EVALUATION OF RUTTING POTENTIAL OF HOT MIX ASPHALT USING THE ASPHALT PAVEMENT ANALYZER

Final Report

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16. ABSTARCT

A comprehensive study involving rut potential of Hot Mix Asphalt (HMA) was conducted. Both cylindrical and beam specimens of HMA were prepared using a Superpave Gyratory Compactor (SGC) and an Asphalt Vibratory Compactor (AVC), respectively. Mixture rutting performance was determined in the Asphalt Pavement Analyzer (APA). Initially, rut tests were conducted on three laboratory-prepared HMA for 8000 cycles of loading with 100 psi hose pressure, 100 lb wheel load, and 50 seating cycles. The rut values (8,000 cycles) varied between 2.0 mm and 6.4 mm. Rut depths were found to be sensitive to temperature when compared that to asphalt content.

Subsequently, this study evaluated rut potential of ten plant-produced mixes. Three of these mixes were of type A and six type B insoluble and one Type C. Only one mix showed a rut depth of more than 4 mm. The AVC beam specimens showed higher rut depth compared to cylindrical specimens. The APA rut test data were analyzed to identify the important contributing factors. Type A mixes were sensitive to percent asphalt content, where as Type B insoluble mixes were sensitive to material passing number 200 sieve.

This research investigated the relationship between rheological and mechanical properties for various Oklahoma unmodified and modified binders based on the asphalt mixture's rutting performance. The tests result showed that binder's Performance Grade (PG) affects mixture performance significantly. In general, modified binder showed better performance compared to the unmodified binders. Modified binders of same PG grade did not show the same performance when test parameters were held constant. Binder's viscosity and rut factor ($G^*/\sin\delta$) did not show significant effects on rutting performance of both modified and unmodified binders. Linear and nonlinear regression analyses were performed to investigate the contribution of binder properties to rutting. The nonlinear regression prediction of rutting was better than the linear prediction.

This study identified the most significant factors from a number of factors, which affect rut potential of HMA. Seven factors: binders PG, specimen type, test temperature, moisture, wheel load, asphalt content, and hose pressure, each at two defined levels were incorporated in a Superpave mix. Rut tests were designed to be the elements of an experimental matrix. The matrix test results were analyzed statistically. The analysis results showed that binders PG, specimen type, test temperature, and moisture, affected a mixture's rutting performance significantly. This study developed and described a statistical procedure to design and analyze an experimental matrix of test results.

This research investigated the repeatability and reproducibility of laboratory test data. An inter-laboratory study was performed on rut tests using the APA between the 'asphalt design laboratory' at the Oklahoma Department of Transportation (ODOT) and the 'asphalt laboratory' at the University of Oklahoma (OU). The tests result showed no significant variability in the collected data from two laboratories. This study developed a rut database for future model development. The APA rut results of HMA materials, which were used in a road section (funded by ODOT) of the National Center for Asphalt Technology (NCAT) Test Track at Alabama, were also included in the rut database.

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DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the information presented herein. The contents do not necessarily reflect the official views of the Oklahoma Department of Transportation (ODOT). This reports does not constitute a standard, specification, or regulation.

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EXECUTIVE SUMMARY

Rutting of flexible pavement is a widespread problem both nationally and in Oklahoma. Rutting is defined as the longitudinal depression along the wheel path due to progressive movement of materials under repeated traffic load. Recent studies have shown that rutting potential of Hot Mix Asphalt (HMA) samples can be evaluated in the laboratory during the design phase of a project using an Asphalt Pavement Analyzer (APA). The rutting susceptibility is evaluated by subjecting HMA samples to moving wheel loads and measuring permanent deformation at selected points along the wheel path as a function of the number of loading cycle. A pressurized rubber hose is placed between the moving wheel and the HMA sample to approximately simulate traffic loading on a pavement in the field. Both rectangular beam and cylindrical samples can be used. A typical test usually involves 8,000 cycles of loading on three beam samples or six cylindrical samples or a combination. The Asphalt Vibratory Compactor (AVC) is used to prepare beam samples, while cylindrical samples are either prepared using a Superpave Gyratory Compactor (SGC) or an AVC. Temperature, magnitude and frequency of moving load, hose pressure and number of cycles can be varied between tests and within the same test, if so desired. Effect of moisture can also be considered by conducting the test on samples submerged under water.

The University of Oklahoma (OU) received funding for a project (Item 2153) to procure an Asphalt Pavement Analyzer and an Asphalt Vibratory Compactor for the Ray Broce Materials Laboratory at OU. This project, funded jointly by the Oklahoma Department of Transportation (ODOT), the Federal Highway Administration (FHWA) and the Oklahoma Asphalt Pavement Association (OAPA), has two major goals: exploratory testing of selected mixes to gain confidence and experience in using APA for evaluation of rut potential, and establishing "baseline data" for selected mixes having low and high rut susceptibility. The following tasks were identified to accomplish the project goals: procurement and installation of APA and AVC, demonstration and training, selection of mixes and collection of materials (ingredients),

preparation of sample, exploratory rut testing, analysis of exploratory test data, conducting tests for baseline data, analysis of baseline data, and preparation of final report.

The APA and the AVC were purchased in August 1999. A new electrical panel was installed in the Broce lab to meet the power requirements. Also, the laboratory compressed air supply was upgraded to provide compressed air to both pieces of equipment. The installation was completed in September 1999. The manufacturer, Pavement Technologies, Inc. of Georgia, conducted a weeklong demonstration and training session in October 1999 that involved calibration of data acquisition system (DAS) for wheel load, horizontal and vertical displacements, DAS setting for beam and cylindrical samples, operation of temperature and preset counter controllers, rubber hose replacement, rut depth measurement (both manual and automated), sample preparation using AVC, safety training, and complete rut and fatigue testing. Three mixes, one for exploratory testing and two for baseline data, were selected in cooperation with ODOT. In addition, ten plant-produced mixes were selected for testing by both the ODOT Materials Division and the OU Team for comparison of results and to address the issue of reliability. Later, another limestone Superpave mix was added for extensive testing in developing baseline data.

The mix design for exploratory testing for one of the mixes (3012-OAPA-99037) was selected from ODOT standards and specifications for type B-insoluble mix. A total of 64 samples were tested for rutting. About half of these samples were compacted with the AVC, while SGC (SGC) was used to compact the remaining samples. Two different temperatures (60° and 64°C) and four different asphalt contents (4.5%, 5%, 5.5% and 6%) were used for this series of tests. In the initial stage, over 50% of the samples did not meet the target air void criteria of 6 to 8% of these samples compacted with the AVC. Sample quality and air void compliance improved over time and as the research team gained experience. Rut tests were conducted for 8000 cycles of loading with 100 psi hose pressure, 100 lb wheel load, and 50 seating cycles. The rut values (8,000 cycles) varied between 2.0 mm and 6.4 mm. Rut depths were found to be sensitive to temperature when compared that to asphalt content. Although, one of the goals of exploratory testing was to address "reproducibility" of data, this goal could not be achieved partly because of the difficulties in achieving the target air void at the initial stage. Also, it became evident that rut potential evaluation using the APA is not a trivial exercise because of the complexities and difficulties involved in preparing "identical" samples for testing, particularly rut measurement (location, averaging, level of accuracy, sensitivity, etc.). This task was completed in June 2000. Based on discussions at the Project Panel Meeting, the project was extended in August 2000 for a year to address the following items that were not addressed in the work plan of the original proposal (Item 2153): comparison of data for the ten plant produced mixes with the ODOT data for the same mixes and packaging of the data, a better control on achieving the air void requirement, reproducibility of test data, correlation between rutting and resilient modulus, and density gradient analysis. An extension for one year has been arranged to address the last two items. Addressing these items is considered important in enriching our knowledge and confidence in APA as a tool for performance-based testing of HMA. Efforts during the past year focused on the first three items, and equipment has been procured to pursue the remaining two items.

Evaluation of rut potentials for ten plant-produced mixes was completed in September 2000. These mixes were selected in cooperation with ODOT Materials Division. Seven of these mixes were type B-insoluble, and three recycled asphalt pavement (RAP). For each mix, six cylindrical (SGC) and two beam samples were prepared and tested, giving a total of 80 samples. A majority of these samples met the target air void ($7 \pm 1\%$). The measured rut depth values varied between 1 mm and 8 mm. The rut depths from beam samples were consistently higher than the corresponding cylindrical samples. Such variations are attributed to sample geometry and rut measurement details. ODOT Materials Division has conducted rut tests using the APA on the same ten plant produced mixes. This data was collected from ODOT, and compared with the corresponding data obtained by the OU Team. There was not a significant difference in measured rut depths for the same mix; therefore, additional rut tests were not conducted. An outlier was used to sort poor data, if there was any. Ranking of these mixes according to their rut potential was completed in December 2000.

ODOT participated in the NCAT Test Track project and provided materials and mix designs for two test sections. In a meeting, the Oklahoma Department of Transportation and Oklahoma Asphalt Pavement Association suggested that the OU Broce Lab participate in rut testing of both mixes. We tested 12 samples (6 SGC cylindrical) x 2 mixes) for rutting. The rut depth from the track will be compared with the APA data when the field data becomes available.

Two gravel mixes (3011-OK99-63070 and 3011-OK99-63071) were selected, in cooperation with ODOT, for the development of "baseline data." For each of the two mixes, we tested 24 samples for rutting (1 gradation x 1–PG binder x 1–aging x 1-temperature x 4 asphalt contents x 6 samples, 4-SGC cylindrical samples and 2-AVC beam samples. At that stage, it was possible to compact HMA specimens to target air voids fairly accurately. Several samples were tested under water and with different loading conditions as well as hose pressure. The baseline data can be used for calibration of APA. As such, the baseline data is reproducible. Since it is very difficult to produce APA samples that are identical, addressing the issue of reproducibility is a difficult task. A duplicate series of APA tests using 24 samples was conducted to address reproducibility.

Later, it was realized that the baseline data was lacking Superpave mixes, so a limestone mix, which was designed in accordance with the Superpave method, was added to the test matrix. The limestone mix was designed using 13 different asphalt binders (unmodified and modified) that are currently used in Oklahoma. A total of 104 cylindrical SGC samples were prepared and tested for rutting, and the results statistically analyzed to enrich the baseline database. Twelve Superpave samples were prepared in the OU laboratory. Half of these samples were tested for rutting at the OU Broce laboratory, while the remaining half was tested at ODOT. Similarly, another 12 samples were prepared at ODOT using the same aggregate and binders used at OU. The rut test values thus obtained was compared to address the issue of repeatability and reproducibility.

CHAPTER 1

INTRODUCTION

1.1 General

Hot Mix Asphalt (HMA) combines bituminous binder and aggregate to give a pavement structure that is flexible over a wide range of climatic conditions. The fact that HMA can be produced from a wide variety of local aggregates and yet perform on a consistent basis makes it the pavement of choice throughout the United States and the rest of the world. Approximately 93% of all roadway surfaces in the United States are paved with HMA. The vehicular miles traveled in America have increased approximately 75% in the past 20 years. The changing demographics in American society have also lead to many rural roads becoming high traffic roads or asphalt roads as the population moves from urban to rural.

Many asphalt roads consist of layer after layer of nonstructural surface mix. These layers have been generated by making temporary repairs, or placing thin overlays to improve the rideability of roads with little attention given to structural strength, which is needed to support the traffic loads. In the last decade, loads on the nation's highways have increased more than 60% (Brock et al., 1999). In addition to the increased loads, the increased distress due to radial tires and high tire pressures make it easy to see why asphalt roads develop ruts.

Rutting is a pavement distress, which has been seen nationwide. Excessive rutting has been reported in Florida, Georgia, Illinois, Pennsylvania, Tennessee, and Virginia (Barkdale, 1993). Rutting is a prevailing concern in Oklahoma today. Roberts et al. (1996) defined rutting as the formation of twin longitudinal depressions under the wheel paths caused by the progressive movement of materials under repeated loads in the asphalt pavement layers or in underlying base through consolidation or plastic flow. A typical rutting profile is shown in Figure 1.1.

These depressions or ruts are of concern for at least two reasons: if the surface is impervious, rut traps water which causes hydroplaning; which is a potential threat to passenger car safety, and as the rut deepens; steering becomes increasingly difficult; which leading to added danger.

Rutting can significantly reduce both structural and functional performance of an existing pavement. Sometimes, the rutting magnitude may not be alarming with regards to its structural performance, but it is important from the safety point of view (Roberts et al. 1996). Accordingly, it would be prudent to categorize existing rutting and the capability to predict or quantify future rutting potential. Rutting can provide useful information in selecting rehabilitation methods if it is categorized (Gramling et al., 1991). In case of consolidation and shear manifest rutting, a heavier overlay can be used to improve serviceability. In case of rutting due to lateral distortion, rehabilitation strategies can involve milling or leveling with a new wearing course, or recycling of the surface course (Gramling et al. 1991).

Depending on the magnitude of the traffic load and the relative strength of the pavement layers, rutting can occur in the subgrade, base, or upper asphalt (HMA) layers.

Studies conducted by the National Center for Asphalt Technology (NCAT) have indicated that rutting generally occurs in the top 75 to 100 mm (3 to 4 inch) of HMA pavement (Kandhal, et al., 1993; Brown et al., 1992). HMA is a composite material composed of a carefully graded aggregates embedded in a matrix of asphalt cement that fills part of the space between the aggregate particles and binds them together. The properties of the individual components and how they react with each other in the system affect its performance behavior. There are occasions when the asphalt binder and aggregate blends are adequate, but the mix fails to exhibit the desired performance because of poor compaction, incorrect binder content, poor adhesion or some other problems associated with the mixture. Also, mix properties alone are not sufficient to ensure satisfactory performance. Rutting results primarily from high-pressure truck tires and increased wheel loads. The stress pattern induced in a three-dimensional pavement structure due to traffic loading is complex. When the response depends on the time or rate of loading and temperature, the characterization becomes even more difficult.

Rutting prediction of a given circumstances requires detailed knowledge of the elastic, viscous and plastic deformation characteristic that influence constituents of a pavement. However, it is possible to control rutting by selecting quality aggregates with proper gradation and a asphalt binder; with the appropriate performance, and proportioned accordingly, so that adequate voids in the mix to resist permanent deformation.

Traditionally, predicting field performance of HMA has been complicated. A safeguard is needed to protect against making substantial investments in asphalt pavement only to discover, after opening to traffic, that the pavement will not meet performance expectations. Several types of laboratory equipment have been developed to measure rutting potential including the French Rut Tester, the Georgia Loaded Wheel Tester (LWT) and the Asphalt Pavement Analyzer (APA) (Collins, 1995). A detailed discussion about the strengths and weaknesses of some of these types of equipment are given in Chapter II: Literature Review for this report. Recent studies have shown that rutting potential of HMA samples can be evaluated in the laboratory during the design phase of a project using an APA. The APA test results can be used to rank mixture performance in the laboratory before costly surprises are encountered in the field. This equipment evaluates rutting susceptibility by subjecting HMA samples to a moving wheel loads and measuring permanent deformation at selected points along the wheel path as a function of the number of loading cycles. This study employed the APA to perform a series of laboratory tests.

1.2 Hypotheses

Hypothesis 1

Rutting is a mix related problem. It results from accumulated deformation in the asphalt layers rather than in the underlying subgrade. It occurs each time a heavy truck applies a load on an asphalt pavement layer with inadequate shear strength. A higher pavement temperature normally increases the rate of rutting. The recently developed APA can closely simulate and control the field conditions (truck load, tire pressure, temperature, wet and dry conditions) in a laboratory. It is hypothesized that mixture's rutting potential can be evaluated based on the APA test results.

Hypothesis 2

Rutting is influenced by numerous parameters. It is difficult to separate the effect of an individual parameter on rutting due to their interaction and combined effect. However, the APA can be employed to investigate the influence of some of the main parameters on rutting potential of HMA. It is hypothesized that a statistical model can be developed to investigate rut-influencing parameters.

Hypothesis 3

Currently, there exists no model to incorporate many of the rut influencing parameters. A neuron-based model can be developed to predict rutting by incorporating the parameters that affect rutting. However, training and calibration is needed on a number of data sets in the development of a neural network model. It is hypothesized that rutting database (baseline data) can be developed based on the APA test results.

1.3 Objectives

The primary goals of this study are: to evaluate and analyze the rutting susceptibility of asphalt mixes based on the APA data, and to evaluate and analyze pertinent mix properties that lead to differential rutting potentials of HMA specimens. To accomplish these goals, the following objectives were defined below:

- review of pertinent rutting literature,
- conduct a series of the APA rutting tests as exploratory and rank the mixtures based on their rutting performance,

- perform simple and multiple regression analyses to identify the significant rut influencing parameters develop a statistical method that uses the relationships between two or more quantitative variables to generate a model, which can predict rutting from others,
- develop relationships of asphalt, aggregate and mixture properties with rutting of HMA,
- produce rut data to develop a rut database, perform a series of tests to develop baseline data, use baseline data for the calibration of the APA and for verification of a developed model, and
- conduct tests at OU and ODOT on materials under similar testing conditions, compare OU data with ODOT data to examine the variability issue of using the APA.

1.4 Report Outline

This report is composed of eight chapters. Chapter I provides a brief statement of rutting problems, including specific goals and objectives of the study. Chapter II provides a comprehensive review of literature focusing on the experimental and modeling aspects of rutting, particularly on evaluation of rutting potential using the asphalt pavement analyzer, and mechanisms of rutting. The APA test data of exploratory and base mixes of gravel type is discussed in chapter III. The APA data of plant-produced mixes along with a discussion of the results and rutting susceptibility of the mixes are discussed in Chapter IV. Also, the APA data is compared with the ODOT data, in Chapter IV. Chapter V discusses the binder's effect on the mixture performance of rutting. Chapter VI discusses the statistical evaluation of rut parameters. Chapter VII discusses the repeatability and reproducibility of the APA rut testing. Finally, the contribution of this research and a recommendation for potential future studies are presented in Chapter VIII.

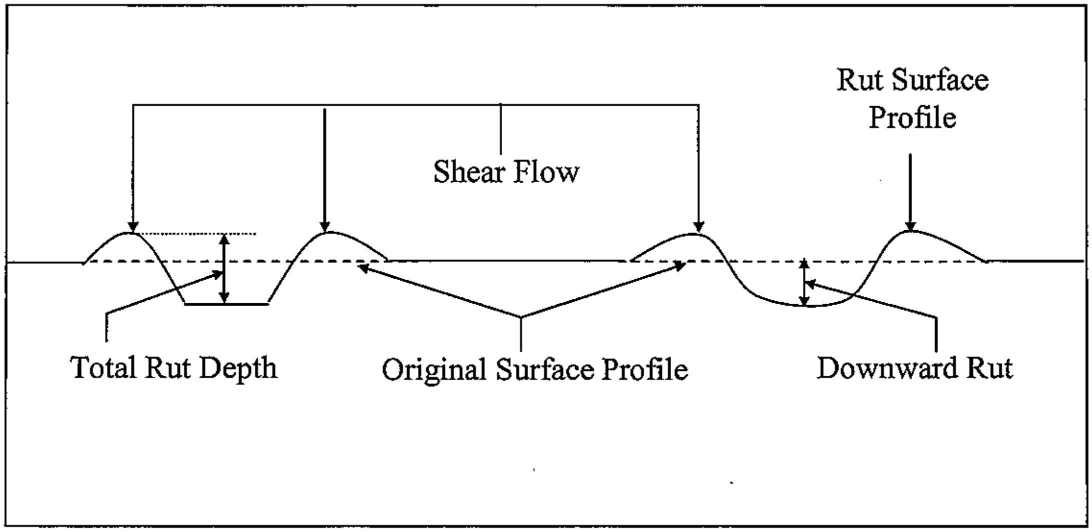


Figure 1.1 Typical Rutting Profile

CHAPTER 2

LITERATURE REVIEW

2.1 Laboratory Rut Testing

During the past three decades, a wide variety of equipment and procedures have been developed and used to assess rutting characteristics of HMA mixes in the laboratory. These include: the traditional Marshall and Hveem tests, uniaxial and triaxial static and dynamic creep tests, and the Superpave direct shear test. Among these, the Marshall and Hveem methods are widely used in the United States to establish optimum asphalt contents of HMA mixes based on the concept of stability (resistance to deformation). This stability, however, is neither based on fundamental engineering properties nor has been validated in the field to predict rutting in HMA pavements. The Marshall and Hveem test methods also do not indicate the potential for fatigue cracking in HMA pavements (Lai, 1996). Researchers have used various types of creep tests for laboratory evaluation of HMA permanent deformation (Collins et al., 1995). Neither AASHTO nor ASTM has adopted any creep test nor has validated any creep test in the field. Recently, an asphalt aggregate mix analysis system (AAMAS) was developed to evaluate HMA for permanent deformation and fatigue cracking. However, the AAMAS has also not been validated in the field.

The Strategic Highway Research Program (SHRP), which was conducted between October 1987 and March 1993, developed the Superior Performing Asphalt Pavements (Superpave) mix designs and analysis system. The adoption of Superpave methods by governmental agencies in the wake of the Strategic Highway Research Program (SHRP) has attracted worldwide attention, as pavement professionals seek to advance mix design methodologies to keep pace with across the board increases in traffic volumes and axle loads. Internationally, many developing countries will likely follow the American lead as they seek to implement more cost-effective methods to build and maintain necessary transportation infrastructures at lower life cycle costs. While the hot-mix asphalt (HMA) industry has invested resources in improving designs, and traditional test methods intended to quantify performance in mixes with dense aggregate structures (e.g. Marshall stability testing) will no longer applicable for new mixes with stone-on-stone gradations. Thus, materials engineers have struggled with exactly how to evaluate performance in the practical manner to which they have become accustomed.

As Superpave implementation nears, the industry has been naturally drawn towards relatively new types of empirical tests to fill the consequential performance evaluation void. A standardized laboratory equipment and test procedure that predicts field-rutting potential would be of great benefit to the HMA industry. As mix design evolved from conventional Marshall design to the superpave design and beyond, it becomes increasingly important to identify practical laboratory test methods to predict the performance of HMA pavements. Performance testing has been deemed necessary for a broad acceptance of the Superpave mix design system. Researchers have sought for a simple and yet reliable testing procedure to assess rutting potential of HMA for more than a decade.

