shall be cause for the Engineer to stop the work until the Contractor evaluates the cause of the stoppage and has provided a definite action plan for correction of the interruptions. Any interruption will require the thorough check of the area immediately under the spreader and any variances shall be corrected immediately or the material removed and replaced, as directed, without additional compensation.

Material placed in the spreader shall be immediately spread and screeded to such uniform depth that the average weight of the mixture required per square yard is secured. Alignment of the outside edges of the pavement shall be controlled by preset control lines, and shall be finished in conformity with these controls.

Any spreading operation which cannot produce acceptable joints within the surface tolerances and density requirements shall be cause for requiring the Contractor to modify his operations to include additional spreading equipment.

412.05(b)2. Compaction

As soon as the mixture has been spread and has set sufficiently to prevent undue cracking or shoving, rolling shall begin. A delay in the initial rolling will not be tolerated and the initial or breakdown rolling should in general be performed by rolling longitudinally, beginning at the sides and proceeding toward the center of the surface.

When paving abuts a previously placed lane, the longitudinal joint shall be rolled in the first pass. On superelevated curves rolling shall begin at the low side and progress toward the high side. The roller shall not compact within six inches of the edge of the surface where an adjacent lane is to follow, while the surface is still hot.

If any displacement occurs during rolling, it shall be corrected at once. To prevent adhesion of surface mixture to the rollers, the wheels shall be kept adequately moistened with water and a non-foaming detergent, but an excess of water will not be permitted.

Adequate precaution shall be taken to prevent dropping of gasoline or oil on the pavement. In places inaccessible to a roller, compaction shall be obtained with hand or mechanical tampers of adequate weight to produce required density.

412.05(b)3. Joints

412.05(b)3.a. General

Placing of hot mix asphalt paving layers shall be as continuous as possible. All joints shall be made in a careful manner in such a way as to provide a smooth, well bonded and sealed joint meeting the density and surface requirements given in this Section. Failure to meet requirements noted above shall be cause for ordering the removal and reconstruction of the joint without extra compensation.

The contact surface shall be treated with a thin coat of liquid asphalt binder material, tack material or the liquid asphalt binder material used in the mix, prior to construction of the joint. When directed by the Engineer, the same treatment noted above shall be used on cold asphalt joints.

412.05(b)3.b. Longitudinal

Longitudinal joints in the wearing surface shall conform with the edges of proposed traffic lanes insofar as practical. Any necessary longitudinal joints in underlying layers shall be offset so as to be at least 6 inches from the joint in the next overlying layer.

412.05(b)3.c. Transverse

Transverse joints shall be carefully constructed. Rollers shall not pass over the unprotected edge of the freshly laid mixture unless laying operations are to be discontinued. To facilitate the expeditious removal of the plant mix joint when laying operations are resumed, the Contractor shall place a heavy wrapping paper on the underlying surface across the joint and place plant mix on top of the paper.

Upon resumption of the work a neat joint shall be formed by sawing back vertically into the previously laid material to expose the full depth of the layer. The fresh mixture shall be raked and tamped to provide a well-bonded and sealed joint meeting surface and density requirements.

412.05(c) Layer Thickness

Each test section will be approximately four inches thick and shall be placed in two layers. Any test section that does not result in a placement rate within the required limits shall be removed by milling and shall be replaced at no cost to the Department.

412.05(d) Surface Smoothness

412.05(d)1. General

Surface smoothness and roadway section will be checked by the use of string, Engineer's level and straight edge.

The Contractor shall furnish string, straightedges and the necessary personnel to handle them under the supervision of the Engineer.

Surface smoothness tests shall be made continuously during and immediately after rolling so that irregularities may be eliminated to the extent possible by rolling while the material is still workable, otherwise deficiencies shall be corrected as provided in Article 410.06.

412.05(d)2. Requirements for all Surfaces

The finished surface of all base, binder, and wearing surface layers shall not vary more than 1/4 inch from the required section measured at right angles to the pavement centerline. The finished surface shall not vary more than 3/8 inch in any 25 foot section from a taut string applied parallel to the surface and roadbed centerline at the following locations: one foot inside of the edges of pavement, at the centerline, and at other points as designated. The variance from the designated grade shall not increase or decrease more than 1/2 inch in 100 feet.

The surface shall not vary more than 1/4 inch from a 16 foot straightedge placed parallel to the centerline at points directed. A 16 foot rolling straightedge, equipped with marking capability, may be used in lieu of the fixed straightedge if approved by the Engineer.

412.05(e) Edge Requirements

Surface, binder and leveling pavement edges not confined by curbing or other structures shall be lightly tamped, generally with a lute immediately behind the placement operation, to form an approximately 1:1 slope as a preventative measure against cracking and bulging during the rolling process. This procedure shall also be required on the initial edge of a longitudinal cold joint. These edges shall be neatly shaped to line behind the breakdown roller and shall be trimmed as necessary after final rolling, to an accurately lined string or wire providing a maximum tolerance of 2 inches outside the theoretical edge of pavement, with a maximum variation from a true line of 1/2 inch in 10 feet and a slope not flatter than 1:1. Edges that are distorted by rolling shall be corrected promptly.

412.05(f) Rideability Requirements

The rideability of the constructed sections shall meet the requirements of Subarticle 410.05(c)

412.06 DEFECTIVE OR DEFFICIENT AREAS

Areas of the test sections that are determined to be defective due to the operations of the Contractor shall be removed and replaced at no cost to the Department.

412.07 METHOD OF MEASUREMENT

The asphalt mix for the test track test sections and the trial mixes will be measured in tons.

412.08 BASIS OF PAYMENT

412.08(a) Unit Price Coverage

The asphalt plant mix for the test track test sections will be paid for at the contract unit price bid per ton of plant mix placed on the track. This shall be full compensation for all materials (except materials furnished to the Contractor), equipment, tools and labor required to construct the test track test sections. No payment will be made for excess mix produced.

The wasting of trial mixes will be paid for at the contract unit price bid per ton of plant mix wasted. This shall be full compensation for all materials (except materials furnished to the Contractor), equipment, tools and labor required for the production of trial mixes for testing and for subsequent placement on the plant parking area.

412.08(b) Payment Will Be Made Under Item Number:

1) 412–A Test Section Bituminous Concrete Pavement – per ton
2) 412–B Wasting Trial Mixes – per ton

APPENDIX B

AS-BUILT PROPERTIES OF TEST SECTIONS

Sectio N1

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Tuesday, April 25, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	75
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	55
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/Slag	Lift Type:	dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	100
3/8"	92
No. 4	69
No. 8	52
No. 16	33
No. 30	22
No. 50	15
No. 100	10
No. 200	6.7

Asphalt Binder Content:	7.4%
Compacted Pill Bulk Gravity:	2.306
Theoretical Maximum Gravity:	2.365
Computed Air Voids:	2.5%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		7.2%
Slag	78	32.0%
Slag	8910	28.0%
Limestone	Manufactured Sand	40.0%

Approximate Length (ft):	201
Surveyed Thickness of Section (in):	3.9
Std Dev of Section Thickness (in):	0.3
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	306
Average Section Compaction:	95.1%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N2

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Tuesday, April 25, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	75
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	55
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/Slag	Lift Type:	dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	99
3/8"	90
No. 4	66
No. 8	50
No. 16	33
No. 30	22
No. 50	16
No. 100	11
No. 200	7.6

Asphalt Binder Content:	7.8%
Compacted Pill Bulk Gravity:	2.300
Theoretical Maximum Gravity:	2.352
Computed Air Voids:	2.2%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		7.7%
Slag	78	32.0%
Slag	8910	28.0%
Limestone	Manufactured Sand	40.0%

Approximate Length (ft):	200
Surveyed Thickness of Section (in):	4.3
Std Dev of Section Thickness (in):	0.3
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	316
Average Section Compaction:	94.7%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N3

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Wednesday, April 26, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	85
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	55
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/Slag	Lift Type:	dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		7.5%
3/4"	100	Slag	78	32.0%
1/2"	99	Slag	8910	28.0%
3/8"	91	Limestone	Manufactured Sand	40.0%
No. 4	68			
No. 8	51			
No. 16	33			
No. 30	22			
No. 50	15	Approximate Length (ft):		200
No. 100	10	Surveyed Thickness of Section (in):		4.2
No. 200	6.5	Std Dev of Section Thickness (in):		0.1
Asphalt Binder Content:	7.6%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.294	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.369	Avg Mat Temperature Behind Paver (F):		295
Computed Air Voids:	3.2%	Average Section Compaction:		94.1%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N4

Laboratory Diary		Construction Diary		
General Description of Mix and Materials		**Relevant Conditions for Construction**		
Design Method:	Superpave	Completion Date:		Thursday, May 18, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):		85
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):		55
Modifier Type:	NA	24 Hour Rainfall (in):		0.00
Aggregate Type:	Lms/Slag	Lift Type:		dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):		4.0
Avg. Lab Properties of Plant Produced Mix		**Plant Configuration and Placement Details**		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		7.1%
3/4"	100	Slag	78	32.0%
1/2"	99	Slag	8910	28.0%
3/8"	91	Limestone	Manufactured Sand	40.0%
No. 4	68			
No. 8	52			
No. 16	35			
No. 30	23			
No. 50	15	Approximate Length (ft):		199
No. 100	9	Surveyed Thickness of Section (in):		4.2
No. 200	6.0	Std Dev of Section Thickness (in):		0.1
Asphalt Binder Content:	6.8%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.296	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.400	Avg Mat Temperature Behind Paver (F):		288
Computed Air Voids:	4.3%	Average Section Compaction:		93.4%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N5