Currently, the most common type of laboratory equipment of this nature is a loaded wheel tester (LWT). Several LWTs currently are being used in the United States. They include the Georgia Loaded Wheel Tester (GLWT), Asphalt Pavement Analyzer (APA), Hamburg Wheel Tracking Device (HWTD), LCPC (French) Wheel Tracker, Purdue University Laboratory Wheel Tracking Device (PURWheel), and one-third scale Model Mobile Load Simulator (MMLS3) (Colley et. al., 2001).

2.2 APA Rut Testing

The most recent and significant change in equipment and procedure occurred when the Pavement Technology Inc. (PTI) started a commercial development of the APA. The APA is the modified version of the Georgia LWT. In addition, the PTI is the developer of the AVC, which is used to compact either beam or cylindrical samples. The PTI formed the APA users group to share ideas and collectively worked toward refining the rut test procedure and other (fatigue) test procedure using the APA. During 1998 and 1999, the APA User Group performed a ruggedness study to identify the APA testing factors that have the greatest influence on the outcome of tests (West, 1999). Currently, a "Method of test for Determining Rutting Susceptibility Using the Asphalt Pavement Analyzer" is in the development stage (proposed to be included as an ASTM procedure).

A study by Jackson and Ownby, noted that the APA is capable of providing valuable data on permanent deformation, and it can be used in conjunction with the Superpave design (Jackson et al., 1998). Most recently, Kandhal and Mallick have shown that the APA is sensitive to aggregates, gradations and binder types and, therefore, has the potential to predict relative rutting of hot mix asphalt mixtures (Kandhal et. al., 1999). Mixes from poor, fair and good performing pavements were tested with the APA to develop rut depth criteria for the evaluation of mixes. They have found that in case of granite and limestone mixes, the gradation below the restricted zone generally showed the highest amount of rutting whereas, the gradation through the restricted zone showed the lowest rut depth. However, in case of gravel mixes, the gradation below the restricted zone showed the least amount of rutting whereas, the gradation above the restricted zone showed the maximum amount rutting. The APA was also found to be sensitive to the PG asphalt binders based on statistical significance of differences in rut depths. The rut depths of mixes with PG 58-22 asphalt binder (tested at 58°C) were higher than the depths of those mixes with PG 64-22 asphalt binder (tested at 64°C). In case of granite and limestone surface course mixes, the rut depth increased with an increase in asphalt film thickness. However, an opposite effect was observed in case of gravel surface course mixes, and binder course mixes containing granite and limestone. Based on very limited data, they suggested that the APA rut depth after 8000 passes should be less than 4.5 to 5.0 mm to minimize rutting in the field. However, more laboratory and field-test sections need to be evaluated to establish reliable criteria.

2.2.1 Asphalt Pavement Analyzer

The Asphalt Pavement Analyzer is a widely used piece of laboratory equipment designed to determine the rutting susceptibility of HMA mixes by applying repetitive linear wheel loads through pressurized hoses for compacting test specimens (Figure 2.1).

The APA specifications are as follows (Table 2.1). Dimensions of the device are 35 in (89 cm) x 70in (178 cm) x 80 in (203 cm), with weight of 3,000 lbs (1,361 kg), and the water tank capacity is eight cubic feet (0.226 m^3). The APA consists of the following basic components:

a. Wheel Tracking/Loading System (WTS), which consists of drive, loading, and valve assemblies with three special rubber hoses. The WTS applies wheel loading on repetitive linear wheel tracking actions that control magnitude and contact pressure on beam and cylindrical samples for rut testing.

b. Sampling Holding Assembly (SHA), consisting of sample tray and molds, holds the asphalt concrete samples directly underneath the rubber hoses to allow the samples to be subjected to the wheel tracking actions during rut testing. The sliding tray design allows the samples to be pulled out from inside the machine, making it easier to perform rut depth measurements and for installation of the sample.

c. Temperature Control System (TCS): the temperature of the APA chamber can be controlled and maintained accurately. The test and conditioning chamber temperatures are set at any point between 86°F and 140°F (30°C and 60°C) within ± 34°F (1°C).

d. Water Submersion System (WSS) consists of water tank, water tray and pneumatic cylinder. The WSS allows the water to cover the test sample during the submerged-in-water test and automatically drains the water upon completing the test before the sample tray is pulled out.

e. Operating Controls: all the controls for operating the machine are mounted on the control panel located in the front of the machine. The function of each feature on the control panel is self-explanatory.

f. Sample Temperature Conditioning Shelf is located inside the lower front doors. It can hold extra beams or cylindrical samples to allow heat soaking.

2.2.2 APA Results Versus Field Performance

The APA is the modified version of GLWT. The researchers showed that the GLWT was capable of ranking mixtures similar to actual field performance (Lai, 1986). A similar study conducted in Florida (West et. al., 1991) used three mixes of known field performance. One of these mixes had very good rutting performance, one was poor, and the third had a moderate field history. Again, results from the GLWT were able to rank the mixtures similar to the actual field rutting performance. The University of Wyoming and Wyoming Department of Transportation participated in a study (Miller et. al., 1995) to evaluate the ability of the GLWT to predict rutting. For this study, 150-mm cores were obtained from 13 pavements that provided a range of rutting performance. Results showed that the GLWT correlated well with actual field rutting when project elevation and pavement surface type were considered.

After the APA came on the market, the Florida Department of Transportation conducted a study (Choubane et al., 1998) similar to the GLWT study described previously (West et. al., 1991). Again, three mixes of known field performance were tested in the APA. Within this study, however, beams and cylinders were both tested. Results showed that both sample types ranked the mixes similar to the field performance data. Therefore, the authors concluded that the APA had the capability to rank mixes according to their rutting potential.

A joint study by the FHWA and Virginia Transportation Research Council (Williams et al., 1999) evaluated the ability of three LWTs to predict rutting performance on mixes placed at the full-scale pavement study WesTrack. The three LWTs were the APA, FRT, and HWTD. For this research, 10 test sections from WesTrack were used. The relationship between LWT and field rutting for all three LWTs was strong. The HWTD had the highest correlation (R^2 =0.91), followed by the APA (R^2 =0.90) and FRT (R^2 =0.83). Based upon review of the laboratory wheel tracking devices and the related literature detailing the laboratory and field research projects Cooley et al concluded that results obtained from the APA seem to correlate reasonably well to actual field performance when the in-service loading and environmental conditions of that location is considered (Cooley et. al., 2001).

2.3 Compaction of Rut Specimens

The compaction method used to prepare rut specimens is a significant component in any mix design and analysis method. The compaction methods evaluated by various researchers include: the rotating bases Marshall compactor, the Superpave Gyratory Compactor (SGC), and the Asphalt Vibratory Compactor (AVC). It is a standard practice in most agencies in the United States to design HMA by the Marshall mix design method in general accordance with ASTM D 1559-89 and the Asphalt Institute Manual Series Number 2. The Marshall compaction method was developed with close correspondence between the density achieved in the laboratory and density observed on the roadway after exposure to traffic (Roberts, et al., 1996). It has been argued that the impact compaction used in Marshall design does not adequately simulate the compaction during construction (Von Quintus, et. al., 1991).

The gyratory compaction was identified to be the most suitable method for a Superpave mix design project. The SGC can orient the aggregate particles in a way that is similar to that observed in the field and has the capability to accommodate larger aggregates (up to 50 mm) in the mix (Roberts, et. al., 1996). However, the SGC has a tendency to compact mixes in excess of what can be achieved with conventional paving equipment in the field. The bulk density values of the AVC compacted cylindrical specimens are similar to those of field compacted specimens.

2.3.1 Superpave Gyratory Compactor

The Superpave Gyratory Compactor (SGC) is a mechanical device that can be perceived as a modified version of the Texas Gyratory Compactor. The Superpave design procedure, at least Level 1 procedure, was rapidly becoming the standard HMA mix design method in the United States. However, there are some concerns from the asphalt industry in implementing the Superpave Levels 2 and 3 procedures because of the complexities of the apparatus needed and time required to perform these procedures. On the other hand, the Superpave Level 1 method alone is not sufficient for assessing permanent deformation of asphalt mixes (Lai, 1996). It employs the compaction principles of the French Gyratory Compactor. It is a device that was well suited to mixing facility quality control and quality assurance. The compaction angle of the SGC is 1.25 degrees, and the applied vertical load to the specimen is 87 psi (600 kPa). The loading ram diameter nominally matches the inside diameter (6 in or 150 mm) of the mold. This device can make from 30 to 40 gyrations per minute. A photographic view of the SGC is shown in Figure 2.2. The SGC consists of the following components:

a. Compactor Assembly, which is a rigid steel cubic construction.

b. Testing Mold Chamber, where the mold is placed with a safety door on the rotating set.

c. Specimens Extractor is equipped with an air cylinder to the extract compacted specimen.

d. Control Panel: remote control allows initialization, compaction time and height control of the specimen. Also, data can be stored on a diskette and printed out, as desired.

2.3.2 Asphalt Vibratory Compactor

A photographic view of the Asphalt Vibratory Compactor (AVC), Model AVCII, used in this study is shown in Figure 2.3. The AVC dimensions are 34 in (86.36 cm) x 50 in (127 cm) x 84 in (213.36 cm), and it weighs 2344 lbs (1063 kg). It requires compressed air of 3 SCFM @ 120 psi (827 kPa) and can be used for fabricating both cylindrical and beam samples, with the attachment of appropriate compaction heads. The AVC consists of the following components:

a. Compactor Assembly, which is a rigid steel frame mounted on noise absorbing isolators and supports.

b. Sample Table, where the compaction mold is placed. The AVC has provision for using two different steel molds; one is for preparing beam samples, while the other is for cylindrical samples.

c. Specimens Extractor is equipped with an air cylinder for the extraction of a compacted specimen.

d. Control Panel: remote control allows initialization, compaction time and height control of the specimen. The AVC is equipped with a power switch and button for emergency stop. It is also equipped with a switch for automatic operation.

When preparing samples with the AVC, the forward pressure should be kept at 14.5 psi (100 kPa) and the back pressure at 5.8 psi (40 kPa). The time to compact beam specimens can be fixed at 35 second. The Asphalt Vibratory compactor (AVC), developed by PTI, can be used to prepare beam or cylindrical samples with consistent bulk density values that can more closely simulate the compaction of asphalt mixes in the field than some other compactors (e.g., Texas Gyratory Compactor) (Jackson & Ownby, 1998).

2.3.3 SGC Versus AVC

In the SGC compaction is achieved through gyration, while in the AVC compaction is achieved through vibration. Vibratory compaction tends to result in more compaction at top and less compaction at the bottom of samples. This is generally true for both beam and cylindrical samples. Gyratory compacted samples, on the other hand, show less compaction in the top and the bottom of samples and significantly more compaction in the middle. In compaction by the AVC, orientation of the particles has been reported to be more representative of the field situation. With the SGC compaction, it is easier to achieve a desired level of density. While with the AVC compaction, it is difficult to reach the desired level of density (Cooley & Kandhal, 1999).

Volumetric properties were observed to be relatively uniform throughout the vibratory compacted specimens (Jackson and Ownby, 1998). However, the vibratory specimens do exhibit greater variability throughout a given specimen than was observed in the Marshall or Gyratory specimen. Cooley and Kandhal (October, 1999) evaluated the density gradients in terms of variation in air voids within samples common to the APA and compared the two types of compactive effort used for the APA samples: vibratory and gyratory compaction. They concluded that density gradient occurs in beam samples compacted with the AVC, cylindrical specimens compacted with the AVC, and cylindrical specimens compacted with the SGC. Vibratory compaction tends to result in more compaction at the top and less compaction at the bottom of samples. This was consistent for both beams and cylinders.

Gyratory samples showed less compaction in the top and bottom of samples. Significantly more compaction was noted in the middle. In general, AVC compacted specimens have significantly less density on the top when compared to the bottom. The AVC compacted specimens are recommended to apply loading on the top of specimen in the APA. The SGC compacted samples can be subjected to loading from any end because the density in the top and bottom layers have no significant difference.

2.4 Rutting Mechanisms

2.4.1 General

Flexible pavement carries load in shear deformation. An element of HMA layer subjected to traffic loading transfers the load from the surface to the underlying layers through intergranular contact and resistance to flow of the binder matrix. The stress pulse consists of vertical, horizontal and shear stress components. These stresses are transient and change with time as the wheel passes. The vertical and horizontal stresses are positive in unbound layers since unbound granular materials do not carry significant tensile stresses. The shear stress is reversed as the wheel passes and there is a rotation of the principal stress axes. Pavements with surface, base and subgrade of adequate strength and thickness can exhibit significant resistance to rutting (Button, 1990). Permanent deformation is generally considered to be the result of three mechanisms: consolidation, distortion, and attrition (Lekarp et al. 1996).

2.4.2 Distortion

Bending of flat particles, sliding and rolling of rounded grains are considered to be distortion. HMA materials flow laterally due to loss of interlocking of contracting particles, rather than densification (Gramling et al. 1991). This type of rutting is mainly caused by an asphalt mixture with very low shear strength to resist the repeated heavy loads to which it is subjected (Figure 2.4). Morris e al. (1974) conducted sophisticated triaxial tests with both the deviatoric and confining stresses applied dynamically. They found that the mechanism of rutting in asphalt concrete pavements, subjected to moderate tensile stresses, is almost entirely due to lateral distortion in the tension zone. There is no mechanistic-empirical model that adequately considers the lateral flow problem. However, in the laboratory, the APA can successfully be used to evaluate lateral flow, but with varying degrees of success.

2.4.3 Consolidation

The change in shape and compressibility of particle assemblies is considered as consolidation. Volume changes due to changes in grain arrangements, particle orientation, and generalized contraction of the assembly without modification of the soil structure. Rutting caused by densification of high air void mixtures are usually not considered during initial mix design. It is assumed that good engineering and construction practices will be followed, proper compaction will be achieved on the roadway. However, at high air void levels, one-dimensional densification can be a problem.

Consolidation type rutting normally occurs in subgrade, subbase, or base below the asphalt layer (Figure 2.5). Although stiffer paving materials will partially reduce this type of rutting, it is normally considered more of a structural problem rather than a materials problem. It is often the result of a too thin pavement section because there is simply not enough depth of cover on the subgrade to reduce the stress from applied loads to a tolerable level. It may also be the result of a subgrade that has been unexpectedly weakened by the intrusion of moisture.

2.4.4 Attrition

The change in a material's fabric and packing is considered an attrition. It is due to crushing and breakage of particles, particularly at inter-particle contact points. Permanent deformation can continue as long as attrition occurs in a granular assembly.

2.5 Rut Prediction Model

Although rutting is one of the most common problems in flexible pavements, no rational model to predict rutting has been developed that encompasses all field variables. Researchers have proposed a number of models to predict rutting, and some of them are briefly described:

2.5.1 Empirical Rut Models

Pavement features, structural and physical properties, loading and environmental conditions are statistically correlated in this category of model. Some of the existing empirical models are briefly reviewed.

Majidzadeh et al. (1978) developed a semi-empirical relationship for the rutting of asphalt concrete pavements in Ohio. There were many arbitrary parameters in their model that were not derived from the fundamental concepts as they did not have a physical meaning. Van de Loo (1974, 1978) developed a method for estimating rut depth due to permanent deformation of the bituminous layer. It was incorporated in the Shell Pavement Design Manual. These researchers estimated the rut depth from,

$$R = C_m h_1 \left(\frac{\sigma_{av}}{S_{mix}} \right) \qquad (1)$$

where,

R= rut depth,

C_m = correction factor for dynamic effect,

σ_{av} = average vertical stress in the asphalt concrete,

S_{mix} = the stiffness modulus of the mix, and

h_1= thickness of the asphalt layer.

Finn et al. (1986) proposed empirical equations to estimate permanent deformation (rutting) of pavements layers. AASHTO Road Test data was used to develop these empirical models. A very strong correlation was reported among traffic density, surface deflection, and stress on the top of the base. Sousa and Solaimanian (1994) correlated the rut depth with permanent shear strain (PSS). Permanent shear deformation resistance of laboratory compacted pavement materials was evaluated by *Harvey et al. (1995)* using the repetitive simple shear test-constant height (RSST-CH). These researchers modified the developed relationship in an attempt to correlate the model with available field data.

Pidwerbesky et al. (1997) postulated that the surface deflection and strains within the pavement layers increase linearly with increasing wheel load. The relationship between total surface deflection and subsurface strains within subgrade and pavement layers were given as,

$$\varepsilon_{cvb} = e^{6.19+0.47D} \quad \text{.......(2)}$$

$$\varepsilon_{cvl} = e^{6.66+0.43D} \quad \text{.......(3)}$$

$$\varepsilon_{cvs} = e^{6.19+0.72D} \quad \text{.......(4)}$$

where,

ε_{cvb} = peak vertical compressive strain (μm/m) at mid-depth of the base course,

ε_{cvl} = peak vertical compressive strain (μm/m) in the lower base course,

ε_{cvs} = peak vertical compressive strain (μm/m) in the subgrade, and

D = peak surface deflection (mm).

Chen and Lin (1998) proposed a general rutting model correlating the rutting to surface deflection, load repetitions, and compressive strength. This empirical model is based on field data from two sites, F5 in Victoria TX and US281S1 in Jacksboro TX, with the Texas Mobile Load Simulator (MLS),

$$\log(RR) = -7.424 + 1.151\log d - 0.486\log(N_{18}) + 1.26\log(\sigma_c) \qquad (5)$$

where,

RR = rate of rutting,

d = surface deflection in mm,

σ_c = vertical compressive stress on the top of base in kPa, and

N_{18} = number of 18-kip single axle repititions/100,000.

2.5.2 Mechanistic-Empirical Rut Models

Purely mechanistic models are based on some primary response (behavior) parameter such as stress, strain, or deflection. A purely mechanistic based model has not been developed yet because pavement engineers do not use primary or fundamental response parameters as ends in themselves. Rather, they are only useful if they can be related to pavement distress, or to pavement properties used in other models such as for overlay design. Consequently, the mechanistic-empirical type of deterioration modeling approach has been developed.

In mechanistic-empirical models, a response parameter (stress, strain, or deflection) is related to measured structural deterioration (roughness, cracking, rutting etc.) or functional deterioration (PSI, safety etc.) through the regression equations. In this approach, the mechanism of rutting is hypothesized and a structural response is related to rutting. Primary responses such as surface deflection, horizontal tensile stress, strain and strain energy at the bottom of the asphalt layer, and vertical stress and strain at the top of the subgrade are calculated. Attempts are made to relate these responses to observed distress and pavement conditions such as roughness, cracking, rutting through regression analysis.

Sousa et al. (1992) presented a comprehensive, combined viscoelastic-plastic model to characterize the rutting behavior of asphalt mixes. Their model included numerous constants that made it difficult to use. *Gillespie et al. (1993)* analyzed pavement deformation in different layers using the physical pavement model. The viscoelastic Poisson's ratio was set to 0.5 in all the layers. The layer viscosity were chosen so that the proportion of the overall permanent deformation occurring within each layer was the same, as reported in AASHO Road Test. It was reported that 32% of the overall permanent deformation occurred in the asphalt layer, 14% in the crushed stone base, 45% in the subbase, and 9% in the subgrade.

Zaghloul and White (1994) used a three-dimensional dynamic finite element (3D-DFEM) program (ABAQUS) to analyze flexible pavements subjected to moving loads at various speeds. A multilayer elastic analysis assuming static load and linear elastic material was used to verify 3D-DFEM predictions. A number of material models were used to represent actual material characteristics, such as viscoelasticity and elastoplasticity. They used two single-axle loads with dual wheels (80-kN and 258-kN) having a 2.8 km/hour speed. It was reported that the permanent deformation for the 80-kN load developed primarily in the asphalt layer, whereas 85% of the permanent deformation for the 258-kN axle load developed in the subgrade layer.

Rutting in the base course and asphalt surface, as a result of the 258-kN axle load, was about 10 and 5 percent of the total rutting, respectively. Collep et al. (1995) presented a model to determine the rut depth of asphalt concrete under repetitive loading, treating it as a linear viscoelastic flow phenomenon. A list of some of the constitutive equations reviewed in this section, including the name of the researchers who developed them, is presented in Table 2.2. Groenendijk et al. (1996) indicated that all rutting in AC pavements could be ascribed to subgrade deformation. Their test results revealed that less than 1% of total rutting occurs in the AC layer. They conducted research on two test pavements of 0.15m and 0.08m gravel AC on a 5-m sand subgrade 75-kN super-single wheel load using the linear tracking device. No shear deformation within the asphalt layer was observed in their study. They reported a relationship between subgrade strains due to a wheel load as,

$$\varepsilon_{subgrade} = 2.8\times10^{-2}\times N^{-0.25} \qquad (6)$$

where,

$\varepsilon_{subgrade}$ = permissible strain at the subgrade surface (m/m); and

N = allowable number of load application.

Bonaquist and Witczak (1997) used finite element approach with constitutive model to analyze pavement response including permanent deformation or rutting. The permanent strains for a given state of stress were represented as,

$$\xi = \left[\frac{0.002408\sqrt{\gamma}\left((I_1 + k/\sqrt{\gamma})/P_a\right)^{3.05}}{\gamma\left((I_1 + k/\sqrt{\gamma})/P_a\right)^2 - J_2/P_a} \right]^{2.22} \qquad (7)$$

where,

ξ = plastic strain trajectory for load cycle N,

I_1 = first invariant of the stress tensor;

J_2 = second invariant of the deviatoric stress tensor,

P_a = atmospheric pressure,

k = Drucker-Prager cohesion parameter, and

γ = material parameter.

According to these researchers, these permanent strains can be summed over the thickness of the pavement to obtain the permanent deformation in the pavement. With the total stresses known, the above equation can be solved for the corresponding permanent strains using the appropriate Drucker-Prager strength parameters.

Ali et al. (1998) developed a mechanistic model to predict rut depth as a function of the vertical compressive elastic strain in all pavement layers. The model was derived from a well-established plastic deformation functional. To be compatible with mechanistic analysis, the model form allows the characterization of traffic in terms of loading groups, rather than ESALs. The proposed model form was developed based on the assumption that the relationship between the plastic and elastic strains is linear, for all pavement layers. It further assumes that this relationship is nonlinear in terms of the number of load applications. The model parameters indicate that the AC layer contribution to surface rutting is marginal. The combined base/subbase layer contributed the most to the measured rutting. The contribution of the subgrade to the measured rutting was greater than that of the AC layer, but less than that of the base layer.