Laboratory Diary		Construction Diary		
General Description of Mix and Materials		**Relevant Conditions for Construction**		
Design Method:	Superpave	Completion Date:	Thursday, May 18, 2000	
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	89	
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	68	
Modifier Type:	NA	24 Hour Rainfall (in):	0.00	
Aggregate Type:	Lms/Slag	Lift Type:	dual	
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	4.0	
Avg. Lab Properties of Plant Produced Mix		**Plant Configuration and Placement Details**		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		6.9%
3/4"	100	Slag	78	53.0%
1/2"	99	Slag	8910	17.0%
3/8"	84	Limestone	Modified 8910	30.0%
No. 4	52			
No. 8	38			
No. 16	26			
No. 30	18			
No. 50	14	Approximate Length (ft):	201	
No. 100	11	Surveyed Thickness of Section (in):	4.4	
No. 200	8.3	Std Dev of Section Thickness (in):	0.3	
Asphalt Binder Content:	6.9%	Type of Tack Coat Utilized:	CQS-1h	
Compacted Pill Bulk Gravity:	2.285	Target Tack Application Rate:	0.03 gal / sy	
Theoretical Maximum Gravity:	2.355	Avg Mat Temperature Behind Paver (F):	282	
Computed Air Voids:	3.0%	Average Section Compaction:	93.8%	

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N6

Laboratory Diary

General Description of Mix and Materials

Design Method:	Superpave
Compactive Effort:	100 gyrations
Binder Performance Grade:	67-22
Modifier Type:	NA
Aggregate Type:	Lms/Slag
Gradation Type:	BRZ

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	99
3/8"	85
No. 4	54
No. 8	37
No. 16	25
No. 30	17
No. 50	13
No. 100	10
No. 200	8.2

Asphalt Binder Content:	6.8%
Compacted Pill Bulk Gravity:	2.270
Theoretical Maximum Gravity:	2.348
Computed Air Voids:	3.3%

Construction Diary

Relevant Conditions for Construction

Completion Date:	Thursday, June 01, 2000
24 Hour High Temperature (F):	92
24 Hour Low Temperature (F):	67
24 Hour Rainfall (in):	0.00
Lift Type:	dual
Design Thickness of Test Mix (in):	4.0

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.5%
Slag	78	53.0%
Slag	8910	17.0%
Limestone	Modified 8910	30.0%

Approximate Length (ft):	197
Surveyed Thickness of Section (in):	4.1
Std Dev of Section Thickness (in):	0.2
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	289
Average Section Compaction:	94.4%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N7

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Thursday, June 01, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	92
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	67
Modifier Type:	SBR	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/Slag	Lift Type:	dual
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	98
3/8"	83
No. 4	52
No. 8	36
No. 16	24
No. 30	17
No. 50	13
No. 100	10
No. 200	7.8

Asphalt Binder Content:	6.9%
Compacted Pill Bulk Gravity:	2.281
Theoretical Maximum Gravity:	2.330
Computed Air Voids:	2.1%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.9%
Slag	78	53.0%
Slag	8910	17.0%
Limestone	Modified 8910	30.0%

Approximate Length (ft):	203
Surveyed Thickness of Section (in):	3.9
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	323
Average Section Compaction:	93.9%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N8

Laboratory Diary		Construction Diary		
General Description of Mix and Materials		**Relevant Conditions for Construction**		
Design Method:	Superpave	Completion Date:	Monday, June 05, 2000	
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	77	
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	60	
Modifier Type:	SBR	24 Hour Rainfall (in):	0.08	
Aggregate Type:	Lms/Slag	Lift Type:	dual	
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	4.0	
Avg. Lab Properties of Plant Produced Mix		**Plant Configuration and Placement Details**		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		6.4%
3/4"	100	Slag	78	53.0%
1/2"	99	Slag	8910	17.0%
3/8"	85	Limestone	Modified 8910	30.0%
No. 4	55			
No. 8	37			
No. 16	24			
No. 30	17			
No. 50	13	Approximate Length (ft):	203	
No. 100	10	Surveyed Thickness of Section (in):	3.9	
No. 200	7.5	Std Dev of Section Thickness (in):	0.2	
Asphalt Binder Content:	6.6%	Type of Tack Coat Utilized:	CQS-1h	
Compacted Pill Bulk Gravity:	2.256	Target Tack Application Rate:	0.03 gal / sy	
Theoretical Maximum Gravity:	2.351	Avg Mat Temperature Behind Paver (F):	319	
Computed Air Voids:	4.0%	Average Section Compaction:	94.7%	

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N9

Laboratory Diary		Construction Diary		
General Description of Mix and Materials		**Relevant Conditions for Construction**		
Design Method:	Superpave	Completion Date:	Wednesday, June 07, 2000	
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	84	
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	60	
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00	
Aggregate Type:	Lms/Slag	Lift Type:	dual	
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	4.0	
Avg. Lab Properties of Plant Produced Mix		**Plant Configuration and Placement Details**		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		6.4%
3/4"	100	Slag	78	53.0%
1/2"	99	Slag	8910	17.0%
3/8"	87	Limestone	Modified 8910	30.0%
No. 4	57			
No. 8	40			
No. 16	26			
No. 30	19			
No. 50	14	Approximate Length (ft):	197	
No. 100	11	Surveyed Thickness of Section (in):	3.9	
No. 200	8.8	Std Dev of Section Thickness (in):	0.2	
Asphalt Binder Content:	6.7%	Type of Tack Coat Utilized:	CQS-1h	
Compacted Pill Bulk Gravity:	2.279	Target Tack Application Rate:	0.03 gal / sy	
Theoretical Maximum Gravity:	2.354	Avg Mat Temperature Behind Paver (F):	314	
Computed Air Voids:	3.2%	Average Section Compaction:	94.5%	