Ramsamooj et al (1998) predicted the stress-strain response of asphalt concrete pavement under cyclic loading using an elasto-plastic model. It was reported that the primary component of rutting at temperature up to 32°C is the plasticity of the asphalt concrete, and the amount of rutting can be predicted from the fundamental properties and the stress-dilatancy theory. It was concluded that selecting dense graded asphalt concrete or styrene-butadiene-styrene modified asphalt concrete with a higher value of coefficient of lateral earth pressure, which depends on aggregate interlocking and aggregate characteristics, could decrease the rutting.

2.5.3 Neural Network Rut Models

In recent years neural network (NN) modeling has emerged as a very powerful tool to find correlations between dependent and independent variables in a set of data. A typical deformation analysis deals with finding the stresses and displacements due to static and dynamic loads, and with the verification that the structure is sufficiently stable under such loads. Deformation analysis is a complex scientific domain incorporating many traditional methods or mathematical models. These models may be based on differential, variational or integral formulations. The first approach deals with (partial) differential equations, to be solved by integration, subject to some boundary conditions. The second approach uses test functions that find the stationary value of some functional, subject to satisfying the boundary conditions. The third approach is based on the reciprocity theorem and deals with integral equations to be solved on the structure boundary. All of these modeling techniques are useful only when the physics or mechanics of a problem is known or can be expressed in a differential equation form.

Rutting as the focus of the study is a complex problem that is poorly understood. There are an infinite number of variables (some of the variables are listed in Table 3) in the different types of aggregates, combination of aggregates, and the variety of binders used in making asphalt pavements which make modeling as well as accurate prediction of rutting very difficult. On the other hand, NN is a modeling technique, which is particularly useful when physics or mechanics of a problem is too complex to express in a differential equation form, includes a large number of parameters that is poorly understood. It is a very powerful tool to determine correlations between dependent and independent variables in a large set of data. It has high-speed parallel processing property with an inexpensive simulation. Therefore, the choice of the study to employ such modeling technique to evaluate rutting is a good decision.

A neural network (NN) is an interconnected assembly of simple processing units or nodes (called neurons) used to represent the mapping or relationship embedded in any set of data. The architecture of a network allows it to approximate the mapping function in the absence of knowledge about the mathematical form of the mapping between an input signal and the corresponding output signal. The approximating ability of a NN is stored in the interconnections (called weights) obtained by a process of adaptation to or learning from a set of training patterns. In NN modeling procedure, a representative sample data set that includes a set of input signals and their corresponding output signals is used to determine the connecting weights in each mapping. The weights are updated in an iterative manner until the difference between the predicted output signals and the actual signals corresponding to the input signals is negligible. This weight updating process is called training. The trained network is then subjected for validation. The validated model can propagate a new input signal through the network and predict the resulting output signal.

Creating a neural net solution to a problem involves the steps of defining inputs, designing network architecture and algorithm, training the network on examples of the problem, and running the trained network to solve new examples of the problem. The input of a neural net consists of a series of known values. The values can vary from one to n-dimensional array of known numbers. The structure of NN mainly consists of an input layer made of several input nodes that are presumed by the designer to account for and explain the variability observed in the outputs of the problem. The output layer is designed to contain output nodes (variables). An intermediate layer (hidden layer) contains a number of units that have no interaction with the external environment but are interconnected with the nodes of other layer. The nodes in a certain layer are connected with the nodes of other layers.

In NN architecture, each neuron consists of multiple inputs in which each input is connected to either the output of another neuron or one of the input numbers. The neuron consisting of single output is connected to the input of other neurons or to the final output. Each connection is assigned an initial synaptic strength. These weights can start out all the same, assigned randomly, or determined in evolutionary depending on the network algorithm. Once the neuron and connections are set up, each weighted input to the neuron is computed by multiplying the output of the other neuron (or initial input) that the input to this neuron is connected to by the synaptic strength of that connection.

All of these weighted inputs to the neuron are summed. If this sum is greater than the firing threshold of this neuron, then this neuron is considered to fire and its output is 1. Otherwise, its output is 0. Repeated trials on sample problems are executed. After each trial, the synaptic strengths of all the inter-neuronal connections are adjusted to improve the performance of the neural net on this trial. This training is continued until the accuracy rate of the neural net is no longer improving. The dynamics of the network can be described perfectly by the state transition table or diagram. However, greater insight may be derived if the dynamics can be expressed in terms of energy function, and using the formulation, if it is possible to show that the stable states can always be reached in the developed network. Figure 2 represents a mechanism-based flow diagram, which will be incorporated for the development of a neural architecture. This type of study will employ the programming language MATLAB to carry out the training and prediction as well as for model development.

Simpson et al. (1995) developed a neural network (NN) model using the LTPP data. The independent variables as used by Simpson et al. (1995) are: AC thickness, air void, asphalt cement viscosity, annual precipitation, freeze-thaw cycles, plasticity index, subgrade moisture, subgrade passing #200 sieve, base thickness, and cumulative ESALs. According to this study, a strong relationship exists between the transverse surface rutting profile and the contributions of different layers to rutting. However, they did not provide the adequate information about the NN architecture, the training scheme used, and data sets used for training, and validation. Also, no information was given on the weighting matrix of the trained network, which makes it difficult for others to use their NN model.

2.5.4 Other Rut Models

A number of procedures are available for the estimation of the amount of rutting from repeated traffic loading. One of the analysis approaches follows elastic layered theory, in which materials are characterized by repeated load triaxial test or creep test. Another approach follows visco-elastic layered theory, in which materials are characterized by creep test. Although several techniques have been proposed for the second approach, it has not been widely used because of the complexity in obtaining elasto-plastic or visco-plastic characterization for the various paving materials.

Elastic Layered Approach

A pavement system can be represented as a layered elastic system that can be determination by the state of stress or strain, resulting from surface loading. The total rut depth can be estimated by summing the contribution from each layer, i.e.,

$$\delta_i^p(x,y)=\sum_{i=1}^{n}(\varepsilon_i^p\,\Delta z_i)\qquad(8)$$

Where,

δ_i^p = rut depth in the ith position at point (x, y) in the horizontal plane,
ε_i^p = average permanent strain at depth $[z_i + \Delta z_i/2]$, and
Δz_i = difference in depth.

Viscoelastic Layered Approach

Pavement is represented as a viscoelastic-layered system. This methodology requires the determination of creep compliance of each material in each layer at a given time.

VESYS Approach

Permanent strain due to a single load application is proportional to the elastic or resilience strain at the 200th load repetition,

$$\varepsilon_p(N) = \mu\varepsilon_{200}N^{-\alpha} \quad (9)$$

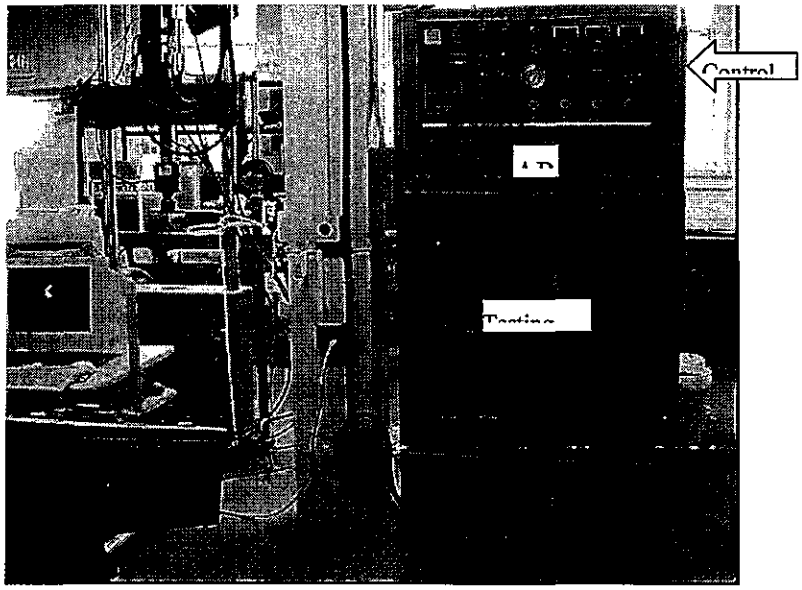
where,

$\varepsilon_p(N)$ = Permanent or plastic strain at *N*th load application,

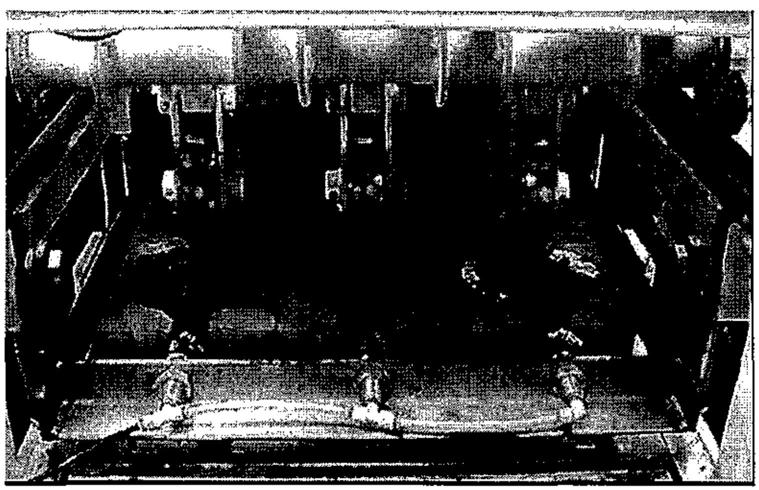
ε_{200} = Elastic or resilience strains at 200th load repetition,

μ = constant of proportionality between elastic and plastic strain, and

N = Load application number, α = constant, representing the permanent deformation rate.

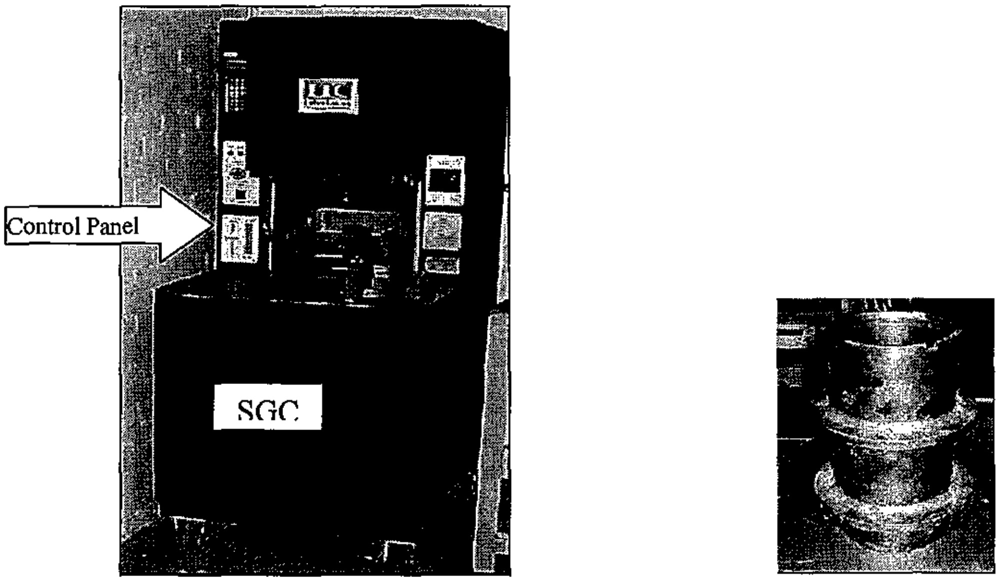


(a) Asphalt Pavement Analyzer (APA)



(b) Inside View of APA Chamber

Figure 2.1 Photographic View of Asphalt Pavement Analyzer (APA)



(a) Superpave Gyratory Compactor (SGC) (b) Compaction Mold

Figure 2.2 Photographic View of Superpave Gyratory Compactor (SGC)

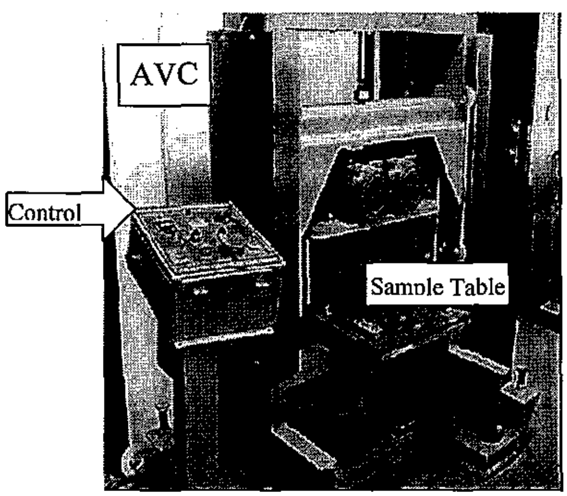


Figure 2.3 Photographic View of Asphalt Vibratory Compactor (AVC)

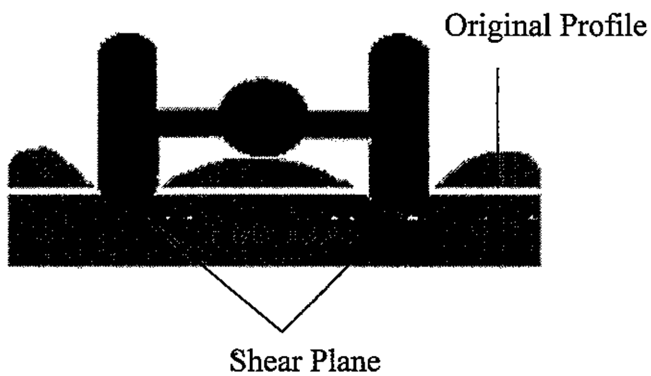


Figure 2.4 Rutting from a Weak HMA

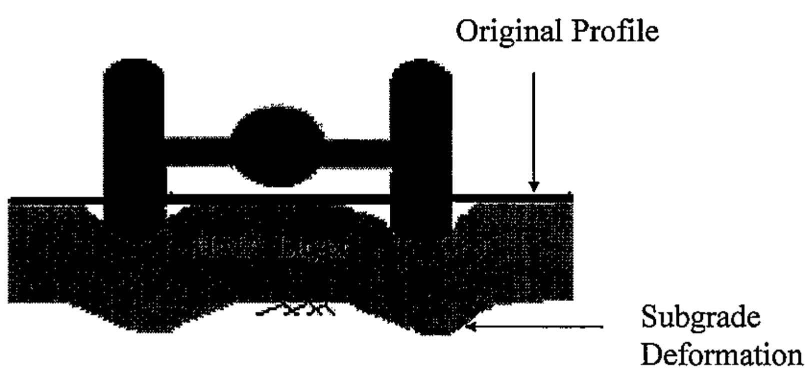


Figure 2.5 Rutting from a Weak Subgrade

Table 2.1 APA Testing Protocol

Specimen Dimensions

Specimen type	Cylindrical	Beam
No. of specimens tested simultaneously	6	3
Specimen size (mm)	300x150x75	150x125x75

Environmental Conditions

Range of test temperature	60-64 ^{0}C
Environmental condition	Dry cycle testing or Wet cycle testing

Wheel Configurations

Wheel speed	0.6 m/ s
Wheel type	Aluminum wheel on pressurized hose
Hose pressure	100 (psi)
Hose size	29 mm diameter
Load	100 (psi)
Load cycle	80,000

Measurements

Rut depth measurement....................	Three locations centered 90 mm about the center of the specimens
Method of rut depth measurement.	Automatic linear voltage displacement Transducers
Acquisition of data.	Automatic
Frequency of measurement................	Every 250 wheel passes

Table 2.2 Prediction Equations from Repeated load Tests

Developer	Material	Constitutive Equation from Repeated Load Tests
Snaith	Asphalt concrete	$\log \varepsilon^{p} = (a + b \log t)$
McLean & Monismith	Asphalt concrete	$\log \varepsilon^{p} = C_0 + C_1(\log N) + C_2(\log N)^2 + C_3(\log N)^3$
Freeme and Minismith	Asphalt concrete	$\varepsilon_z^{p} = cN^{\alpha}(\overline{\sigma})^{n-1}[\sigma_z - 1/2(\sigma_x + \sigma_y)]$ $\overline{\sigma} = 1/2[(\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2$
Barksdale	Granular material	$\frac{\varepsilon_p}{\overline{\sigma}} = \frac{(N/N_0)^m}{K}\{1 - [\overline{\sigma} R_f (1 - \sin\phi)]/[2(D\cos\phi + \sigma_3 \sin\phi)]\}$

CHAPTER 3

EXPLORATORY AND BASELINE TESTS

3.1 General

Initially, three mixes were selected for rut testing with the co-operation of ODOT. One mix (Project ID: NHY-8N (005) and Design ID: 3012-OAPA-99037) was selected to be evaluated by "Exploratory Tests". Another two-gravel mix (Project ID: NHY-8N (005) with Design ID: 3011-OK99-63070 and Design ID: 3011-OK99-63071) was selected for "Baseline Tests". Aggregates and asphalt binders were supplied by ODOT. The contractors supplied the source of materials and the proportions used for batching and mixing. The Job-mix formula (JMF) recommended by the contractors was followed for this research. However, combined aggregate gradation for selected percentages was computed and compared with the requirements as a counter check of contractor's specification.

The Average daily traffic for the pavements constructed with these mixes was more than three million ESALs. The rut test temperature has to be representative of the environment in which the paving mixture was utilized and ranged from 58°C to 64°C. Aggregate tests performed by the contractors included: gradation, Los Angeles abrasion, sand equivalent, durability, insoluble organic contents, fractured faces, insoluble residue, and effective specific gravity. Mix information is given in Table 3.1.

3.2 Aggregate Tests

Gradation tests were performed for all mixes. It is perhaps the most important property of an aggregate. It affects almost all the important properties of a HMA, including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture damage. Therefore, gradation was a primary consideration in asphalt mix design, and the specifications used by most states limit the gradations that can be used in HMA. Figure 3.1 shows the gradation, which was a straight line on the 0.45 power gradation paper. The gradation used in the mixture plots a smooth curve and above the maximum density line. This mix should have high resistance to deformation under load. Figure 3.2 shows the gradation of two base mixes. Sieve analysis (ASTM C 136 or AASHTO T 27) was performed during mix production.

The L.A. abrasion test was performed to check the design specifications. The L.A. abrasion test is most often used to obtain an indication of desired toughness and abrasion characteristics of aggregate. The test method ASTM C 131 or AASHTO T 96 is a measure of degradation of mineral aggregates. It gives a combination of actions including abrasion or attrition, impact, and grinding for a prescribed number of revolutions in a rotating steel drum containing with a specific number of steel spheres. This test has been widely used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions. Both the exploratory and the base aggregate have a L.A. abrasion value of about 29.

The Sand Equivalent Test was performed to determine the relative proportions of plastic fines and dust in a fine aggregate mix. Dust specially, clay adhering to aggregate, prevents good bond between the asphalt binder and aggregate. In this test, the amount of clay was measured (ASTM D 2419 or AASHTO T 176). The sand equivalent is the ratio of the height of sand to the height of clay times 100. Both aggregate showed a higher sand equivalent value than the minimum specified sand equivalent of 45.

Aggregate particles with more fractured faces exhibit greater interlock and internal friction, and hence result in greater mechanical stability and resistance to rutting than do the rounded particles. Currently, there is no ASTM or ASSHTO standard test procedure for measuring the percentage of fractured faces for an aggregate. In this study, a sample of coarse aggregate (retained sieve No. 8) was divided into 3 stacks. The particles that had none, one, and two or more fractured faces were counted. One stack contained all the particles with zero fractured faces. The second stack contained all particles with one fractured face, and the third stack contains all particles with two or more fractured faces. The percentage by weight of each stack with one or more fractured faces and with two or more fractured faces was then determined (OHD Designation: L 18). The exploratory mix had greater number of fractured faces when compared to the base mixes.

All batch aggregate were tested for effective specific gravity. Specific Gravity of aggregate is the ratio of the mass (or weight in air) of a unit volume of coarse material to the mass of the same volume of water at stated temperatures. The specific gravity of coarse aggregate is useful in making weight-volume conversions and in calculating the void content (ASTM C 29) in a compacted mix. Absorption is the increase in the weight of aggregate due to water in the pores of material, but not including water adhering to the outside surface of the particles, expressed as a percentage of dry weight. The aggregate is considered dry when it has been maintained at a temperature of $110 \pm 2^{0}C$ for sufficient time to remove all uncombined water. Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pores spaces within the constituent particles (ASTM C 127 and C 128or AASHTO T 85 and T 84).

3.3 Mixture Test

The mixture test was performed as follows. Batch aggregate was dried and sieved into sizes (preferably individual sizes) and 3 percent moisture is added to a minus no. 10 sieve aggregate to prevent segregation. Batch aggregate was then heated to mixing temperature. Asphalt cement must be heated to achieve a viscosity of 170±20 centistoke. For modified asphalt binders, the compaction temperature recommended by the binder manufacturer is used. The temperature for mixing and testing is listed in Table 3.2. Asphalt and aggregates were mixed using a mechanical mixer. Laboratory prepared specimens must be compacted to contain 7.0±1 percent air voids using the AVC.

The bulk specific gravity for each specimen was determined by weighing in air. This test was conducted in accordance with ASTM D 2726 (AASHTO T 166). Rice specific gravities on the loose HMA mix samples were measured in accordance with AASHTO T 209 (ASTM D2041). Air void contents of the test specimens were determined in accordance with ASTM D 3203 (AASHTO T 269). The rut test was performed in accordance with the OHD L-43 procedure. The bulk specific gravity of the compacted bituminous mixture (lab-molded specimen) was used in calculating the unit weight of the compacted mixture (ASTM D 2726 or AASHTO T 166). The steps in determining bulk specific gravity involve in weighing the compacted specimen in air (W_D), submerging the samples in water and allowing saturation prior to getting submerged weight in SSD condition (W_{sub}), then removing the sample and weighing in air in saturated surface dry condition (W_{SSD}). Bulk Specific Gravity, $G_{mb} = W_D / W_{SSD} - W_{sub}$. The density of each specimen is then calculated using water density, $\rho_s = G_{mb} \times \rho_w$; The air void content in the compacted dense-graded HMA specimen at optimum asphalt content is suggested by most agencies to lie between 3 and 5 percent (ASTM D 3203 or AASHTO T 269).

Air voids in asphalt concrete cannot bear stress. Lower air void content results in greater stiffness because it reflects a more homogeneous structure with better stress distribution. Using the bulk specific gravity (G_{mb}) and the Rice Specific gravity (G_{mm}), the percent air void can be calculated as, $\% \, Air \, Void = (1 - G_{mb} / G_{mm}) \times 100$; VMA is the total volume of voids within the mass of the compacted aggregate. It is calculated using the bulk specific gravity of the aggregate (G_{ab}), the bulk specific gravity of the compacted mix (G_{mb}) and the asphalt content by weight of total mix (P_b). It can be calculated using the formula, $VMA = (1 - G_{mb} x \, (1-P_b) / G_{sb}) \times 100$;

There are a number of states that include percent voids filled with asphalt cement. If a specifying agency includes a VMA requirement and exercises air void control during construction, percent VFA is a redundant requirement for dense graded HMA. Most states that include percent VFA requirements generally specify that the VFA range is from 70 t0 85 percent. VFA for each specimen can be calculated using the percent void and VMA as, VFA = 100 x (VMA - %Void)/VMA.

3.4 Data Analysis

Figure 3.3 shows a typical rut versus number of cycles for exploratory mix. It can be seen as a small difference in rut value between the left and middle samples. However, rut depth varies about 1 mm between the left and right samples. This is due to the difference in air voids. The testing parameters are listed in Table 3.3. Initially, the AVC was used for rut testing. The asphalt content was varied for different tests and samples. A sample of rut versus cycles data is shown in Table 3.4. Both of this table data will be useful for neuron based model development.

From Figure 3.3, it can be seen that rut depth at 64 degree centigrade is more than double of rut depth at 60 ^{0}C. There is no clear trend of increasing rut depth with the increasing air voids as in Figure 3.4. It is also seen that the SGC samples are more uniform in consideration of air voids. For samples with air void more than 5%, rut depth increases with the increase of air void. For samples with air void less than 4%, rut depth actually increases with the decrease of air voids.

Figure 3.5 shows air voids, percent asphalt content and rut depth for one of the base mixes. The percent asphalt content is in the design range. Therefore, the rut depth did not vary too much from sample to sample. The AVC samples shows higher rut depth when compare to the SGC sample. Here only 20 samples data are shown. Other data is included in chapter 6.

A total of 26 samples data was plotted in a bar chat as in Figure 3.6. The rut depth at 60 ^{0}C is about 4.5 mm. But the rut depth at 64 ^{0}C is about 6mm. The rut depth for the gravel mix is higher than the exploratory mix. Once again, the air void was not in the range of 6-8%. However, this data will be useful in developing a neural network model.

Figure 3.7 shows the correlation of rut depth with air voids. A poor correlation was obtained for this base mix. Therefore, air void is not the primary factor for rutting of gravel mix. Rather, the round shape of the particles might be responsible for higher rut.

Figure 3.8 shows the effect of gradation on rut depth for all of these three mixes. It can be seen that the mix (3011-OK-63072), which gradation passes through the restricted zone, showed maximum rut depth. Of the two mixes passing above the maximum density line, the exploratory mix showed less rut potential compared to the base gravel mix (3011-OK-63071).

The NCAT mix was added to enrich the baseline database. Two mixes, one type B and a Superpave mix, were included in the NCAT mix. A total of 12 samples (each mix with six samples) were tested for rut. The test result is plotted in Figure 3.9. The SGC was used for compaction. The Type B mix showed a rut depth of 2 mm, whereas the Superpave mix showed a rut depth of about 2.2 mm. Therefore, from the APA data, it can be concluded that the superpave mix is not performing better than traditional B mix. However, field data will be helpful in validating such performance of the mixes.

Table 3.1 Selected Mix Information

Selected Mix Design No.	...	3012-OAPA-99037	3011-OK99-63070	3011-OK99-63071
Asphalt Concrete Type	...	B Insoluble	A	A
Project No.	...	NHY-8N(005)-10088(13)	NHY-8N(005)-10088(13)	NHY-8N(005)-10088(13)
Highway	...	US54	US54	US54
Avg. Daily Traffic	...	3M+	3M+	3M+
Contractor	...	Duit Construction	Duit Construction	Duit Construction
Producer	...	Highway Contractors Inc.	Highway Contractors Inc.	Highway Contractors Inc.
Blended Materials	Source		% Used	
1-1/2" Rock	Vega Sand & Gravel @ Oldham Co., Tx.	...	15	15
3/4" Chips	Vega Sand & Gravel @ Vega, Tx.	25	20	30
3/8" Chips	Vega Sand & Gravel @ Vega, Tx.	30	...	...
Crushed Gravel	E.D. Baker Corp. @ Borger, Tx.	...	38	20
Screenings	Vega Sand & Gravel @ Vega, Tx.	30	27	35
Sand	Long Pit @ Texas County, Ok	15	...	...
		Asphalt Information		
Asphalt Type	...	PG70-28	PG64-22	PG64-22
Asphalt Content	...	5.0 - 6.0	4.5 - 5.5	4.3 - 5.3
Asphalt Source	...	Royal Trading @ Tulsa, OK	Total Petroleum @ Armore, OK	Total Petroleum @ Armore, OK
Asphalt Sp. Gr. @ 77	...	1.0177	1.0078	1.0078
Aggregate Property	Required			
Sand Equivalent	45 min.	48	61	46
L.A. Abrasion % Wear	40 max.	29.5	28.9	28.9
Durability (DC)	40 min.	76	78	78
IOC		0.34	0.42	0.53
Insoluble Residue (Ca)	40 min.	80	0	N/A
Fractured Faces	75 w/2	83	83	79.1
ESG		2.657	2.636	2.649
Mixture Property	Required			
Compaction (% of G_{mm})	94 - 96	...	...	...
VMA, (Min. %)		15	13	13
Retained Strength (%)	75	...	...	...
Hveem Stability (Min)	40	...	...	...

Table 3.2 Mixing and Testing Temperature

Procedure	Temperature (^{0}F)			Time (hr)
	3012-OAPA-99037	3011-OK99-63070	3011-OK99-63071	
Oven drying of Aggregate	230	230	230	over-night
Gradation Test	77	77	77	> 2
Preheating Aggregate	325+/-10	325+/-10	325+/-10	>1.5
Mixing	325+/-10	325+/-10	325+/-10	3 minutes
Short-Term Aging	305+/-10	290+/-10	305+/-10	>2 < 4
Compaction	305+/-10	290+/-10	305+/-10	35 sec
Cooling	77	77	77	>4
Density and G_{mm} Test	77	77	77	0.5
Sample Conditioning	147.2	147.2	147.2	>10
Testing	147.2	147.2	147.2	2.5

Table 3.3 Rut Parameter for Mix ID: 3012-OAPA-99037

Design No. 3011-OK99-63037			
Parameters Included	Left	Middle	Right
Asphalt content	5.75	5.75	5.25
Bulk Specific Gravity	2.333	2.364	2.372
Maximum Sp. Gravity	2.432	2.432	2.450
% Air void	4.1	2.8	3.2
% Material passing #200 sieve	6%	6%	6%
% Material passing #10	40%	40%	40%
Test Temp	64	64	64
Fractured Face	75 w/2%	75 w/2%	75 w/2%
% Natural Sand	15	15	15
Binder Specific Gravity at 77 degree Celsius	1.0177	1.0177	1.0177

Table 3.4 Rut-Cycle Relations

Number of Cycle	Rut (mm)		
	Left Specimen	Middle Specimen	Right Specimen
1	0.000	0.000	0.000
2	0.000	0.000	0.000
3	0.003	0.001	0.005
4	0..0039	0.003	0.008
5	0.006	0.005	0.010
6	0.025	0.008	0.016
7	0.046	0.009	0.049
8	0.070	0.018	0.075
9	0.081	0.030	0.085
10	0.092	0.033	0.096
20	0.093	0.036	0.103
30	0.110	0.038	0.141
40	0.171	0.108	0.199
50	0.219	0.171	0.236
60	0.246	0.198	0.269
70	0.276	0.224	0.292
80	0.314	0.253	0.310
90	0.344	0.276	0.329
100	0.376	0.324	0.341
200	0.509	0.494	0.498
300	0.637	0.651	0.635
400	0.744	0.746	0.685
500	0.834	0.802	0.717
1000	1.139	1.145	1.016
1500	1.353	1.457	1.278
2000	1.553	1.646	1.472
3000	1.988	1.991	1.769
4000	2.453	2.462	2.072
5000	3.049	2.999	2.415
6000	3.712	3.625	2.867
7000	4.491	4.311	3.380
8000	5.266	4.965	3.987

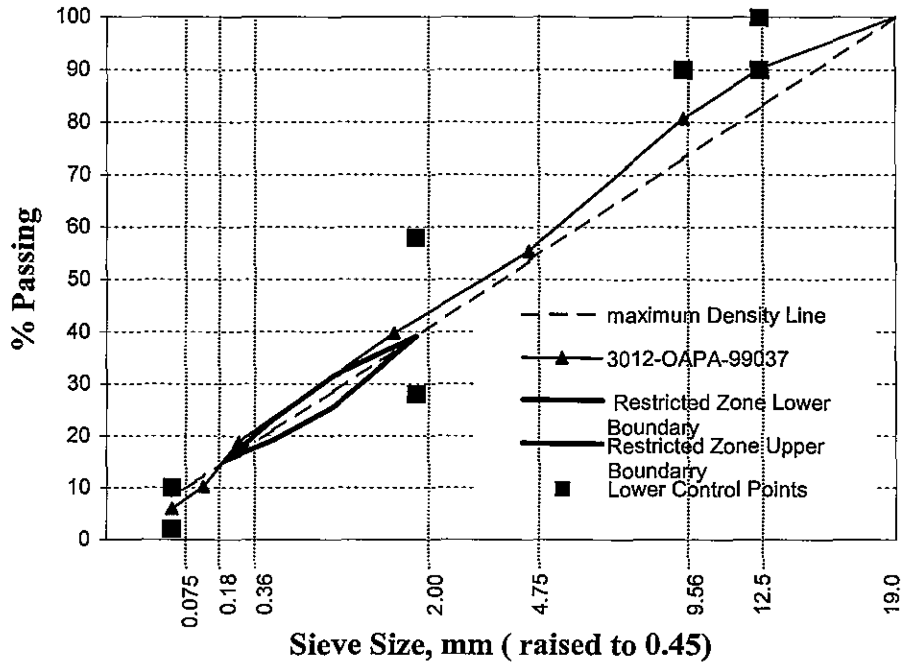


Figure 3.1 Gradation plot of Exploratory mix on 0.45 power chart

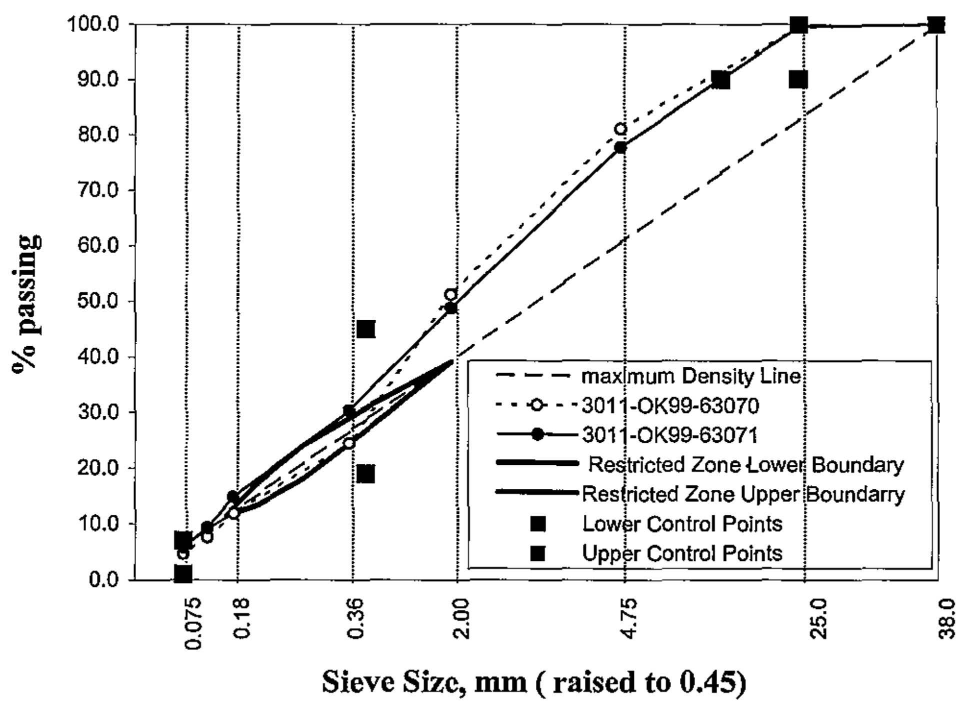


Figure 3.2 Gradation Plot of Base Mixes on 0.45 power chart

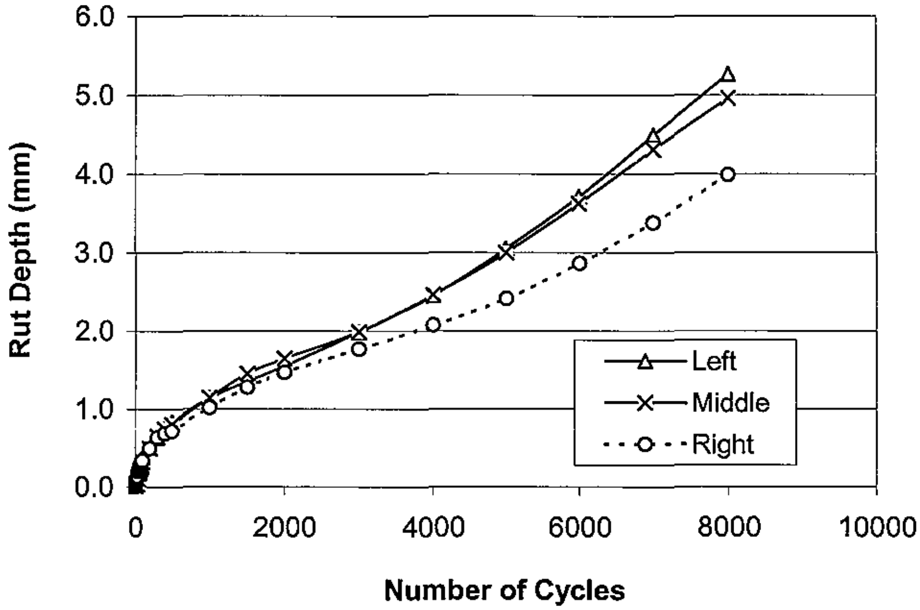
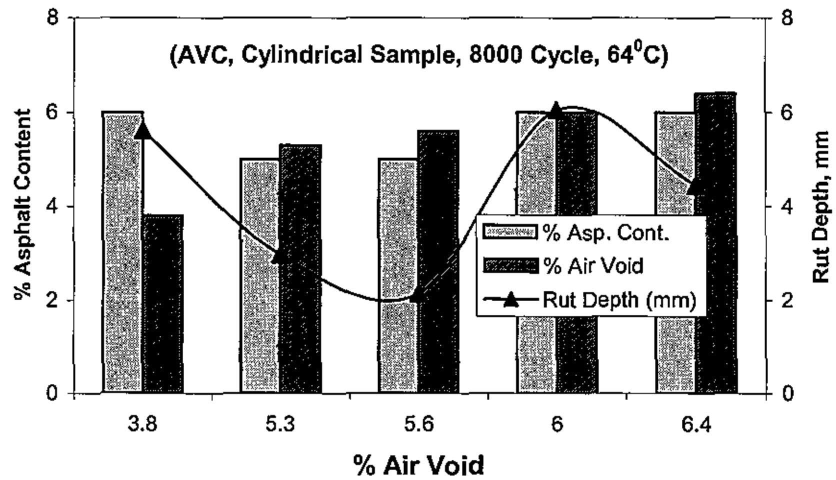
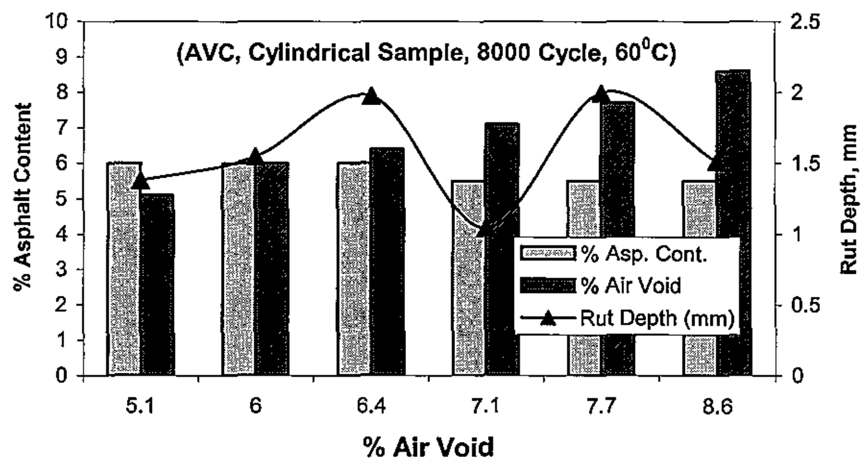


Figure 3.3 Typical Rut Plot of Exploratory Mix ID: 3012-OAPA-99037





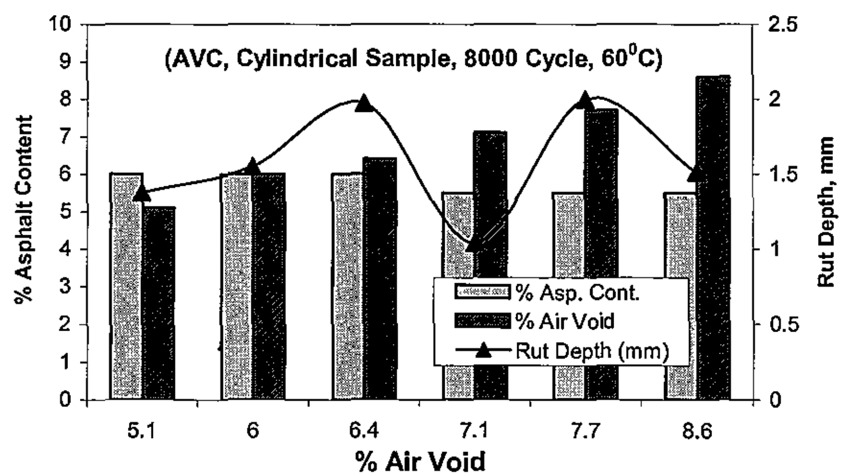


Figure 3.3 Rut Plot of Exploratory Mix (3012-OAPA-99037)

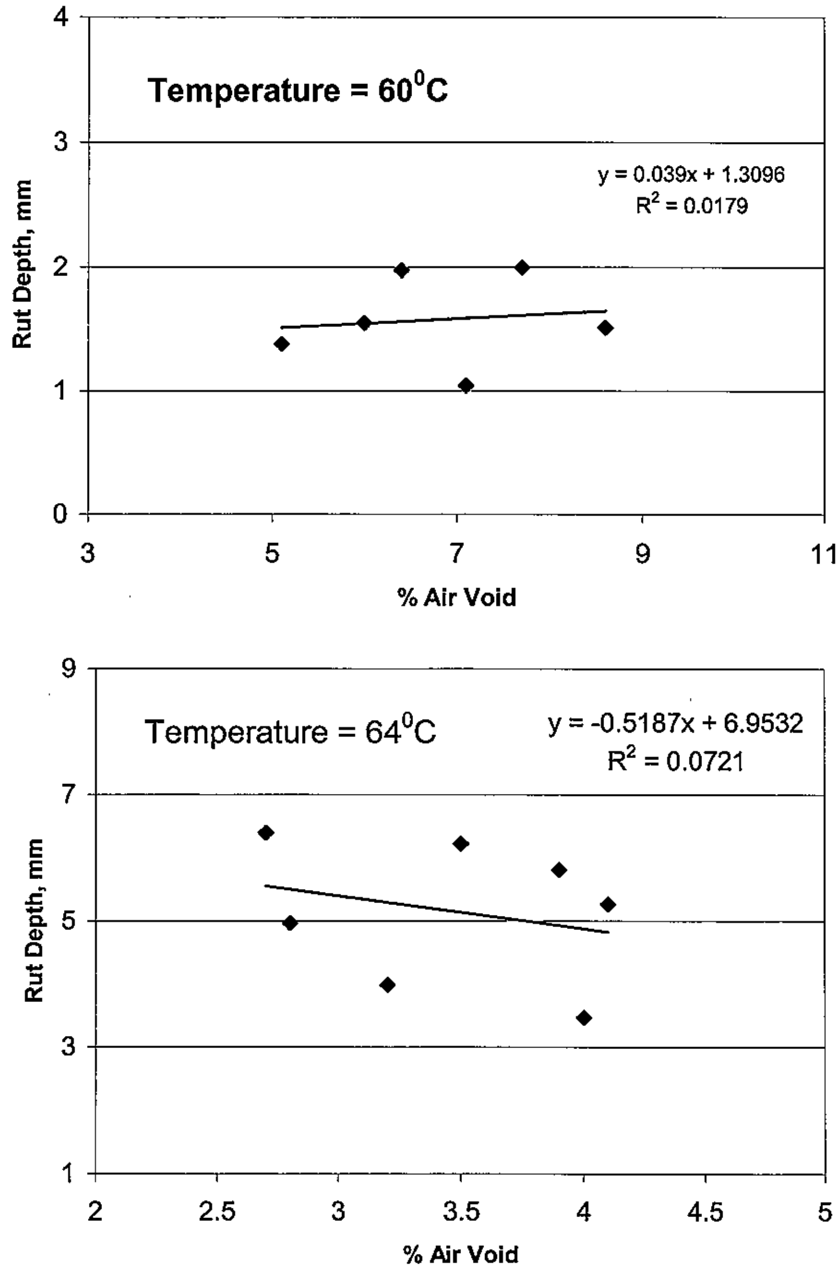
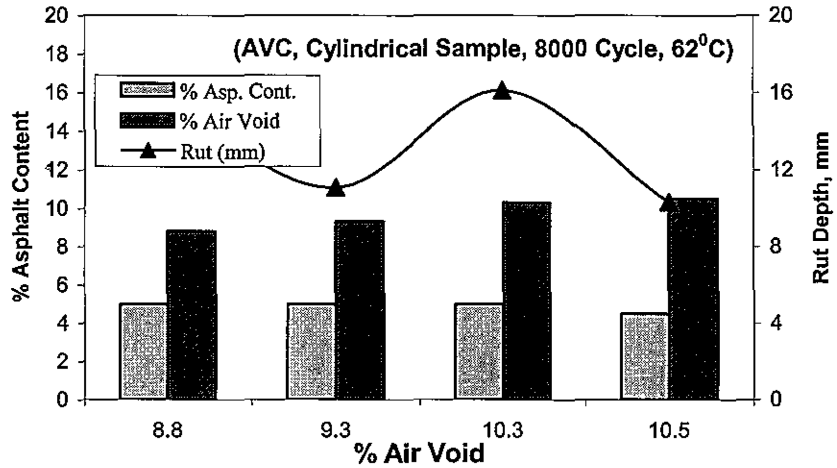