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N10

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Wednesday, June 07, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	84
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	60
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/Slag	Lift Type:	dual
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		6.9%
3/4"	100	Slag	78	53.0%
1/2"	98	Slag	8910	17.0%
3/8"	84	Limestone	Modified 8910	30.0%
No. 4	51			
No. 8	34			
No. 16	23			
No. 30	17			
No. 50	13	Approximate Length (ft):		206
No. 100	10	Surveyed Thickness of Section (in):		4.2
No. 200	7.7	Std Dev of Section Thickness (in):		0.3
Asphalt Binder Content:	6.8%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.257	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.339	Avg Mat Temperature Behind Paver (F):		324
Computed Air Voids:	3.5%	Average Section Compaction:		94.7%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N11 (Lower Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Tuesday, June 06, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	85
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	68
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	lower
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	2.5

Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		4.3%
3/4"	100	Granite	6	27.0%
1/2"	81	Granite	7	17.0%
3/8"	70	Granite	89	20.0%
No. 4	46	Granite	M10	24.0%
No. 8	34	Granite	W10	11.0%
No. 16	27	Antistrip	Hydrated Lime	1.0%
No. 30	21			
No. 50	15	Approximate Length (ft):		195
No. 100	10	Surveyed Thickness of Section (in):		NA
No. 200	6.3	Std Dev of Section Thickness (in):		NA
Asphalt Binder Content: 4.1%		Type of Tack Coat Utilized:		PG 67-22
Compacted Pill Bulk Gravity: 2.448		Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity: 2.529		Avg Mat Temperature Behind Paver (F):		296
Computed Air Voids: 3.2%		Average Section Compaction:		92.7%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N11 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Monday, June 12, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	83
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	54
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	upper
Gradation Type:	TRZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	97
3/8"	80
No. 4	52
No. 8	37
No. 16	30
No. 30	24
No. 50	18
No. 100	11
No. 200	7.2

Asphalt Binder Content:	4.3%
Compacted Pill Bulk Gravity:	2.434
Theoretical Maximum Gravity:	2.519
Computed Air Voids:	3.4%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.5%
Granite	7	38.0%
Granite	89	18.0%
Granite	M10	32.0%
Granite	W10	11.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	195
Surveyed Thickness of Section (in):	4.1
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	PG 67-22
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	327
Average Section Compaction:	93.1%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N12 (Lower Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Tuesday, June 06, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	85
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	68
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	lower
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	2.5

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	99
1/2"	83
3/8"	72
No. 4	49
No. 8	36
No. 16	28
No. 30	22
No. 50	16
No. 100	10
No. 200	6.5

Asphalt Binder Content:	4.2%
Compacted Pill Bulk Gravity:	2.445
Theoretical Maximum Gravity:	2.535
Computed Air Voids:	3.5%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.3%
Granite	6	27.0%
Granite	7	17.0%
Granite	89	20.0%
Granite	M10	24.0%
Granite	W10	11.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	201
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	PG 67-22
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	293
Average Section Compaction:	92.4%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N12 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	SMA	Completion Date:	Monday, June 12, 2000
Compactive Effort:	50 blows	24 Hour High Temperature (F):	93
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	67
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	upper
Gradation Type:	SMA	Design Thickness of Test Mix (in):	1.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	96
3/8"	73
No. 4	32
No. 8	23
No. 16	21
No. 30	19
No. 50	17
No. 100	14
No. 200	11.8

Asphalt Binder Content:	6.2%
Compacted Pill Bulk Gravity:	2.335
Theoretical Maximum Gravity:	2.401
Computed Air Voids:	2.7%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.1%
Granite	7	60.0%
Granite	89	22.0%
Granite	M10	10.0%
Stabilizer	Fiber	0.4%
Filler	Fly Ash	7.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	201
Surveyed Thickness of Section (in):	1.9
Std Dev of Section Thickness (in):	0.2
Type of Tack Coat Utilized:	PG 67-22
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	343
Average Section Compaction:	94.6%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N13 (Lower Layer)

Laboratory Diary

General Description of Mix and Materials

Design Method:	Superpave
Compactive Effort:	100 gyrations
Binder Performance Grade:	76-22
Modifier Type:	SBS
Aggregate Type:	Gravel
Gradation Type:	BRZ

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	88
3/8"	73
No. 4	51
No. 8	33
No. 16	25
No. 30	20
No. 50	13
No. 100	8
No. 200	6.3

Asphalt Binder Content:	5.0%
Compacted Pill Bulk Gravity:	2.310
Theoretical Maximum Gravity:	2.384
Computed Air Voids:	3.1%

Construction Diary

Relevant Conditions for Construction

Completion Date:	Wednesday, June 07, 2000
24 Hour High Temperature (F):	84
24 Hour Low Temperature (F):	60
24 Hour Rainfall (in):	0.00
Lift Type:	lower
Design Thickness of Test Mix (in):	2.5

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.9%
Gravel	3/4" Crushed Gr	38.0%
Gravel	3/8" Crushed Gravel	43.0%
Limestone	Modified 8910	8.0%
Gravel	Coarse Sand	10.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	199
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	318
Average Section Compaction:	92.5%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio N13 (Upper Layer)