Figure 3.4 Correlations Between Rut And Air Void



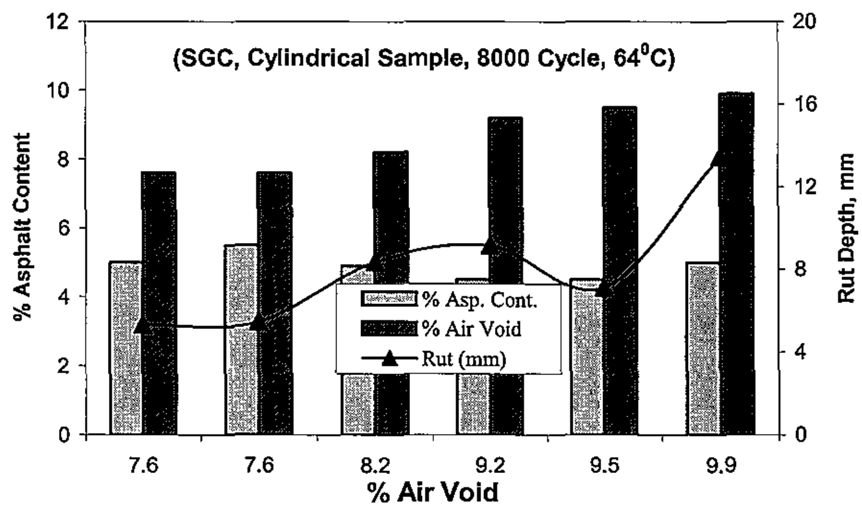


Figure 3.5 Rut Plot of Base Mix (ID: 3011-OK99- 63070)

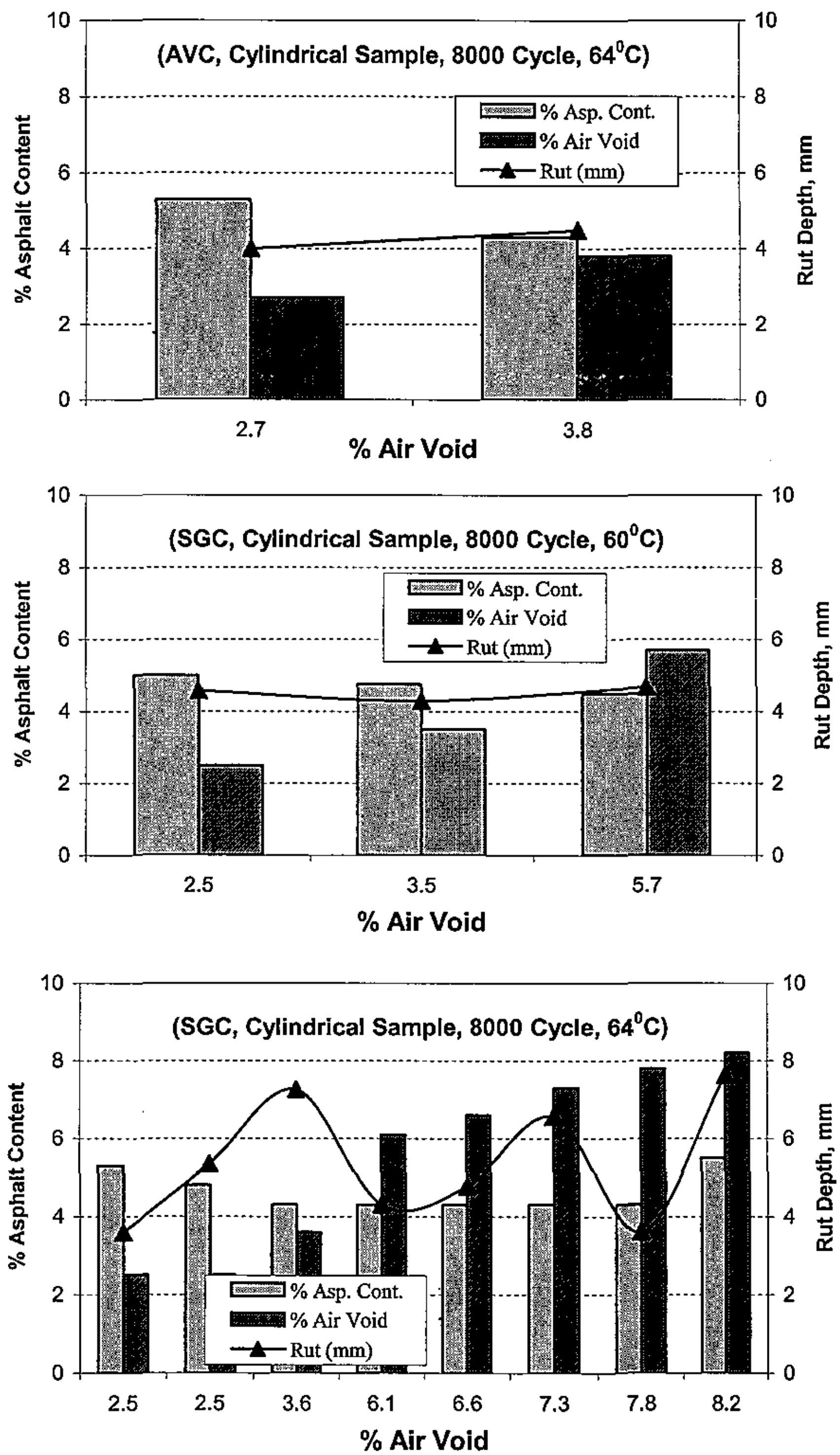


Figure 3.6 Rut Plot of Base Mix (ID: 3011-OK99- 63071)

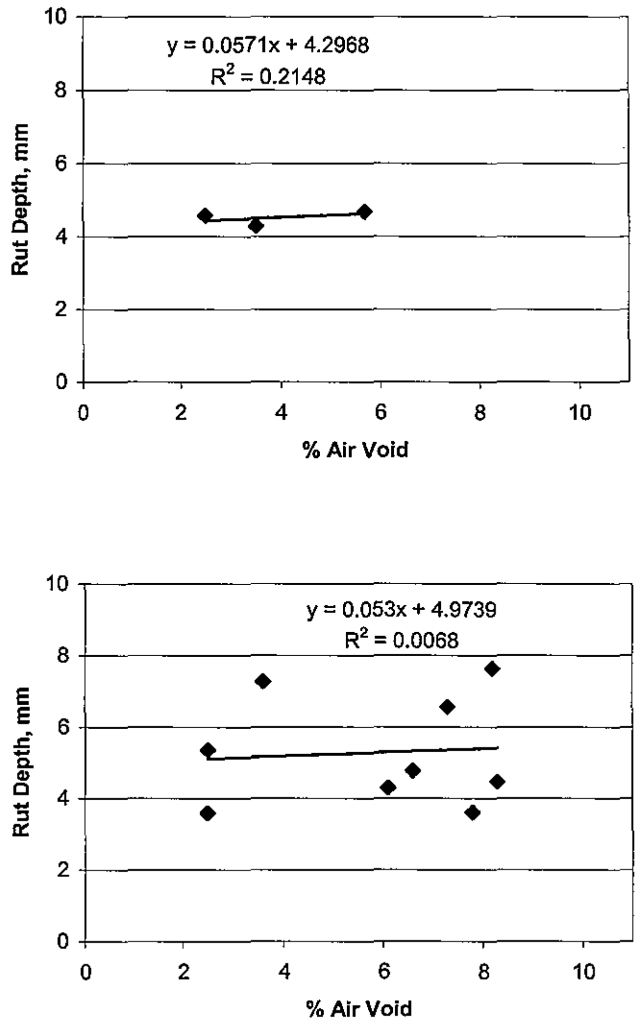


Figure 3.7 Correlation of Rut with Air Void

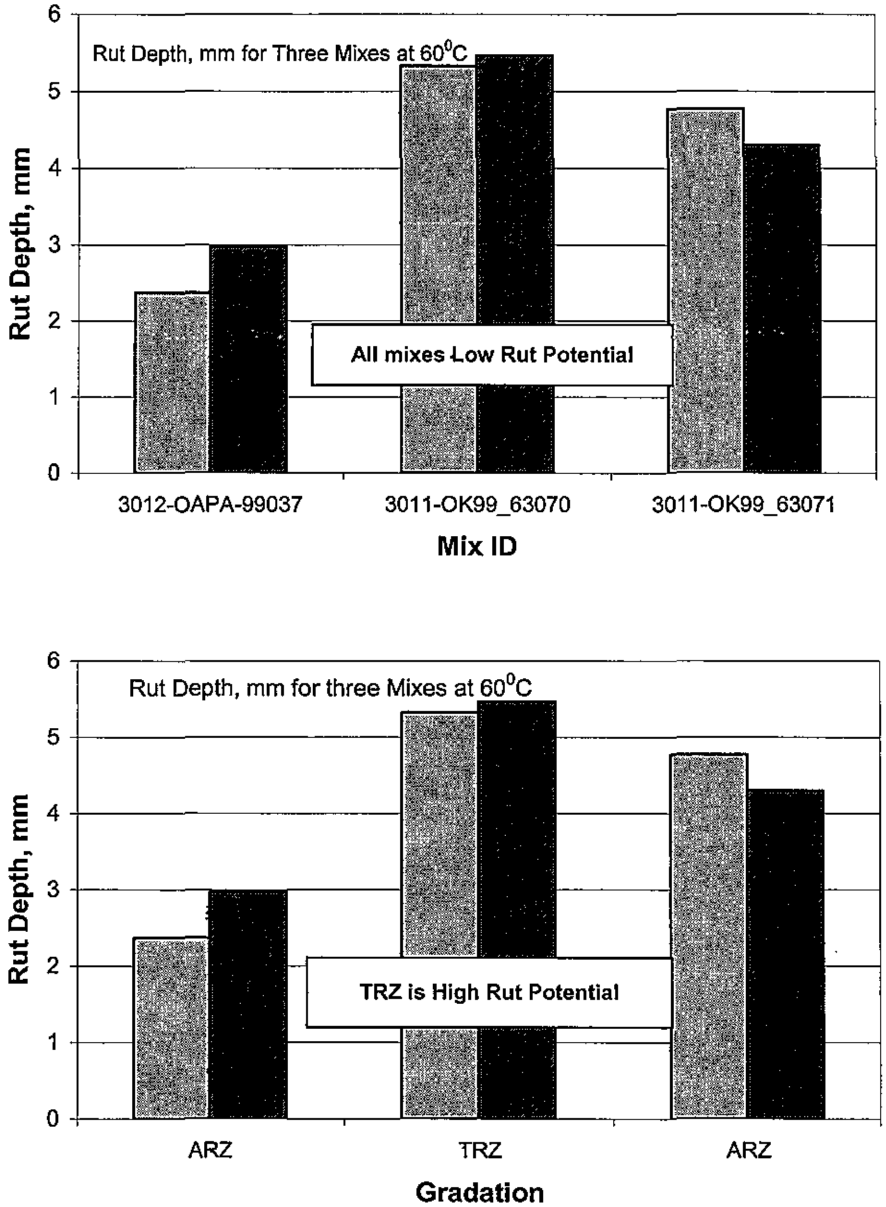


Figure 3.8 Comparison of Rut Potential of Exploratory and Base Mixes

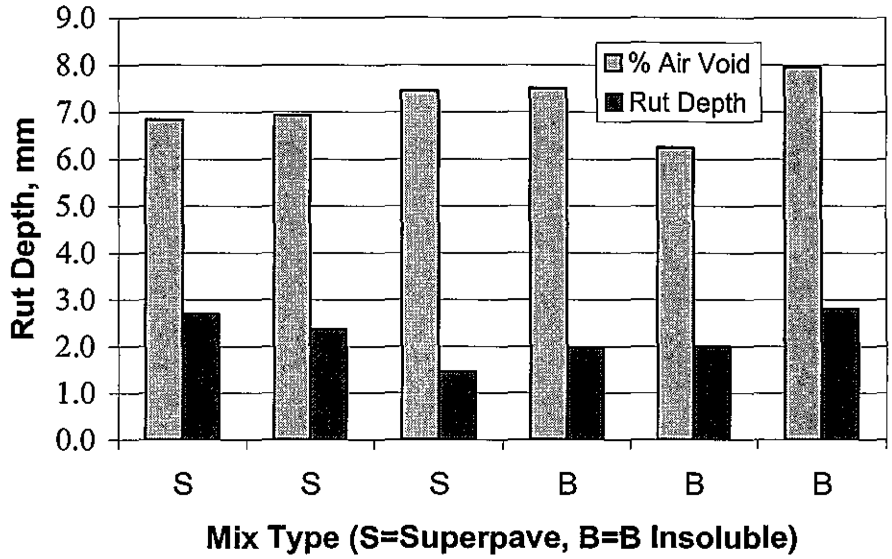


Figure 3.9 Rut Depths of Test Track Mixes

CHAPTER 4

PLANT MIX EVALUATION

4.1 General

The rutting potential of hot mix asphalt samples can be evaluated in the laboratory during the design phase of a project using an asphalt pavement analyzer. The APA test results can be used to rank mix performance in the laboratory before costly surprises are encountered in the field (Brock et. al., 1999). This chapter deals with the rutting susceptibility of 10 selected HMA mixes that are commonly used in Oklahoma for pavement construction. The primary goal is to rank these mixes based on their rutting potential as indicated by the APA data. The objectives are to evaluate the rutting susceptibility of selected asphalt mixes based on the APA data, and to examine the pertinent mix parameters that lead to differential rutting potentials of HMA specimens.

4.2 Experimental Methodology

4.2.1 Mix Selection

A total of ten different HMA mixes were selected in cooperation with the Oklahoma Department of Transportation. An attempt was made to select mixes that are representative of these commonly used in the State. The identification of mix, project identification number, design identification number, construction site (county), highway, and average daily traffic for each HMA concrete is listed in Table 4.1. The selected mixes are Types A and Type B HMA.

Mix 1, Mix 5 and Mix 7 are Recycled or Milled Asphalt Pavements (RAP or MAP) whereas the other mixes are Type B except Mix 8, which is a Type C (ODOT 1999). Mix 2 was designed for less than three millions Equivalent Single Axle Loads (ESAL). Mix 1, Mix 3 and Mix 8 were designed for more than 0.3 million ESALs. All of other mixes were designed for more than 3.0 million ESALs.

4.2.2 Material Collection

Materials from each project were collected in sufficient amount for rut testing. Each sample consisted of four bags with approximately 14 to 20 kg (30 to 44 lbs) of HMA materials. Two to three beam samples were fabricated from each mix; each beam sample required 6 to 6.5 kg (13 to 14 lbs) of HMA, while six cylindrical samples were molded from each project, each sample requiring about 3 kg (6.5 lbs) of HMA mix. The extra materials were burned in the NCAT ignition oven to determine the asphalt content and aggregate gradation as well as other properties of the mix.

4.2.3 Specimen Preparation

HMA mixes were heated first in a Blue M oven for about two hours, with all other tools such as spatulas, spoons, bowls, and molds at 149^0 C (300^0 F). Cylindrical specimens required about 3 kg (6.5 lbs) of HMA mix, while beam samples required about 6 to 6.5 kg (13 to 14 lbs) of the mix. For cylindrical specimens, the Superpave Gyratory Compactor (SGC) was used for compaction. In the molding procedure, the cylindrical mold was filled with the heated HMA mix in three layers, each layer placed and speculated by spatula to make sure that the mix was placed homogenously in the mold, according to standards and specifications (AASHTO PP28-00).

Specimens were compacted to the height of 3 in (75 mm) to achieve the target air void of 7.0 ± 1.0%. For beam specimens, the AVC was used for compaction with 700 kPa (100psi) of forward pressure and 245 kPa (35 psi) of back pressure for 35 second to achieve the target air void of 7.0 ± 1.0%. Compacted specimens were left at room temperature (approximately 25^0C or 77^0F) to allow the entire specimen to cool for ten hours.

The bulk specific gravity of compacted specimens was determined (AASHTO T 166). The maximum specific gravity (G_{mm}) for all HMA mixes was determined (AASHTO T 209). The percent air voids was calculated for each specimen, and then the specimens were arranged, and categorized according to their percent voids before the rutting test was started (AASHTO T 269). A total of 54 cylindrical specimens and 14 beam specimens were prepared and tested for rutting susceptibility using the APA.

4.2.4 APA Rut Test

A typical test uses either a three-beam specimens, each 75 mm x 125 mm x 300 mm (3 in x 5 in x 12 in) or six-cylindrical specimens, each 150 mm diameter x 75 mm (6 in x 3 in). Specimens were preconditioned at testing temperature of 64^0 C for a minimum of 10 hours. The test temperature was representative of Oklahoma's environment in which the paving mix will be utilized in the field.

The preconditioned modeled specimens were tested in the APA. According to the APA testing protocol. The vertical wheel load was kept at 445 N (100 lbs), and the pressure was adjusted to a pressure of 700 kPa (100 psi). The APA was run for 8000 load cycles. The rut depth was measured as a function of load the cycles.

Figure 4.1 shows a typical plot of rut depth versus load cycles prepared for Mix 6 from the APA data. It can be observed that the cylindrical specimens exhibited a rapid change in rut depth for the first 1000 cycles; as the number of cycles increased, the rut depth increased with a decreasing rate of rut. The cylindrical specimens for Mix 6 showed a maximum rut depth of 2.1 mm (0.082 inch). However, beam specimens of the same mix exhibited a total rut depth less than 3.0 mm (0.12 inch). A straight-line relationship between the rut depth and the number of cycle was established. Beam specimens, when compared to the cylindrical specimens, exhibited low rut depth for the first 1000 cycles, then changed sharply; eventually reaching higher rut depths at 8000 loading cycles.

4.3 Mixture Analysis

Each mix was burned for asphalt content using the National Center for Asphalt Technology (NCAT) ignition oven. Aggregate gradation based on sieve analysis was performed (AASHTO T 27). The proportions of the aggregate used in HMA mixes are listed in Table 4.2. Typically, three to four aggregates of different gradations are blended to achieve certain desirable gradation required for HMA mixes. Table 4.2 also shows that Mix 1, Mix 5 and Mix 7 have used 37mm (1½ inch) rocks; therefore, the nominal maximum size is 25.4mm (1 inch). The gradation information for all mixes is listed in Table 4.3.

The blend gradations for 3 mixes are plotted in Figure 4.2. The plots represent the gradation by percent passing versus the sieve size raised to the 0.45 power. It can be seen that Mix 2 passes below the restricted zone, whereas Mix 3 is above the restricted zone and Mix 8 passes through the restricted zone. The purpose of the restricted zone is to control the percent natural sand in a typical HMA mix. The binder's Performance Grade (PG), aggregate properties and mix volumetric properties are listed in Table 4.4. Asphalt cement Performance Grade (PG) PG 64-22 was used for Mix 1, Mix 2, Mix 3, and Mix 8. Mix 6 used PG 76-28. Asphalt cement PG 70-28 was used for the other mixes. The percentage of asphalt cement used in the design mix varied from 4.4% to 6.3%.

4.4 Mix Ranking

Figure 4.3 is a histogram showing all mixes with increasing rut values for cylindrical samples. Mixes have been labeled E (Excellent), G (Good), F (Fair) and P (Poor) on the basis of rut value in millimeter. Four mixes exhibited rut values below 2 mm (0.079 inch) and are labeled as excellent. Three mixes exhibited rut depth more than 2 mm (0.079 inch) and less than 3 mm (0.118 inch) and are classified as good. Mixes with rut potential of 3 mm to 4 mm (0.118 inch to 0.16 inch) have been characterized as fair mixes. Mix 3 showed a rut depth of more 4 mm (0.16 inch) and is classified as a poor. Figure 4.4 is a histogram which ranks the mixes based on beam specimen's rut values. For all cases, beam specimens rutted more than the cylindrical specimens. The ranking criteria for beam samples were fixed by increasing the rut depth criteria of cylindrical samples by 1 mm. Therefore, it can be seen that 2 mixes are excellent, one is good and others are poor performing mixes of the seven mixes. It can be seen that Mix 3 is poor performing in both cases. Some of the excellent performing mixes, when tested as cylinders, showed poor performance when tested as beams. Achieving target air void values for beam samples is tedious. Beam specimens show high variability in ruts for two identical samples.

4.5 Rut Parameter Interpretation

The APA data were analyzed carefully to establish any correlations between rutting and other parameters. Specifically, compaction method and sample geometry, mix type, aggregate size, asphalt content, binder grade, dust content, aggregate gradation and air void on the rutting susceptibility were evaluated.

4.5.1 Asphalt Concrete Type

Figure 4.5 shows rut depth versus asphalt mix type for the cylindrical samples. Three of the ten mixes used in this study are Type A (RAP) mixes, six mixes are Type B insoluble and one is a C insoluble. Type A mixes exhibited a mean rut of about 2.3 mm (0.09 inch) with a standard deviation of 0.45, while the Type B mixes exhibit a mean rut depth of 2.5 mm (0.098 inch) with a standard deviation of 1.1. Type C mix exhibited rut depth of 3.2 mm (0.12 inch). This is because the Type A mixes combine larger aggregates (nominal maximum size of aggregate 19.0 mm) compared to the Type B mixes (nominal maximum size of aggregate 12.5 mm) or the Type C mixes (nominal maximum size of aggregate 9.5 mm). The coarse aggregate provides the shear strength to resist rutting where as the fines are used to fill the voids in coarse aggregates.