Laboratory Diary

General Description of Mix and Materials

Design Method:	SMA
Compactive Effort:	50 blows
Binder Performance Grade:	76-22
Modifier Type:	SBS
Aggregate Type:	Gravel
Gradation Type:	SMA

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	99
3/8"	74
No. 4	30
No. 8	25
No. 16	23
No. 30	21
No. 50	17
No. 100	13
No. 200	11.5

Asphalt Binder Content:	6.8%
Compacted Pill Bulk Gravity:	2.175
Theoretical Maximum Gravity:	2.266
Computed Air Voids:	4.0%

Construction Diary

Relevant Conditions for Construction

Completion Date:	Monday, June 12, 2000
24 Hour High Temperature (F):	93
24 Hour Low Temperature (F):	67
24 Hour Rainfall (in):	0.00
Lift Type:	upper
Design Thickness of Test Mix (in):	1.0

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.7%
Gravel	1/2" Crushed Gra	72.5%
Gravel	3/8" Crushed Gravel	10.0%
Gravel	Coarse Sand	9.5%
Stabilizer	Fiber	0.5%
Filler	Fly Ash	8.0%

Approximate Length (ft):	199
Surveyed Thickness of Section (in):	1.0
Std Dev of Section Thickness (in):	0.2
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	325
Average Section Compaction:	92.0%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S1 (Lower Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		Relevant Conditions for Construction	
Design Method:	Superpave	Completion Date:	Thursday, June 22, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	92
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	75
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	lower
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	2.5
Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details	
Sieve Size:	% Passing:	Component:	% Setting:
1"	100	Liquid Binder Setting	4.7%
3/4"	97	Granite 6M	52.0%
1/2"	66	Granite 789	20.0%
3/8"	48	Granite Manufactured Sand	10.0%
No. 4	32	Granite Regular Screenings	18.0%
No. 8	24		
No. 16	20		
No. 30	16		
No. 50	11	Approximate Length (ft):	200
No. 100	7	Surveyed Thickness of Section (in):	NA
No. 200	4.1	Std Dev of Section Thickness (in):	NA
Asphalt Binder Content:	5.0%	Type of Tack Coat Utilized:	CQS-1h
Compacted Pill Bulk Gravity:	2.408	Target Tack Application Rate:	0.03 gal / sy
Theoretical Maximum Gravity:	2.484	Avg Mat Temperature Behind Paver (F):	320
Computed Air Voids:	3.1%	Average Section Compaction:	93.7%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S1 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Monday, June 26, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	92
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	75
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	upper
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	1.0

Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		5.2%
3/4"	100	Granite	6M	10.0%
1/2"	95	Granite	789	53.0%
3/8"	86	Granite	Manufactured Sand	12.0%
No. 4	54	Granite	Regular Screenings	25.0%
No. 8	36			
No. 16	28			
No. 30	21			
No. 50	15	Approximate Length (ft):		200
No. 100	9	Surveyed Thickness of Section (in):		1.9
No. 200	5.5	Std Dev of Section Thickness (in):		0.0
Asphalt Binder Content:	5.0%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.378	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.452	Avg Mat Temperature Behind Paver (F):		335
Computed Air Voids:	3.0%	Average Section Compaction:		94.8%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S2 (Lower Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Thursday, June 22, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	92
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	75
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Gravel	Lift Type:	lower
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	2.5

Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		4.9%
3/4"	100	Gravel	3/4" Crushed Gr	38.0%
1/2"	86	Gravel	3/8" Crushed Gravel	43.0%
3/8"	69	Limestone	Modified 8910	8.0%
No. 4	46	Gravel	Coarse Sand	10.0%
No. 8	30	Antistrip	Hydrated Lime	1.0%
No. 16	23			
No. 30	19			
No. 50	11	Approximate Length (ft):		200
No. 100	7	Surveyed Thickness of Section (in):		NA
No. 200	5.5	Std Dev of Section Thickness (in):		NA
Asphalt Binder Content:	4.9%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.282	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.388	Avg Mat Temperature Behind Paver (F):		320
Computed Air Voids:	4.4%	Average Section Compaction:		93.0%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S2 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Tuesday, June 27, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	92
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	75
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Gravel	Lift Type:	upper
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	1.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	100
3/8"	96
No. 4	67
No. 8	41
No. 16	29
No. 30	22
No. 50	15
No. 100	10
No. 200	8.4

Asphalt Binder Content:	6.0%
Compacted Pill Bulk Gravity:	2.233
Theoretical Maximum Gravity:	2.342
Computed Air Voids:	4.7%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.3%
Gravel	1/2" Crushed Gr	15.0%
Gravel	3/8" Crushed Gravel	72.0%
AggLime	Agricultural Lime	5.0%
Gravel	Coarse Sand	7.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	200
Surveyed Thickness of Section (in):	1.9
Std Dev of Section Thickness (in):	0.0
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	326
Average Section Compaction:	93.8%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S3 (Lower Layer)

Laboratory Diary

General Description of Mix and Materials

Design Method:	Superpave
Compactive Effort:	100 gyrations
Binder Performance Grade:	76-22
Modifier Type:	SBS
Aggregate Type:	Limestone
Gradation Type:	BRZ

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	97
1/2"	86
3/8"	80
No. 4	47
No. 8	27
No. 16	20
No. 30	16
No. 50	12
No. 100	9
No. 200	7.3

Asphalt Binder Content:	4.2%
Compacted Pill Bulk Gravity:	2.461
Theoretical Maximum Gravity:	2.559
Computed Air Voids:	3.8%

Construction Diary

Relevant Conditions for Construction

Completion Date:	Monday, June 26, 2000
24 Hour High Temperature (F):	92
24 Hour Low Temperature (F):	75
24 Hour Rainfall (in):	0.00
Lift Type:	lower
Design Thickness of Test Mix (in):	2.5

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.4%
Limestone	67	26.0%
Limestone	Modified 8910	28.0%
Limestone	89	40.0%
Gravel	Coarse Sand	5.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	201
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	332
Average Section Compaction:	92.8%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S3 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Thursday, July 06, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	98
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	78
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/gravel	Lift Type:	upper
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	1.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	100
3/8"	100
No. 4	70
No. 8	43
No. 16	29
No. 30	21
No. 50	15
No. 100	11
No. 200	8.9

Asphalt Binder Content:	5.6%
Compacted Pill Bulk Gravity:	2.329
Theoretical Maximum Gravity:	2.414
Computed Air Voids:	3.5%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		5.9%
Gravel	3/8" Crushed Gr	47.0%
Limestone	Modified 8910	34.0%
Limestone	89	14.0%
Gravel	Coarse Sand	4.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	201
Surveyed Thickness of Section (in):	1.0
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	316
Average Section Compaction:	92.7%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S4 (Lower Layer)

Laboratory Diary

General Description of Mix and Materials

Design Method:	Superpave
Compactive Effort:	100 gyrations
Binder Performance Grade:	76-22
Modifier Type:	SBS
Aggregate Type:	Lms/RAP
Gradation Type:	ARZ

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	99
1/2"	88
3/8"	69
No. 4	48
No. 8	38
No. 16	30
No. 30	24
No. 50	15
No. 100	9
No. 200	6.5

Asphalt Binder Content:	4.1%
Compacted Pill Bulk Gravity:	2.466
Theoretical Maximum Gravity:	2.567
Computed Air Voids:	3.9%

Construction Diary

Relevant Conditions for Construction

Completion Date:	Wednesday, July 05, 2000
24 Hour High Temperature (F):	96
24 Hour Low Temperature (F):	71
24 Hour Rainfall (in):	0.00
Lift Type:	lower
Design Thickness of Test Mix (in):	2.5

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.3%
Limestone	6	30.0%
Limestone	78	25.0%
Sand	Manufactured	20.0%
Sand	Natural	15.0%
Recycle	RAP	10.0%

Approximate Length (ft):	198
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	322
Average Section Compaction:	93.6%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S4 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Thursday, July 06, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	98
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	78
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Limestone	Lift Type:	upper
Gradation Type:	ARZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix		Plant Configuration and Placement Details		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		5.3%
3/4"	100	Limestone	10's (Hard)	15.0%
1/2"	98	Limestone	10's (Soft)	10.0%
3/8"	88	Limestone	5/8 D Rock	45.0%
No. 4	63	Sand	Manufactured	15.0%
No. 8	46	Sand	Natural	15.0%
No. 16	33			
No. 30	23			
No. 50	13	Approximate Length (ft):		198
No. 100	9	Surveyed Thickness of Section (in):		4.0
No. 200	7.8	Std Dev of Section Thickness (in):		0.1
Asphalt Binder Content:	5.3%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.394	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.449	Avg Mat Temperature Behind Paver (F):		322
Computed Air Voids:	2.2%	Average Section Compaction:		94.3%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S5 (Lower Layer)

Laboratory Diary

General Description of Mix and Materials

Design Method:	Superpave
Compactive Effort:	100 gyrations
Binder Performance Grade:	76-22
Modifier Type:	SBS
Aggregate Type:	Lms/Gravel/RAP
Gradation Type:	BRZ

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	95
1/2"	83
3/8"	73
No. 4	53
No. 8	36
No. 16	27
No. 30	21
No. 50	15
No. 100	12
No. 200	8.7

Asphalt Binder Content:	4.0%
Compacted Pill Bulk Gravity:	2.369
Theoretical Maximum Gravity:	2.446
Computed Air Voids:	3.1%

Construction Diary

Relevant Conditions for Construction

Completion Date:	Wednesday, July 05, 2000
24 Hour High Temperature (F):	96
24 Hour Low Temperature (F):	76
24 Hour Rainfall (in):	0.00
Lift Type:	lower
Design Thickness of Test Mix (in):	2.5

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.6%
Gravel	10	14.0%
Gravel	57	33.0%
Gravel	7	26.0%
Sand	Manufactured	15.0%
Recycle	RAP	12.0%