4.5.2 Asphalt Content and PG

It can be seen from Table 4.5 that for Type A mixes, mix 7 with a percent asphalt content of 4.1 of PG 70-28 had the lowest rut depth, where as Mix 1 with a percent asphalt content of 4.6 of PG 64-22 had the highest rut depth of 2.8 mm. By comparing Mix 7 with Mix 5, it can be seen that the higher asphalt content of Mix 5 had lower rut depth than the lower asphalt content Mix 7. Therefore, the coarse mix, larger nominal maximum size (19.0 mm) was more sensitive to binder's performance grade as well as percent asphalt content. For Type M mix, asphalt content is not a sensitive parameter.

4.5.3 Materials Passing No. 200 Sieve

Table 4.5 also shows that the maximum rut depth for the Type B mixes is 4.3 mm with a minimum of 1.4 mm. The rut depth for type B mixes increases (Mix 3 and Mix 9 show higher ruts compared to other B mixes) as the percent passing # 200 sieve increased. Mix 2 and Mix 4 had less materials passing No. 200 sieve (4.2 and 4.7 percent, respectively) as compared to Mix 9 and Mix 3 (5.4 and 5.7 percent respectively). Mix 2 and Mix 4 have less rut value compared to Mix 9 and Mix 3. Therefore, the mixes with smaller nominal maximum size (12.5 mm) are more sensitive than materials passing No. 200 sieve.

4.5.4 Gradation

Mix gradations passing Below the Restricted Zone (BRZ) are coarser than that of mixes passing Above the Restricted Zone (ARZ). Table 4.5 shows that ARZ mixes have higher rut values compared to the BRZ and TRZ mixes. Again, TRZ mixes have higher rut depths compared to the BRZ. The same is very clear when comparing the Type B insoluble mixes of different gradations. For example, Mix 2 with BRZ had the lowest rut depth (1.4mm) compared to the TRZ and ARX mixes. Mix 4 with TRZ had the second lowest rut when comparing the rut values of the Type B mixes. It is clear from Table 4.5 that aggregate gradations, which pass through the restricted zone, are not susceptible to rutting.

4.5.5 Dust to Asphalt Ratio

The relationship between the dust-asphalt cement content ratio and the rut depth for cylindrical samples are shown in a scatter plot from Figure 4.6. The plot shows that there was no effect of dust-asphalt content ratio on the rut depth for cylindrical samples.

4.5.6 Sand

Figure 4.7 shows the relationship between percent sand and final rut depths of mixes. In general, as the percent of natural sand increases, the rut depths increase. It appears that fair and poor ranked mixes such as Mix 8, Mix 9 and Mix 3 have higher percentage of sand (15 percent sand). However, Mix 4 and Mix 6 have shown low rut potential although these mixes have 15 percent sand. Figure 4.7 also shows that there exists a better correlation between rut depth and percent passing Sieve no. 80.

4.5.6 Compaction and Sample Geometry

The final rut depths for both beam and cylindrical samples were compared in Figures 4.8. For low rut potential mixes (except Mix 2), there was no significant difference in rutting of the AVC and the SGC samples. However, for the high rut potential mixes (Mix 3, Mix 9, Mix 1), the AVC beam specimens yielded collectively higher rutting in the APA. One of the potential reasons for the differences was the molds' configuration. Cylindrical molds accommodate two cylindrical specimens and contain a spacer between the two specimens. The spacer is approximately 12 mm (4.7 inch) below the testing surface of specimens. Therefore, for low rut potential mixes, spacer would not influence rut depths in cylindrical specimens. However, for high rut potential mixes, the spacer might impede the downward movement of pressurized linear hose.

The bridging action of the spacer might be the reason why the cylindrical specimens rut less than the beam specimens. The second potential reason was the density gradient within the specimens. Because the cylinders and beams were compacted using different modes of compaction (one is gyration and another is vibration), a contrasting density gradient may exist in the two specimen types. Previous findings from other studies also confirmed that gradients in density do occur in a beam specimen compacted with the AVC and cylindrical specimens compacted with the SGC (Cooley et al. 1999).

4.5.7 Air Voids

The histogram plotting in Figure 4.9 shows that rut depths do not vary significantly where the air voids are between 6% and 7%. This is true for cylindrical specimens. No clear relationship between the percent air voids and rut depth for cylindrical samples was evident.

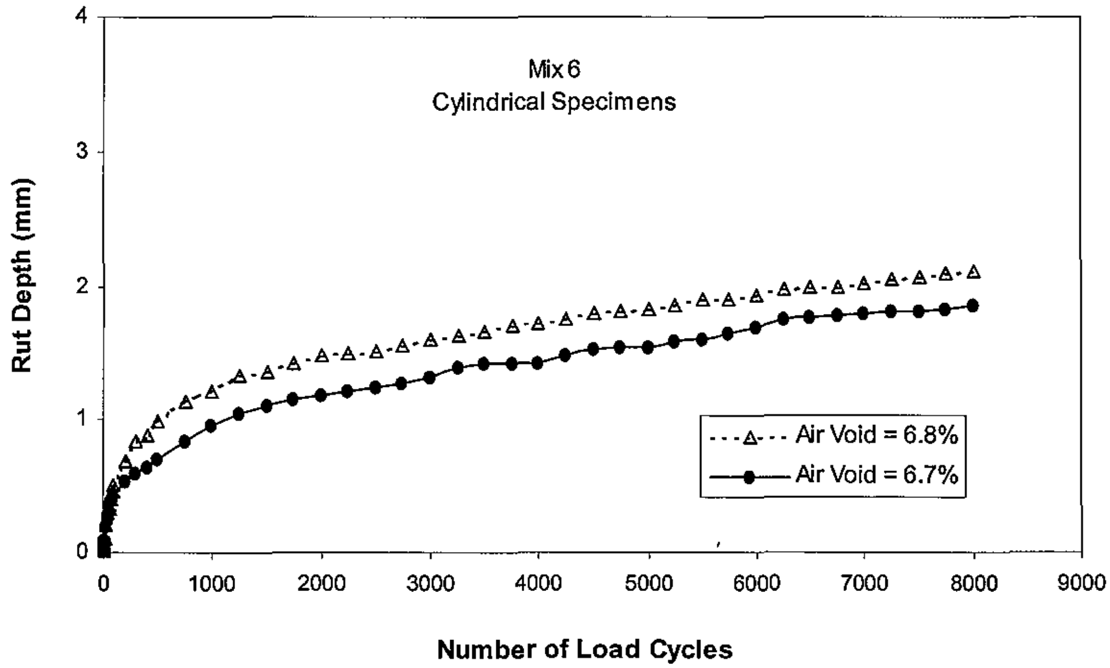
4.5.8 OU Versus ODOT Data

The ODOT's Materials Division conducted rut tests using their APA on 10 plant-produced mixes. The data from these tests were collected, compiled in an organized manner, and compared (Table 4.6a and Table 4.6) with the corresponding data obtained by the OU Team. Only cylindrical samples were compared. The data were sorted to separate bad data. From the table it can be seen that OU had 12 bad data samples (air void was not between 6-8%) out of 60 cylindrical samples and 8 bad data samples out of 16 beam samples. Therefore, the efficiency of testing for cylindrical specimens were 80% where as the efficiency for beam specimens were 50%. It was also found that there is a general difficulty in consistency of beam sample preparation and testing. The OU rut data were graphically compared with the ODOT data as in Figure 4.10. It can be seen that there was no significant difference in measured rut depths for the same mix.

4.6 Summary

Ten different asphalt mixes were selected from different sites in Oklahoma. A total of 54 cylindrical specimens and 14 beam specimens have been tested to determine their rut depth using the APA. The APA gave different values of rut depth for both beam and cylindrical specimens. It was observed, that the APA was sensitive to mix parameter and is a reliable device to be used in the laboratory to measure rut depth for fabricated samples. One of the most advantages of the APA is that it gives immediate results with fewer errors after initial adjustment.

Mixes were ranked based on the rut potential of cylindrical specimens. Only one mix showed poor performance i.e. rut depth more than 4mm. Mix analysis showed that fine mixes were more sensitive to materials passing No. 200 sieve, where as the coarse mixes were sensitive to aggregate size. The AVC specimens rut more than the SGC specimens. The dust to asphalt ratio and percent air void had insignificant effect on rut depth because they were in controlled range. Because of the difference in layer thickness, underlying support, confirming pressure, stream distribution, and among other factors, the results of rut tests from a laboratory APA rut tester will be different from actual rut depths in pavement. However, to recommend a specific rut depth for acceptance or rejection of HMA, there is a need to correlate the results from the APA test and actual rut depths in pavements.



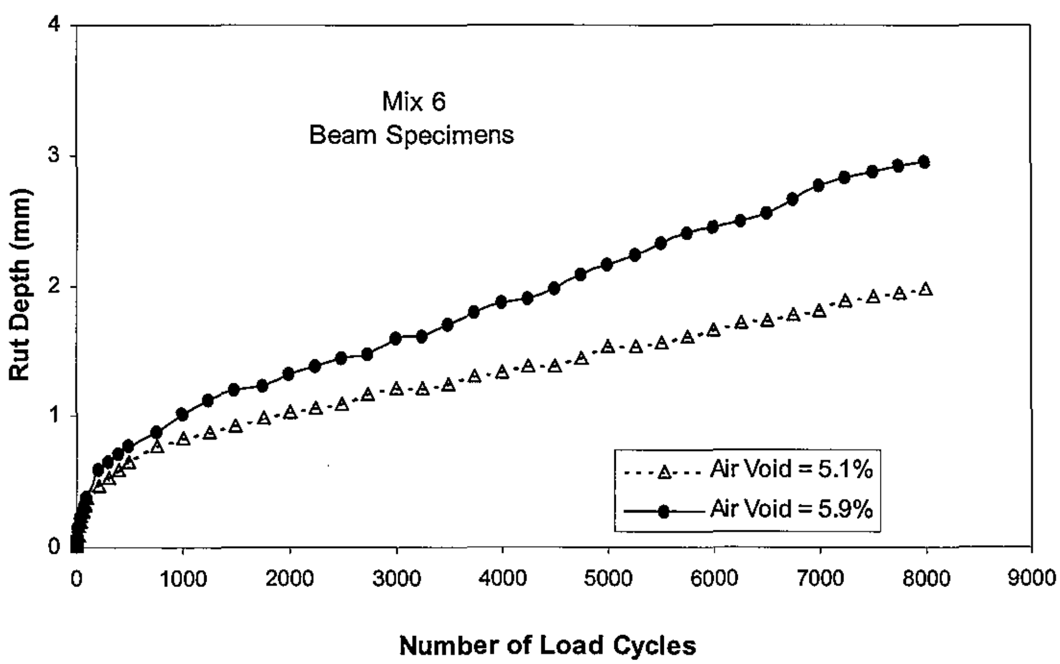


Figure 4.1 Typical Rut Depth versus Loading Cycle Plot by the APA

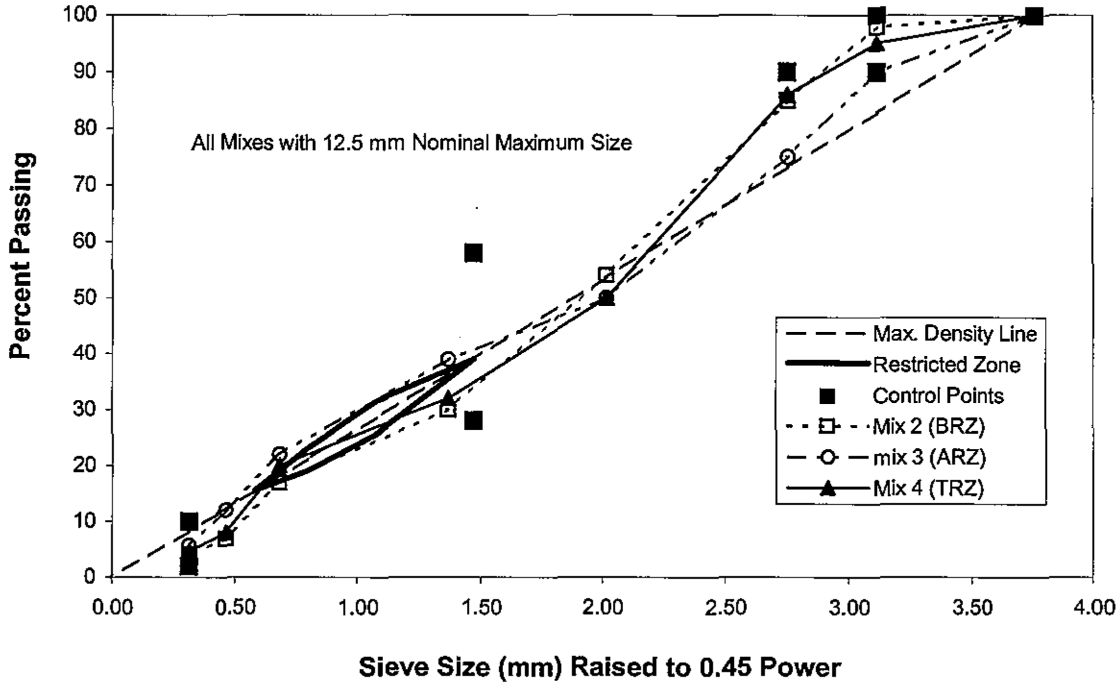


Figure 4.2 Characteristics of Mixes on the 0.45 Power Gradation Chart

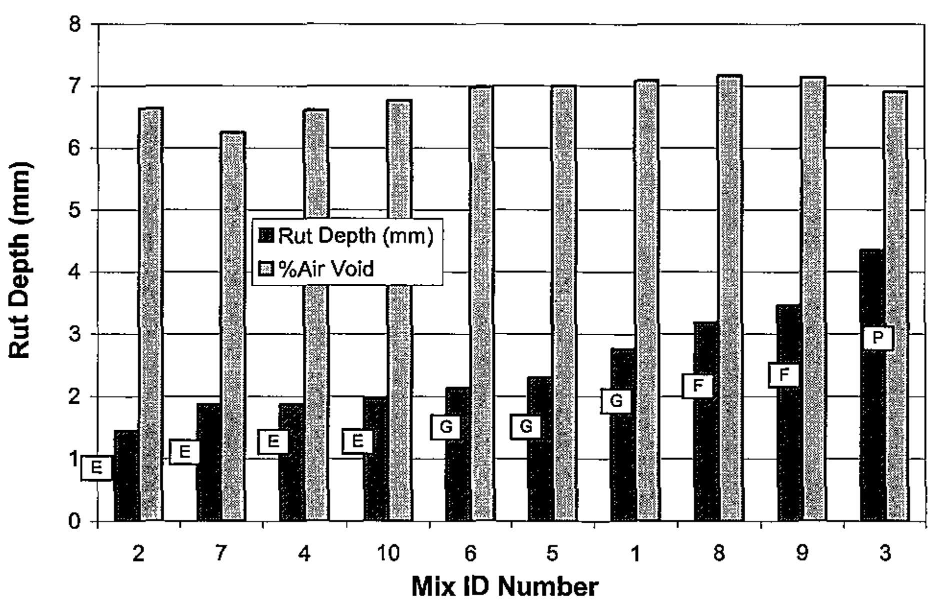


Figure 4.3 Mix Ranking Based on Rutting Potential of Beam Specimen

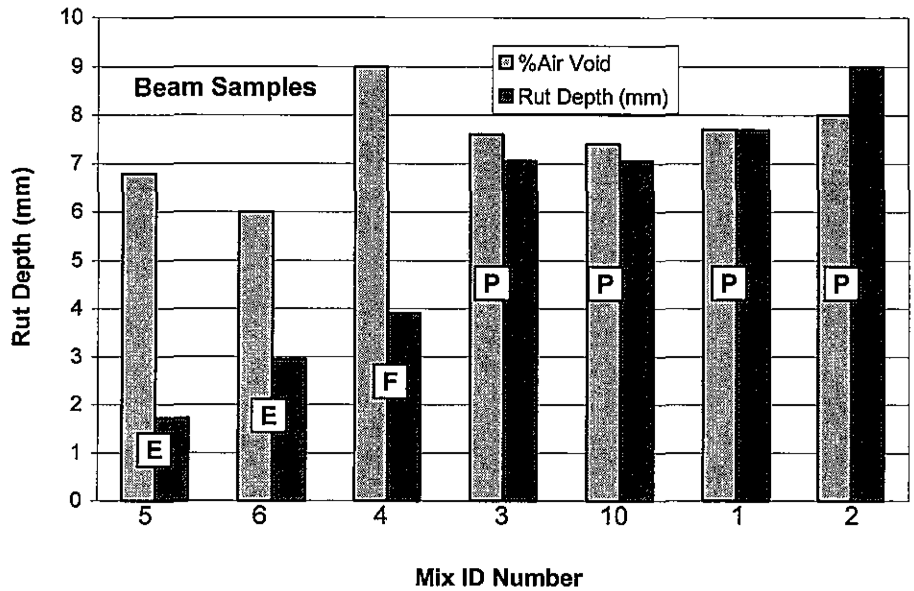


Figure 4.4 Mix Ranking Based on Rutting Potential of Beam Specimen

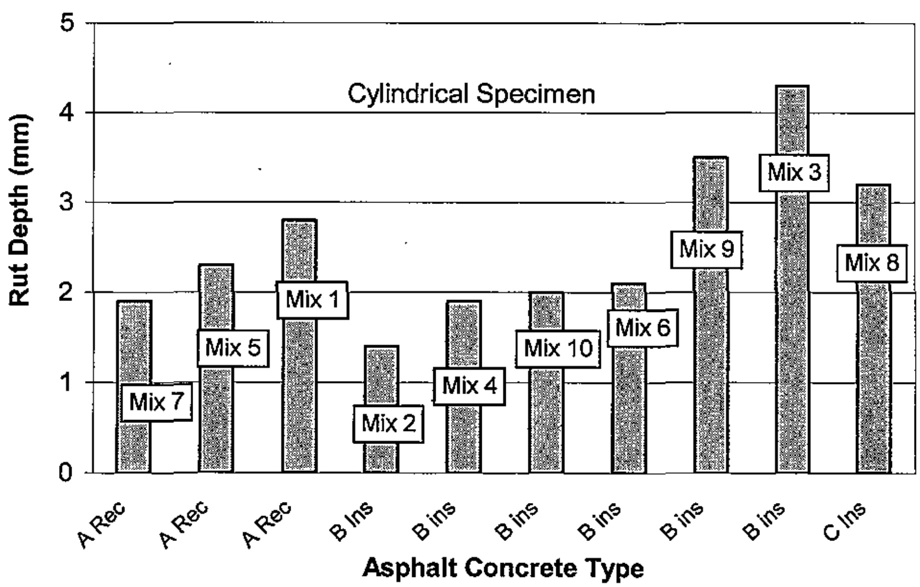


Figure 4.5 Effect of Asphalt Concrete Type on Rut Depth

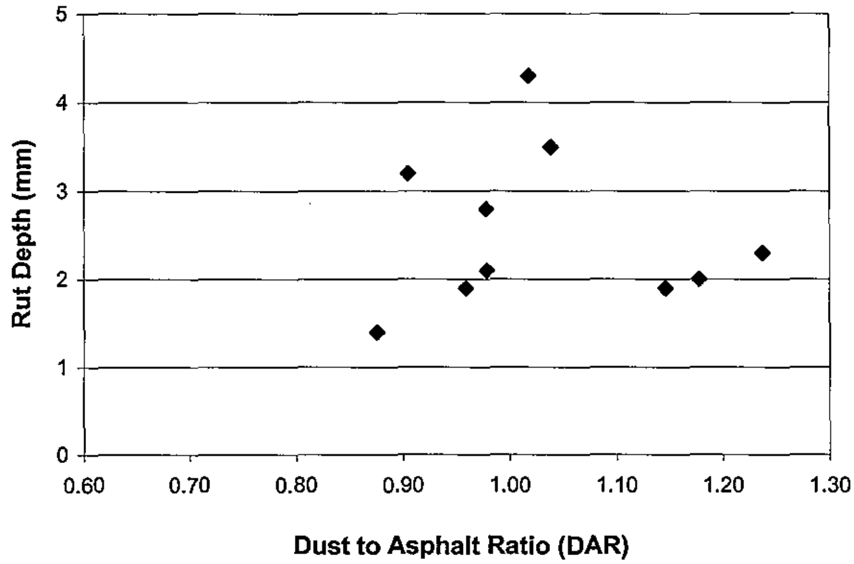
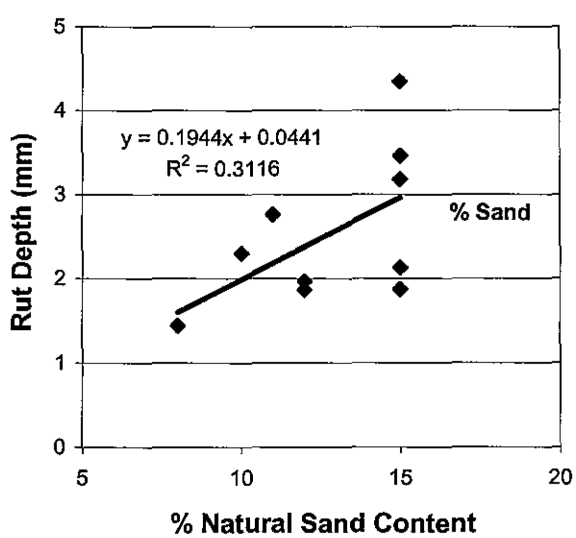


Figure 4.6 Effect of DAR on Rutting



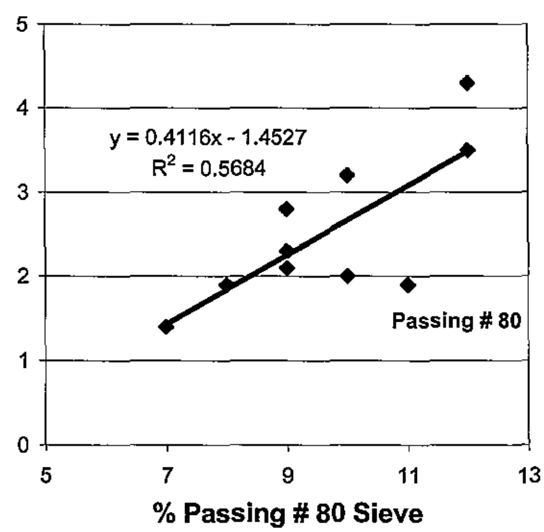


Figure 4.7 Effects of Sand and Passing # 80 Sieve Materials on Rutting

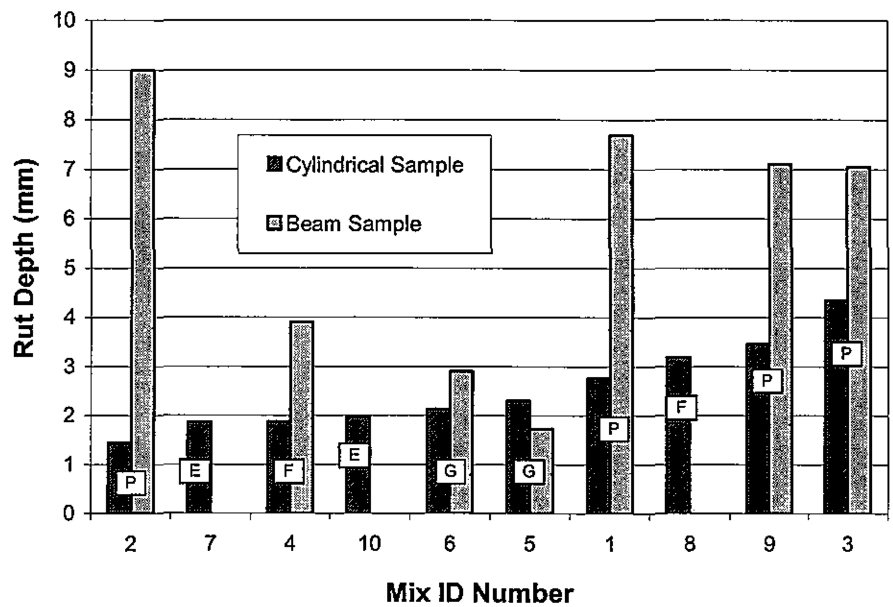


Figure 4.8 Mix Ranking Considering Both Beam and Cylindrical Specimen

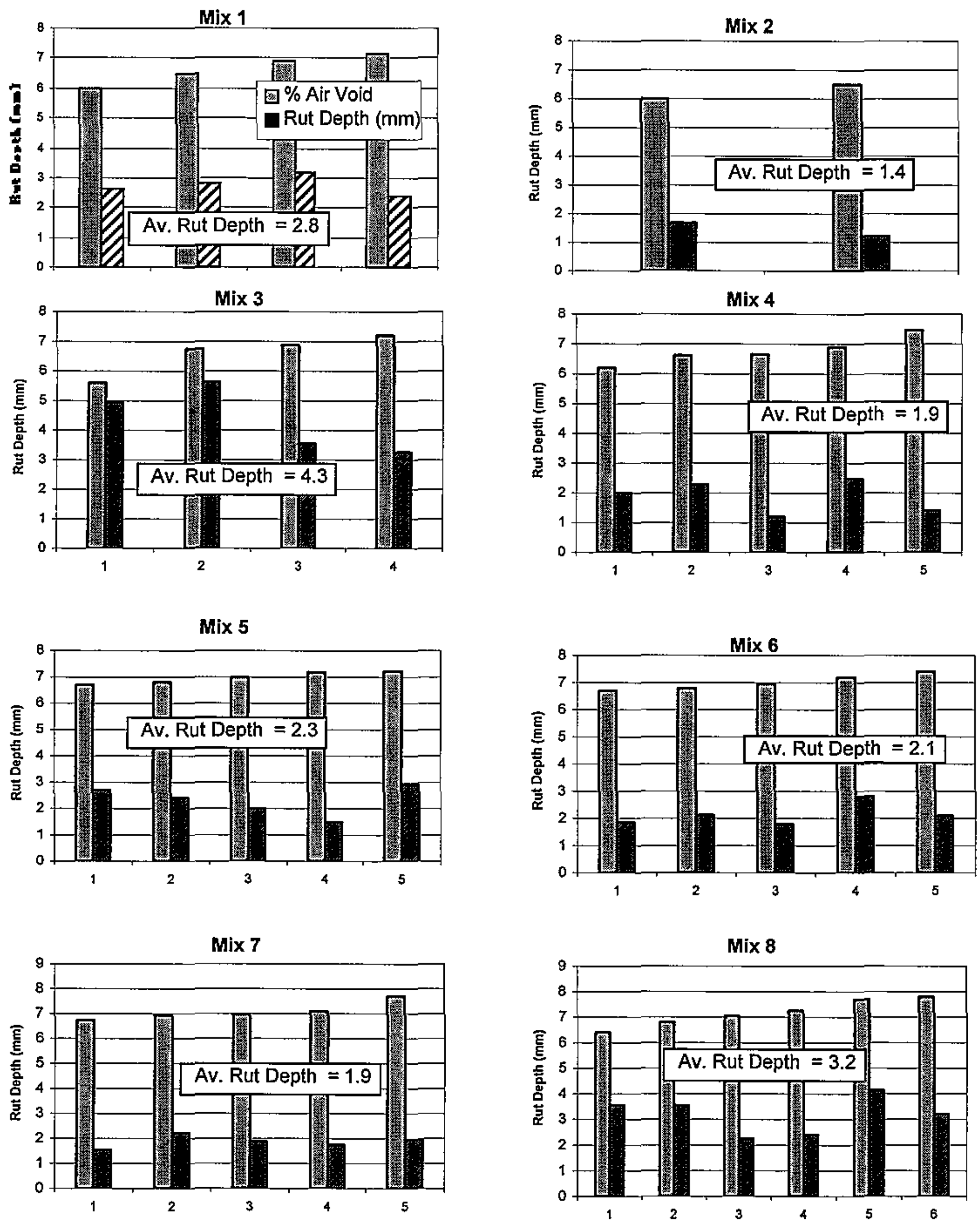


Figure 4.9 Effect of Air Void on Rut Depth for Cylindrical Samples

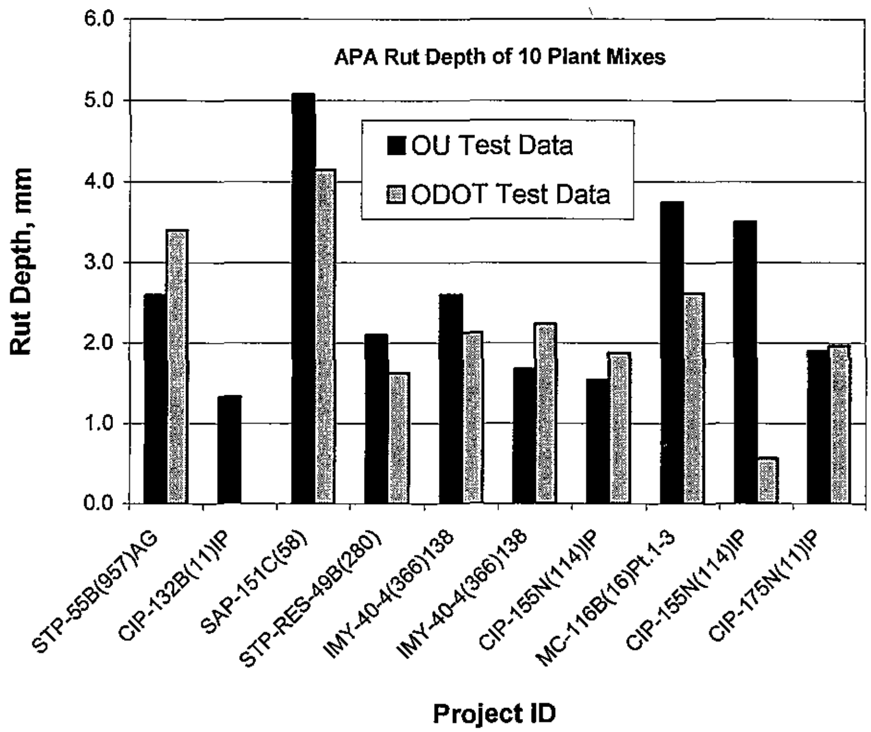


Figure 4.10 Comparison of OU Rut Data with ODOT Rut Data

Table 4.1 Mix and Traffic Information

Mix ID	Project ID	Design ID	County	Highway	AC Type	A.D.T
1	STP-55B(957)AG	3011-56875	Oklahoma	City Street	A Rec	0.3M+
2	CIP-132B(11)IP	3012-OAPA-99048	Hughes	US75	B Ins	0.3M+
3	SAP-151C(58)	3012-OAPA-20095	Muskogee	Lake Road	B ins	0.3M+
4	STP-RES-49B(280)	3012-APAC-99018	Mayes	SH-20	B ins	3M+
5	IMY-40-4(366)138	3011-OAPA-20048	Canadian	I40	A Rec	3M+
6	IMY-40-4(366)138	3012-OAPA-20049	Canadian	I40	B ins	3M+
7	CIP-155N(114)IP	3011-OAPA-20090	Oklahoma	City Street	A Rec	3M+
8	MC-116B(16)Pt.1-3	3013-OAPA-20225	Cimarron	City Street	C Ins	0.3M+
9	CIP-155N(114)IP	3012-OAPA-20095	Oklahoma	City Street	B ins	3M+
10	CIP-175N(11)IP	3012-OAPA-20033	Oklahoma	US183	B ins	3M+

AC= Asphalt Concrete; A.D.T = Average Daily Traffic; Rec= Recycled; Ins= Insoluble

Table 4.2 Types of Aggregate

Mix ID	1-1/2" Rock	3/4" Chips	5/8" Chips	5/8" Mill Run	3/8" Scre-enings	1/4" Chips	Shot	Stone Sand	Chat	No.4 Scre-ening	Scre-ening	MAP	Sand
1	22						20				22	25	11
2			30	34		28							8
3		17	35								33		15
4		26							36		23		15
5	39				13			15				23	10
6			42		18			25					15
7	24						18				21	25	12
8		25	30								30		15
9			28					10			47		15
10		12	30							26	20		12

Table 4.3 Mix Aggregate Gradations

Sieve Size (mm)	1 1/2"	1"	3/4"	1/2"	3/8"	# 4	# 10	# 40	# 80	# 200
Mix ID	(37.5)	(25.4)	(19.5)	(12.5)	(9.5)	(4.75)	(2.0)	(0.425)	(0.18)	(0.075)
1	100	99		84		60	35	20	9	4.5
2			100	98	85	54	30	17	7	4.2
3			100	90	75	50	37	22	12	5.7
4			100	95	86	50	32	20	8	4.7
5	100	98		76		54	40	20	9	4.7
6			100	99	86	60	45	22	9	4.6
7	100	99		82		61	36	23	11	4.7
8				100	95	66	44	18	10	5.7
9			100	99	89	62	44	25	12	5.4
10			100	89	73	57	40	20	10	5.3

Table 4.4 HMA Mix Properties

Mix ID	Binder Properties			Aggregate Properties						Mix Properties		
	PG	Source	Sp. Gr.	S. E.	L.A.	Durability	IOC	IR	FF	P_b	VMA	Hveem Stability
1	PG64-22OK	a	1.0100	70	23.5	69	0.22	87.4	100	4.6	13.7	41
2	PG64-22	d	1.0201	70	27.3	83	0.14	87.4	100	4.8	15.4	48
3	PG64-22OK	e	1.0119	56	34.7	58	1.04	90.0	100	5.6	15	49
4	PG70-28OK	c	1.0198	71	23.4	73	0.22	40.4	100	4.9	16	45
5	PG70-28OK	b	1.0100	77	23.2	73	0.10	87.4	100	3.8	13.7	59
6	PG76-28OK	b	1.0232	79	26.4	77	0.23	40.0	100	4.7	15.7	50
7	PG70-28OK	a	1.0100	62	20.7	72	0.22	79.3	100	4.1	14.5	62
8	PG64-22OK	f	0.9943	75	20.0	84	0.3	80.9	100	6.3	15.5	51
9	PG70-28OK	a	1.0128	59	20.9	77	0.78	70.5	100	5.2	17.2	59
10	PG70-28	c	1.0245	68	25.2	84	0.12	63.5	100	4.5	16.2	53

S.E = Sand Equivalent; L.A. =Los Angeles Abrasion; P_b = Percent Asphalt Content; IOC = Ignition Oven Calibration Factor; IR = Insoluble Residue; FF = Fractured Face; VMA = Void in Mineral Aggregate

Table 4.5 Effect of Asphalt Concrete Type

Mix ID	AC Type	Gradation	Nominal Maximum Size (mm)	% Passing No. 200 Sieve	% Asphalt Content	DAR	Rut Depth (mm)
7	A Rec	ARZ	19.0	4.7	4.1	1.15	1.9
5	A Rec	ARZ	19.0	4.7	3.8	1.24	2.3
1	A Rec	ARZ	19.0	4.5	4.6	0.98	2.8
2	B ins	BRZ	12.5	4.2	4.8	0.88	1.4
4	B ins	TRZ	12.5	4.7	4.9	0.96	1.9
10	B ins	ARZ	12.5	5.3	4.5	1.18	2.0
6	B ins	ARZ	12.5	4.6	4.7	0.98	2.1
9	B ins	ARZ	12.5	5.4	5.2	1.04	3.5
3	B ins	ARZ	12.5	5.7	5.6	1.02	4.3
8	C ins	TRZ	9.5	5.7	6.3	0.90	3.2

Note: Rec =Recycled aggregate, ins = insoluble aggregate, DAR = Dust to Asphalt Ratio

Table 4.6a Comparison of OU APA Data with ODOT APA Data

Mix1	% Air Void	Rut (mm)	**Mix6**	% Air Void	Rut (mm)	**Mix9**	% Air Void	Rut
OU	6.0	2.6	OU	3.9	1.1	OU	6.2	4.1
OU	6.5	2.8	OU	6.7	1.9	OU	7.5	2.5
OU	9.5	2.3	OU	7.4	2.1	OU	7.7	3.8
ODOT	6.9	3.2	ODOT	6.8	2.1	ODOT	6.9	0.7
ODOT	7.1	2.4	ODOT	6.9	1.8	ODOT	7.1	0.7
ODOT	7.2	4.7	ODOT	7.2	2.8	ODOT	8.1	0.3
Mix3	% Air Void	Rut (mm)	**Mix7**	% Air Void	Rut	**Mix2**	% Air Void	rut
OU	5.5	5.1	OU	7.1	1.7	OU	6.0	1.7
OU	5.5	5.1	OU	7.7	1.9	OU	6.5	1.2
OU	5.6	4.9	OU	7.8	0.9	OU	9.5	1.1
ODOT	6.8	5.6	ODOT	6.7	1.6	**Mix5**	% Air Void	rut
ODOT	6.9	3.5	ODOT	6.9	2.2	OU	6.7	2.7
ODOT	7.2	3.3	ODOT	7.0	1.9	OU	6.8	2.4
Mix4	% Air Void	Rut	**Mix8**	% Air Void	Rut	ODOT	7.0	2.0
OU	5.9	1.8	OU	6.4	3.6	ODOT	7.2	1.5
OU	6.2	2.0	OU	6.8	3.5	ODOT	7.2	2.9
OU	6.9	2.5	OU	7.7	4.1	**Mix10**	% Air Void	Rut
ODOT	6.6	2.3	ODOT	7.0	2.2	OU	2.7	1.1
ODOT	6.7	1.2	ODOT	7.3	2.4	OU	4.2	2.1
ODOT	7.5	1.4	ODOT	7.8	3.2	OU	4.6	2.6

Table 4.6b Comparison of OU Rut Data with ODOT Data

Plant Mixes	Mix Information		8000 Cycles Rut Depth, mm	
Mix No.	Project	AC Type	OU	ODOT
1	STP-55B(957)AG	A Recycled	2.6	3.4
2	CIP-132B(11)IP	B Insoluble	1.3	...
3	SAP-151C(58)	B insoluble	5.1	4.1
4	STP-RES-49B(280)	B insoluble	2.1	1.6
5	IMY-40-4(366)138	A Recycled	2.6	2.1
6	IMY-40-4(366)138	B insoluble	1.7	2.2
7	CIP-155N(114)IP	A Recycled	1.5	1.9
8	MC-116B(16)Pt.1-3	C Insoluble	3.7	2.6
9	CIP-155N(114)IP	B insoluble	3.5	0.6
10	CIP-175N(11)IP	BH insoluble	1.9	2.0

CHAPTER 5

BINDER EVOLUTION

5.1 Background

The concept of creating hot mix asphalt concrete with increased resistance to permanent deformation, or rutting was a major driving force behind much of the asphalt-related research performed under the Strategic Highway Research Program (SHRP). The provisional binder specification AASHTO MP1-98 (better known as the SHRP or the Superpave binder specification) represents a historic and logical steppingstone (AASHTO MP1-98, 2000) on the path to a performance-related specification for binders. In the 40's and 50's, the penetration grading system, ASTM D 946 was primarily used for specifying binders (ASTM D 946, 1998). The penetration value did not describe pavement distress, as it was not a fundamental property of a binder.

The next evolutionary step was the viscosity grading system, ASTM D 3381 (ASTM D 3381, 1998). The performance of pavements built with viscosity-graded asphalt binders were thought to be controlled by their viscosity-temperature susceptibility (Anderson et. al., 1991). Asphalt cements classified on the basis of viscosity did not adequately reflect the rheology of the binder. Viscosity does not give a true indication of how asphalt cement will perform within a pavement over its yearly temperature range. A binder can be non-Newtonian (and visco-elastic), therefore, it will require further characterization in addition to the viscosity.

In the late 80's and early 90's, a new specification, called Performance Based Asphalt (PBA), attempted to include regional climate variations and long-term aging in the field (Reese et al., 1993). The Superpave binder specification adopted many of the concepts in PBA specifications. The most significant advancement in the Superpave Binder (SB) specification was the move from empirical testing to advanced performance based testing. With Superpave specifications, a binder can be characterized at a controlled rate and temperature to obtain engineering the properties of that binder. In the Superpave binder specification, the Dynamic Shear Rheometer (DSR), Bending Beam Rheometer (BBR) and Direct Tension (DT) replaced such tests as the viscosity, penetration and ductility testing. Nine-binder grade-classifications are used under the asphalt grading system (AASHTO TP5-98, AASHTO TP1-98, AASHTO TP3-00).

The Oklahoma Department of Transportation (ODOT) adopted the PG (Performance Graded) binder specification in July 1997. The ODOT supplemented the AASHTO MP1 (AASHTO MP1-98, 2000) specifications in 1999 (ODOT, 1999). The new grading system, AASHTO MP1 (AASHTO MP1-98, 2000) more appropriately relates the grade of the asphalt binder to the pavement temperature and traffic loading for a construction project than the previous grading systems. Under a true PG grading system, binders classified the same should have similar performance characteristics. Mixes containing these binders should show similar performance characteristics. PG binders of the same grade, produced from different crudes and manufacturing process, and meeting the specification requirements of MP1-98, may show different performance in HMA mixes (Natu et al., 1999). If different binders of the same PG grade do not perform similarly, then the binder specification may lose its significance. It should be noted that the PG system was a purchase specification. A real attempt was made by the SHRP researchers to relate the various PG grades to actual performance. No binder grading system may fully identify the full mixture performance when binder characteristics alone were considered.

Rutting and fatigue failure models were developed during the SHRP research. These models continue to be refined. The Superpave Shear Test (SST) (AASHTO TP9, 2000) and Indirect Tensile Test (IDT) (AASHTO TP7, 2000) machines were expensive. Only five Superpave centers had these machines in the early 1990's. The cost of these machines makes full use of the SHRP research using the SST and IDT cost and time prohibitive. Full implementation of Superpave, by state and local agencies, using theses machines may be delayed.

The ODOT and the University of Oklahoma purchased the APA and Asphalt Vibratory Compactors (AVC) in 1999. An Oklahoma HMA contractor purchased an APA in 2001 and some contractors have used the APA to determine rutting potential independent of ODOT.

Superpave testing equipment and procedures, for a full evaluation of the permanent deformation resistance for a given mixture, are still under development. Recently, the APA has become increasingly popular in evaluating rutting potential of HMA mixes (Kandhal et al., 1999). Accordingly, many state agencies have started using the APA to evaluate rutting potential. The present study has employed THE APA to investigate the performance of different binders based on HMA rut potential. The main objective of the study is to evaluate and compare the performance of these binders in the context of rut potential of mixes with these binders. A subsequent objective was to examine the performance of binders with the same high temperature PG grade (unmodified binders or modified binders) and the performances of binders with different high temperature PG grade (comparison modified and unmodified binders). The primary goals of this study were to develop rutting prediction equations of HMA mixes and to examine whether MP1-98 specified binders could produce a low rut potential mix.

5.2 Binders Description

This section described thirteen different unmodified and modified binders from different sources and PG grades HMA. These binders were currently being used in different projects within Oklahoma. The unmodified binders referred to as PG1 were PG 64-22 or PG 64-22 OK and they were refined from eight different sources. These binders were produced from crude oil that was high in asphaltenes. These are known as base asphalt. The modified binders PG2 were PG 70-28 and PG 70-28 OK, typically contains 2% styrene-butadiene-styrene (SB) polymer. These two binders used in samples of this study were obtained from two different sources. The modified binder PG3 was a PG 76-28 OK from one of the PG2 sources. It typically contains 5% SB polymer with 0.05% chemical anti-strip additive. The modified binders were produced from the same base asphalts but contain relatively low amount of asphaltenes. The PG 64-22 OK, PG 70-28 OK and PG 76-28 all meet the requirements for PG 64-22, PG 70-28 and PG 76-28 in accordance with AASHTO MP1, as well as the additional requirements of ODOT specification (ODOT, 1999).

5.3 Binders Properties

Tests were conducted to determine G^* and δ values using a Dynamic Shear Rheometer (DSR) at the high PG temperature and at 10 radian/sec frequency of loading. The DSR tests were performed on the original and Rolling Thin Film Oven (RTFO) samples. The Superpave binder specification uses a factor called rutting factor, $G^*/\sin\delta$ to characterize binder stiffness or rut resistance at high pavement service temperature. The rutting factor reflects the total resistance of a binder to deform under repeated loading (G^*), and the relative energy dissipated into non-recoverable deformation ($\sin\delta$) during the loading cycle (Roberts et al., 1996). A higher value of $G^*/\sin\delta$ implies that the binder behaves more like an elastic material, which was desirable for rutting resistance. As the binder ages, the G^* increases and the δ decreases and binders become less viscous. The SHRP rutting factor $G^*/\sin\delta$ for unaged and aged binders was listed in Table 1.