Approximate Length (ft):	203
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	331
Average Section Compaction:	91.5%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S5 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Friday, July 07, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	100
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	79
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Gravel	Lift Type:	upper
Gradation Type:	TRZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	95
3/8"	82
No. 4	61
No. 8	45
No. 16	33
No. 30	22
No. 50	10
No. 100	7
No. 200	5.0

Asphalt Binder Content:	5.6%
Compacted Pill Bulk Gravity:	2.332
Theoretical Maximum Gravity:	2.413
Computed Air Voids:	3.4%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		5.5%
Gravel	10	21.0%
Gravel	5/8 D Rock	60.0%
Sand	Natural	19.0%

Approximate Length (ft):	203
Surveyed Thickness of Section (in):	4.1
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	323
Average Section Compaction:	94.9%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S6

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Friday, July 07, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	100
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	79
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/RAP	Lift Type:	dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	95
3/8"	87
No. 4	74
No. 8	53
No. 16	41
No. 30	33
No. 50	24
No. 100	12
No. 200	5.9

Asphalt Binder Content:	6.2%
Compacted Pill Bulk Gravity:	2.250
Theoretical Maximum Gravity:	2.356
Computed Air Voids:	4.5%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.6%
Limestone	C22 Screenings	45.0%
Limestone	78	20.0%
Limestone	9	20.0%
Recycle	RAP	15.0%

Approximate Length (ft):	198
Surveyed Thickness of Section (in):	4.1
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	332
Average Section Compaction:	92.9%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S7

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Saturday, July 08, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	100
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	79
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Lms/RAP	Lift Type:	dual
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	96
3/8"	88
No. 4	71
No. 8	34
No. 16	25
No. 30	20
No. 50	16
No. 100	10
No. 200	6.2

Asphalt Binder Content:	6.6%
Compacted Pill Bulk Gravity:	2.245
Theoretical Maximum Gravity:	2.321
Computed Air Voids:	3.3%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		6.9%
Limestone	C20 Screenings	16.0%
Limestone	78	20.0%
Limestone	9	51.0%
Recycle	RAP	13.0%

Approximate Length (ft):	202
Surveyed Thickness of Section (in):	4.0
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	313
Average Section Compaction:	93.2%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S8 (Lower Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Friday, July 07, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	100
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	79
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	lower
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	2.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	87
3/8"	70
No. 4	39
No. 8	26
No. 16	18
No. 30	14
No. 50	12
No. 100	10
No. 200	7.1

Asphalt Binder Content:	3.7%
Compacted Pill Bulk Gravity:	2.598
Theoretical Maximum Gravity:	2.667
Computed Air Voids:	2.6%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		3.9%
Granite	67	32.0%
Granite	78M	41.0%
Granite	Manufactured Sand	21.0%
Granite	Regular Screenings	5.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	197
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	331
Average Section Compaction:	93.8%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S8 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Saturday, July 08, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	103
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	79
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	upper
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	1.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	100
3/8"	93
No. 4	58
No. 8	38
No. 16	25
No. 30	19
No. 50	15
No. 100	12
No. 200	7.8

Asphalt Binder Content:	4.2%
Compacted Pill Bulk Gravity:	2.576
Theoretical Maximum Gravity:	2.647
Computed Air Voids:	2.7%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.2%
Granite	78M	49.0%
Granite	Manufactured Sand	45.0%
Granite	Regular Screenings	5.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	197
Surveyed Thickness of Section (in):	1.3
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	322
Average Section Compaction:	91.8%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S9

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Monday, July 10, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	101
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	75
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	dual
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	3.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	93
3/8"	82
No. 4	53
No. 8	36
No. 16	27
No. 30	20
No. 50	14
No. 100	9
No. 200	5.7

Asphalt Binder Content:	4.7%
Compacted Pill Bulk Gravity:	2.419
Theoretical Maximum Gravity:	2.510
Computed Air Voids:	3.6%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.8%
Granite	67	15.0%
Granite	78M	47.0%
Granite	Dry Screenings	20.0%
Granite	Washed Screenings	18.0%

Approximate Length (ft):	206
Surveyed Thickness of Section (in):	3.0
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	331
Average Section Compaction:	93.4%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S10

Laboratory Diary		Construction Diary		
General Description of Mix and Materials		**Relevant Conditions for Construction**		
Design Method:	Superpave	Completion Date:		Tuesday, July 11, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):		98
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):		78
Modifier Type:	NA	24 Hour Rainfall (in):		0.00
Aggregate Type:	Granite	Lift Type:		dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):		3.0
Avg. Lab Properties of Plant Produced Mix		**Plant Configuration and Placement Details**		
Sieve Size:	% Passing:	Component:		% Setting:
1"	100	Liquid Binder Setting		5.3%
3/4"	100	Granite	67	11.0%
1/2"	95	Granite	78M	25.0%
3/8"	88	Granite	Dry Screenings	32.0%
No. 4	69	Granite	Washed Screenings	32.0%
No. 8	52			
No. 16	38			
No. 30	27			
No. 50	19	Approximate Length (ft):		195
No. 100	11	Surveyed Thickness of Section (in):		3.1
No. 200	6.6	Std Dev of Section Thickness (in):		0.1
Asphalt Binder Content:	5.2%	Type of Tack Coat Utilized:		CQS-1h
Compacted Pill Bulk Gravity:	2.407	Target Tack Application Rate:		0.03 gal / sy
Theoretical Maximum Gravity:	2.488	Avg Mat Temperature Behind Paver (F):		300
Computed Air Voids:	3.2%	Average Section Compaction:		93.7%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S11 (Lower Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Monday, July 10, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	101
Binder Performance Grade:	67-22	24 Hour Low Temperature (F):	75
Modifier Type:	NA	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	lower
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	2.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	86
3/8"	70
No. 4	38
No. 8	26
No. 16	18
No. 30	14
No. 50	12
No. 100	10
No. 200	7.2