From Table 1, it can be seen that all binders were within the Superpave specification for the rutting factor, $G^*/\sin\delta$. The value of $G^*/\sin\delta$ is a minimum of 1.00 kPa and 2.20 kPa for unaged, and RTFO aged binders, respectively. The mean rutting factor for the unmodified binder was 1.40, where as for the modified binders the corresponding value was 1.57 at unaged condition. The mean rutting factor for unmodified binder of 3.3 and for the modified binder of 3.10 indicates there was not a significant improvement of rutting factor due to modification. The rutting factor can be compared at the same temperature assuming linear behavior. For example, rutting factors for modified binder (i.e. PG2) of 3.10 at 70 °C would be 6.2 at 64 °C. Therefore, all the modified binders have high rutting factors when compared with unmodified binder at 64 °C. A study by Bahi et al., (Bahia et al., 1999) showed that polymer modification increases the elastic responses and dynamic modulus of bitumen at intermediate and high temperatures, and influence complex and stiffness modulus at high temperature. Polymer can reduces the temperature susceptibility, the glass transition and limiting stiffness temperatures of bitumen (Bahia et al., 1999).

The binders were also tested for viscosity at 135^0 C using a rotational viscometer (AASHTO TP48-97) and the values were listed in Table 5.1. Although the test was usually conducted for mixing and handling performance, this study has attempted to correlate viscosity with rutting performance. The higher viscosity values for modified binders, as shown in Table 5.1, indicates that polymer modification makes binders more resistance to disturbance. Table 5.1 also shows that the viscosity was different for various modified binders depending on the source. The degree of improvement in binder quality generally increases with polymer content, but varies with base bitumen, bitumen source, PG grade and polymer type (Isacsson, 1999)

5.4 Aggregate and Mix Design

Four mineral aggregates consisting 5/8" chips, screenings, shot and sand were incorporated into the Superpave method of mix design to produce asphalt concrete. Aggregate information was listed in Table 5.2. In the experimental procedure one, aggregates were evaluated, and gradation tests were performed to obtain a blend that met all of the Superpave gradation criteria. The final blend gradation plotted on the 0.45 power chart, as shown in Figure 5.1, passes below the maximum density line with a Nominal Maximum Size (NMS) of 12.5 mm. The blended aggregate properties were summarized in Table 5.3. Mix designs were performed using a traffic level of more than

3 and less than 30 million Equivalent Single Axle Loads (ESALs). Although the binder grades of PG 64-22 and PG 64-22 OK were recommended for less than 3 million ESALs in ODOT specification, this study used 3 million ESALs as the design criteria for volumetric properties.

The maximum gyration, N_{max} was 160 and the design gyration, N_{design} was 100. Design mixes were mixed at 163^0 C, aged at 149^0 C for 3 hours and compacted at 149^0C using a Superpave Gyratory Compactor (SGC). The SGC was set at 600 kPa load and 1.25^0 gyratory angle. The optimum asphalt content was determined at 4% air voids at N_{design}. Figure 5.2 and Table 5.4 represents typical examples optimum asphalt content of four binders and volumetric properties as well as Superpave volumetric criteria. After each mix design was completed, the mix was tested for water susceptibility (AASHTO T 283). Only mixes with a Tensile Strength Ratio (TSR) more than 0.80 were used in the final mix design. In addition, some binders were mixed at lower and higher optimum asphalt contents to examine the effect of asphalt binder on rutting performance of mixes.

5.5 Rut Testing

Cylindrical specimens of 75 mm height were compacted in the SGC at a target air void of 6 to 8%. Specimens were preconditioned at 64^0 C for 10 hours before rut testing. In the APA testing procedure, the cylindrical samples were subjected to repeated passes of a 45 kg (100 lb) loaded wheel through a 690 KN/m^2 (100 psi) pressurized hose. Specimens were tested at 64^0 C temperature. The rut depth was measured in millimeters as a function of number of wheel passes. Ninety specimens were prepared and tested for rut depth at 8000 loading cycles. Figure 5.3 shows the typical variations of rut depth in millimeters with the number of load cycle for mixes containing various modified and unmodified binders. Three modified binders out of four showed rut depth of less than 3 mm. Others showed more than 4.5 mm rut depth at 8000 cycles of loading. From the figure it can be observed that more than 50% of the final rutting had occurred within 1000 loading cycles for all mixes.

The initial higher rate of rutting can be attributed to the initial densification or compaction of materials. After completion of initial densification, the rate of rutting (slope of rutting curve) decreases with the increase in loading cycles for each mixture. The slope of rutting curves in the range of 2000 cycle to 8000 cycles was almost equal for all mixes. Therefore, it can be concluded that the major difference in final rut depth was primarily due to densification of materials and not by plastic flow at higher cycles.

5.6 Analysis of Test Results

5.6.1 Overall Ranking

Figure 5.4 was a histogram showing all binders with increasing rut depth for samples with 6 to 8 percent air voids. A threshold value of rut depth for classifying a mix as good or poor performing has yet to be developed by ODOT. This study considered a rut depth of 6 mm as a threshold between excellent and good mixes, and poor mixes. Accordingly, in Figure 5.4, the binders were classified as E (excellent), G (good) and P (poor) on the basis of the threshold value associated with rutting performance. It was evident that 3 mixes fall in the category of excellent, 6 mixes were in the good category and 4 mixes exhibit poor rutting performance. These were the rating of 13 mixes prepared with various binders. It was also evident that the APA can be used for screening of poor mixtures or as a proof tester.

5.6.2 Effect of PG

Figure 5.5 shows that most PG2 and PG3 modified binder mixes have lower rut potential (excellent) compared to the rutting performance of PG1 (unmodified binders). The mean rut depth for the modified binders was 3.4 mm with a standard deviation of 1.8 mm. The unmodified binders showed a mean rut depth of 5.8 mm with a standard deviation value of 0.78 mm. The higher standard deviation for the case of modified binders was due to the poor performance of S8-PG 70-28 OK. From the binder's PG point of view, it can be shown that the overall performance of the modified binders was much better than that of the unmodified binders. This agrees with what was expected from the Superpave binder's specification point of view. However, there was no significant difference when the performance of the modified binder S8-PG 70-28 OK mixture was compared with the performance of unmodified binders. Again, the rutting performance of S7-PG3 did not differ when compared with the performance of the S7-PG2 binder mixture. From the test results, it was evident that the binder's higher performing grade was not a sufficient criterion to conclude that the mixture will perform well. A polymer-modified binders' performance should be evaluated in the mixes for performance.

5.6.3 Effect of Source

One of the objectives of the present study was to examine whether the performance of mixes with same PG binder grade differs with the source. For the PG1 binder, the following source ranking was S6>S5>S3>S1>S8>S4>S7>S2, based on the low to high rutting potential. From Figure 5.5, it can be seen that the rut potential for PG1 binders differs very little by source. But, in the case of the PG2 binder the performance of S8 was worst compared to the source S7. Based on the APA test results, it was evident that THE APA was sensitive to a binder's PG grade and source. A simple APA rut test can facilitate the prediction of binder's actual behavior in a HMA mix. Therefore, binders meeting the specification requirements of MP1-98 should also be evaluated by THE APA rut testing.

5.6.4 Effect of Rutting Factor

Figure 5.6 shows that the rut depth of mixes prepared with modified binders increases with decreasing $G^*/\sin\delta$. However, for the case of unmodified binders, rut depth decreases with the decreasing value of rutting factor. The overall ranking based on rutting factor, as shown in Figure 5.7, did not comply with the overall rank based on rutting performance as noted. Basically, the binder's DSR test properties could not reflect the mix performance. It can also be seen that the S8-G1 has the lowest rutting factor and S5-G1 has the highest rutting factor, but their rutting performance did not differ significantly. Figure 5.8 shows the rut depth at 500 cycles plotted with percentage increase in the binder's rutting factor due to RTFO aging. There was no significant effect of aging on rut depth at 500 cycles for both the modified and unmodified binders.

5.6.5 Effect of Viscosity

Figure 5.9 shows a bar plot of viscosity and rut depth for all the binders. It shows that the modified binders have higher viscosities or resistance to flow. Mixes containing these binders show low rut potential. The unmodified binders have low viscosity and exhibit high rut potential. Therefore, the viscosity of binders at 135^0 C can be a good performance-based binder evaluation parameter

5.7 Statistical Analysis

Many independent variables affect rutting. This study deals only with the variables that cover laboratory mix design, binder properties, rut specimen preparation and THE APA rut testing. The following nine variables were identified for data analysis: mixture binder content (P_b), air void (V_a), Void in Mineral Aggregate (VMA), Void Filled with Asphalt (VFA), absorbed asphalt (P_{ba}), viscosity (R_v), unaged $G^*/\sin\delta$ (DSR_u) and aged $G^*/\sin\delta$ (DSR_a), and THE APA load cycles. A single independent variable, when used to predict rut potential, was shown to give very poor prediction. For example, the amount of air voids was likely to be the most important physical property of asphalt mixes that relates to rutting (Brown et al., 1989). The correlation of air voids to rutting, as shown in Figure 5.10, was very poor. Brown et al. reported that total air voids might actually increase with additional traffic once rutting starts (Brown et al., 1989). A mixture can actually lose density once rutting begins.

According to many engineers, plastic flow was likely to begin once the air void was reduced to approximately 3 percent (Ford M.C., 1988). However, these analyses were performed at an air void of 6 to 7 percent that changes with load cycle. Therefore, air voids cannot reflect the actual correlation with rutting. Two rut prediction models were developed using Linear Multiple Regression (LMR) analysis and Nonlinear Regression (NR) analysis. A total of 45 sets of data, each with an average of 2 specimens were used for model development considering the above-mentioned parameters. The final prediction model includes only significant variables that affect rutting.

5.7.1 LMR Model

The stepwise method was employed for LMR model development. In step one, the independent variable that best correlated with the dependent variable (rutting) was included in the equation. In the second step, the remaining independent with the highest partial correlation with the dependent was entered. This process was repeated, at each stage partialling for previously entered independents, until the addition of a remaining independent did not increase the R-squared value by a significant amount (or until all variables were entered, of course). The dependent variable (rut depth, RD in millimeter) was multiplied by 100 and transferred to a logarithmic scale prior to incorporation into the linear model. The loading cycle was also transferred to logarithmic scale. The established terminal simplified form of the equation was,

$$Ln\ (RD\ .1000)\ = -2.51 - .20\ (R_v) + 5.29\ (P_b) - 4.92\ (P_{be}) - 0.59(G^*/\sin\delta)_u + 0.608\ Ln(Cycle) \ldots\ldots\ldots(5.1)$$

Summary statistics were reported in Table 5.5. The sample multiple correlation coefficient (R = 0.951) measured the degree of relationship between the actual Ln (RD.1000) and the predicted Ln (RD.1000). The value indicates that the relationship between Ln (RD. 1000) and the five independent variables was quite strong and positive. The sample Coefficient of Determination R-squared or R^2 measures the goodness-of-fit of the estimated Sample Regression Equation (SRP). It explains the proportion of the variation in the dependent variable predicted by the fitted SRP. The value of $R^2 = 0.905$ simply means that about 90% of the variation in Ln (RD.1000) was explained or accounted for by the estimated SRP that uses Ln (cycle), R_v, P_b, P_{ba}, DSR_u as the independent variables. Adjusted R-Squared is the sample Coefficient of Determination after adjusting for the degrees-of-freedom lost in the process of estimating the regression parameters. In this case, adjusted $R^2 = 0.904$ was a better measure of the goodness-of-fit of the estimated SRP than its nominal/unadjusted counterpart. Standard Error of Estimate $S_e = 0.507$ means that, on an average, the predicted values of the Ln (RD.1000) could vary by ±0.507 about the estimated regression equation for each value of independent variables during the sample period and by a much larger amount outside the sample period.

5.7.2 NR Model

The present study also employed the iterative estimation of Levenberg-Marquardt method for nonlinear model development. A regression model was called nonlinear, if the derivatives of the model with respect to the model parameters depend on one or more parameters. The specific advantages such as the parameters of a nonlinear model usually have direct interpretation in terms of the process or mechanism under considerations. In the modeling procedure, a nonlinear equation was studied to fit observed rutting giving initial values of parameters. The adjustment of all parameters was considered in one iteration. In the next iteration, the program attempts to improve on the fit by modifying the parameters. If any further improvement was not possible, the fit was considered converged. Iterations were stopped when the relative reduction between successive residual sums of squares was, at most, 1.000E-08. Several models with different parameters were examined. A model (for example, one with more parameters) was satisfactory, if the relative increase in sum-of-squares (going from one to another model) was greater than the relative increase in the degrees-of-freedom of that model, i.e. $(SS1-SS2)/SS2>(DF1-DF2)/DF2$, where, SS = regression sum of square and DF = degrees-of-freedom.

In a linear regression model, the quality of fit of a model was expressed in terms of the coefficient of determination, R^2. In nonlinear regression, such a measure was unfortunately not readily defined. One of the problems with the R^2 definition was that it requires the presence of an intercept, which most nonlinear models do not have. A measure, relatively closely corresponding to R^2 in the nonlinear case was Pseudo-R^2=1–SS (residual) /SS ($\text{Total}_{\text{Corrected}}$). The final form of the nonlinear model with Pseudo-R^2 =0.806 was,

$$RD = -2.57 + 0.35\ (V_a) - 1.09\ (R_v) + 1.68\ (P_b) - 0.41\ (VMA) - 0.71\ (G^*/\sin\delta)_u + 0.2442\ (Cycle)^{0.3359} \quad (5.2)$$

Table 5.6 contains the partitioning of the total sum of squares for the model and data into a regression sum of squares explained by the model and a residual sum of squares. The mean square error of this fit 0.5697 was the estimate of variability in the data when adjusted for the nonlinear model.

5.8 Comparison of Measured Rut Depth with Model Predictions

Figure 5.11 was a typical plot of measured versus model predicted rut depth for unmodified binder, S8-PG1-OK. The figure illustrates that the nonlinear prediction was closer to the measured rut depth and better than the linear prediction. In this case, the linear prediction was 3 mm more than both the measured rut depth and the nonlinear prediction. A poor nonlinear prediction for the case of unmodified binder, S2-PG1 as in Figure 5.12 shows that the nonlinear prediction follows the trend of measured rut depth with a rut depth about 2 mm less than the measured values. The linear predictions are higher than nonlinear predictions. Figure 5.13 and Figure 5.14 were the plots for modified binders S7-PG2 and S7-PG3, respectively. Both figures show that both nonlinear and linear predictions cannot explain the measured rut depth. The linear and nonlinear prediction equations include the viscosity and $G^*/\sin\delta$ (unaged), but these values do not vary significantly with modified binders. Although the final rut depth for linear prediction was better than the nonlinear prediction, the slope of the nonlinear prediction at higher load cycles was almost equal to measured rut depth.

5.9 Cycle-500 Versus Cycle-8000 Rut

The APA rut depth at 500-cycle can be a transition between consolidation and plastic flow of materials. The preceding analyses indicate that the visco-elastic properties of binder were significant at lower numbers of load cycles. At higher number of load cycles, binder properties were less significant and rate of rutting was almost equal for all binders. Therefore, the study has attempted to correlate 8000-cycle APA rut depth to 500-cycle rut depth. From the linear regression analysis, the following relation was obtained with a $R^2 = 0.83$:

$$RD = 1.96 + 1.8\,(RD_{500}) + 0.93\,(G^*/\sin\delta)_u - 2.3\,(G^*/\sin\delta)_a \qquad \text{...............(5.3)}$$

where, RD_{500} was the APA measured rut depth at 500-cycle. A nonlinear analysis was found to give better correlation with $R^2 = 0.89$. The following equation was obtained:

$$RD = 15.76 + 0.53(V_a - 0.17(R_v + 2.67(P_b - 0.8\,(VMA) - 2.16(G^*/\sin\delta)_u + 7.2(P_{ba} - 19.62(RD_{500})^{-0.17} \text{.......(5.4)}$$

The predicted 8000 cycle rut depths for all mixes were plotted against measured rut depth in Figure 5.15 and Figure 5.16 for linear and nonlinear prediction, respectively. These model prediction show that nonlinear prediction has less scatter along a 45^0 line drawn between the measured and predicted rut values. One of the basic ideas behind establishing this kind of relationship was to distinguish rutting performance of a pavement at the end of pavement life from its early life.

5.10 Concluding Remarks

- This study ranked 13 different binders based on mixes' performance and also on their properties. The binders' ranking based on their properties do not match with the mixture performance. A binders PG grade does not ensure the performance of the mixture containing the binder. Therefore, a binder satisfying the Superpave specification requirements should be evaluated by the HMA mix's rutting performance, determined by APA testing.

- The performance of modified binders having the same PG grade can vary significantly with the combining process or source. If the binders were unmodified or neat asphalts then the changing source will not vary in rutting depth more than 1 mm, if the binder satisfies AASHTO MP1-98. The binders' source was a changing target, the ranking of unmodified binder depending on the source become less significant.
- On the basis of the measured predicted results presented in this paper, the authors did not support the theory that a higher rutting factor can ensure lower rutting potential for mixes containing that binder. Rather, a binder's viscosity showed good correlation with the mix performance.
- If a rut depth of 6.00 mm was the divider between good and poor mixes, then ODOT's restriction, for using of unmodified binders in roads with 3M+ ESALs, on some sources should be reinvestigated.
- The study found that if the air voids of laboratory produced rut specimens were kept within 6 to 8%, then air voids played an insignificant role in the contribution to rut potential.
- A 500-cycle APA rut depth was a better predictor than a 8000-cycle rut depth, both for modified and unmodified binders' mix, and both linear and nonlinear regression models.
- The study developed two models based on the APA rut data on laboratory-produced samples. The nonlinear model was much more reliable than the linear prediction model. However, both models over predicted rut depth for mixes with modified binders.

- The study included one gradation of aggregate in the mixture. No consideration for wet rut testing on laboratory specimens was investigated.
- Rutting was a complex phenomenon. It involves many parameters. A neural network model could be very efficient for evaluating a complex phenomenon such as rutting.

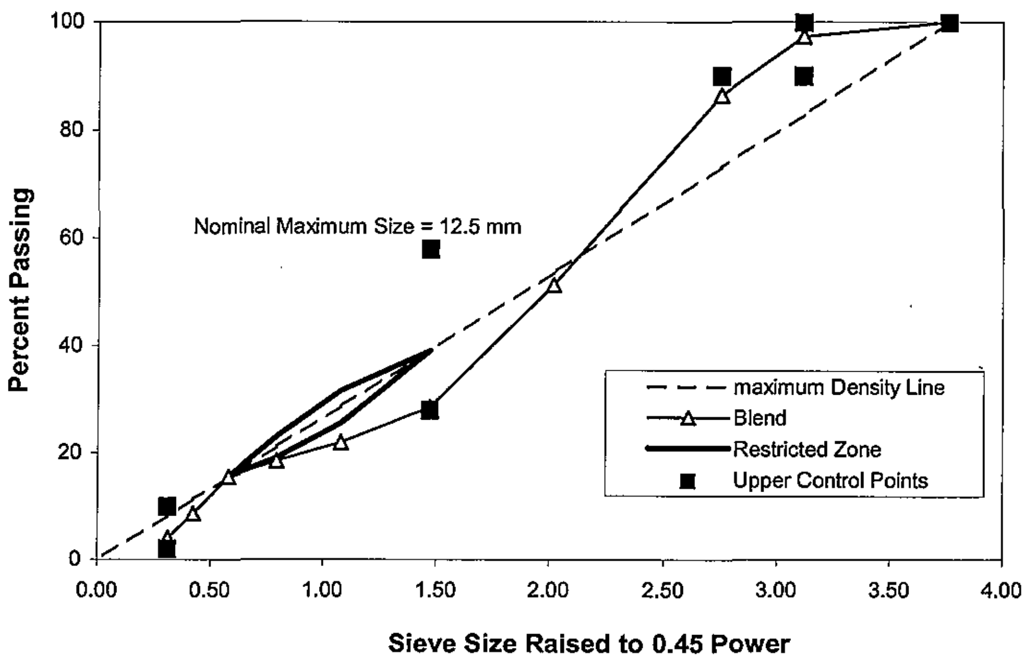


Figure 5.1 Blended Aggregate Gradation Used for Mix Design

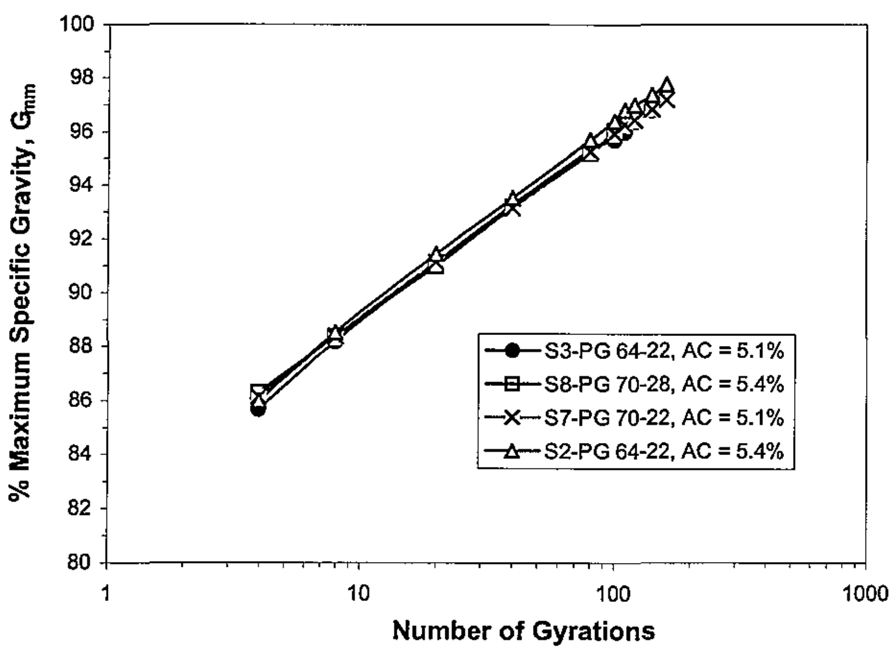


Figure 5.2 Average Densification Curve with Optimum Asphalt Content