Asphalt Binder Content:	3.6%
Compacted Pill Bulk Gravity:	2.600
Theoretical Maximum Gravity:	2.662
Computed Air Voids:	2.3%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		3.9%
Granite	67	37.0%
Granite	78M	41.0%
Granite	Manufactured Sand	16.0%
Granite	Regular Screenings	5.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	202
Surveyed Thickness of Section (in):	NA
Std Dev of Section Thickness (in):	NA
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	324
Average Section Compaction:	94.6%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S11 (Upper Layer)

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Wednesday, July 12, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	101
Binder Performance Grade:	76-22	24 Hour Low Temperature (F):	79
Modifier Type:	SBS	24 Hour Rainfall (in):	0.00
Aggregate Type:	Granite	Lift Type:	upper
Gradation Type:	BRZ	Design Thickness of Test Mix (in):	3.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	100
3/8"	92
No. 4	62
No. 8	47
No. 16	30
No. 30	22
No. 50	17
No. 100	13
No. 200	7.5

Asphalt Binder Content:	3.9%
Compacted Pill Bulk Gravity:	2.567
Theoretical Maximum Gravity:	2.649
Computed Air Voids:	3.1%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.2%
Granite	78M	49.0%
Granite	Manufacture Sand	45.0%
Granite	Regular Screenings	5.0%
Antistrip	Hydrated Lime	1.0%

Approximate Length (ft):	202
Surveyed Thickness of Section (in):	3.0
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	334
Average Section Compaction:	93.2%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S12

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Hveem	Completion Date:	Thursday, July 13, 2000
Compactive Effort:	NA	24 Hour High Temperature (F):	100
Binder Performance Grade:	70-28	24 Hour Low Temperature (F):	79
Modifier Type:	SB	24 Hour Rainfall (in):	0.67
Aggregate Type:	Limestone	Lift Type:	dual
Gradation Type:	TRZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	97
3/8"	82
No. 4	63
No. 8	46
No. 16	32
No. 30	23
No. 50	16
No. 100	10
No. 200	7.0

Asphalt Binder Content:	4.5%
Compacted Pill Bulk Gravity:	2.399
Theoretical Maximum Gravity:	2.494
Computed Air Voids:	3.8%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		4.7%
Limestone	5/8" Chips	35.0%
Limestone	Coarse Screenings	32.0%
Sand	Natural	8.0%
Sand	Stone Sand	25.0%

Approximate Length (ft):	199
Surveyed Thickness of Section (in):	3.8
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	318
Average Section Compaction:	93.9%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.

Sectio S13

Laboratory Diary		Construction Diary	
General Description of Mix and Materials		**Relevant Conditions for Construction**	
Design Method:	Superpave	Completion Date:	Thursday, July 13, 2000
Compactive Effort:	100 gyrations	24 Hour High Temperature (F):	100
Binder Performance Grade:	70-28	24 Hour Low Temperature (F):	79
Modifier Type:	SB	24 Hour Rainfall (in):	0.67
Aggregate Type:	Granite	Lift Type:	dual
Gradation Type:	ARZ	Design Thickness of Test Mix (in):	4.0

Avg. Lab Properties of Plant Produced Mix

Sieve Size:	% Passing:
1"	100
3/4"	100
1/2"	93
3/8"	80
No. 4	68
No. 8	50
No. 16	37
No. 30	27
No. 50	19
No. 100	11
No. 200	6.6

Asphalt Binder Content:	5.3%
Compacted Pill Bulk Gravity:	2.290
Theoretical Maximum Gravity:	2.405
Computed Air Voids:	4.8%

Plant Configuration and Placement Details

Component:		% Setting:
Liquid Binder Setting		5.3%
Granite	1/2" Chips	16.0%
Granite	3/4" Chips	22.0%
Granite	Regular Screenings	22.0%
Sand	Manufactured	40.0%

Approximate Length (ft):	201
Surveyed Thickness of Section (in):	4.0
Std Dev of Section Thickness (in):	0.1
Type of Tack Coat Utilized:	CQS-1h
Target Tack Application Rate:	0.03 gal / sy
Avg Mat Temperature Behind Paver (F):	322
Average Section Compaction:	93.4%

General Notes:

1) Mixes are listed chronologically in order of completion date (i.e., construction began with E
2) Sections are referenced by quadrant and sequence number, where "E2" refers to section 2 of th
3) "dual " lift type indicates that the lower and upper lifts were constructed with the same ex
4) The total thickness of all experimental sections is 4 inches by design, with the exception o
5) ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricte
6) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively.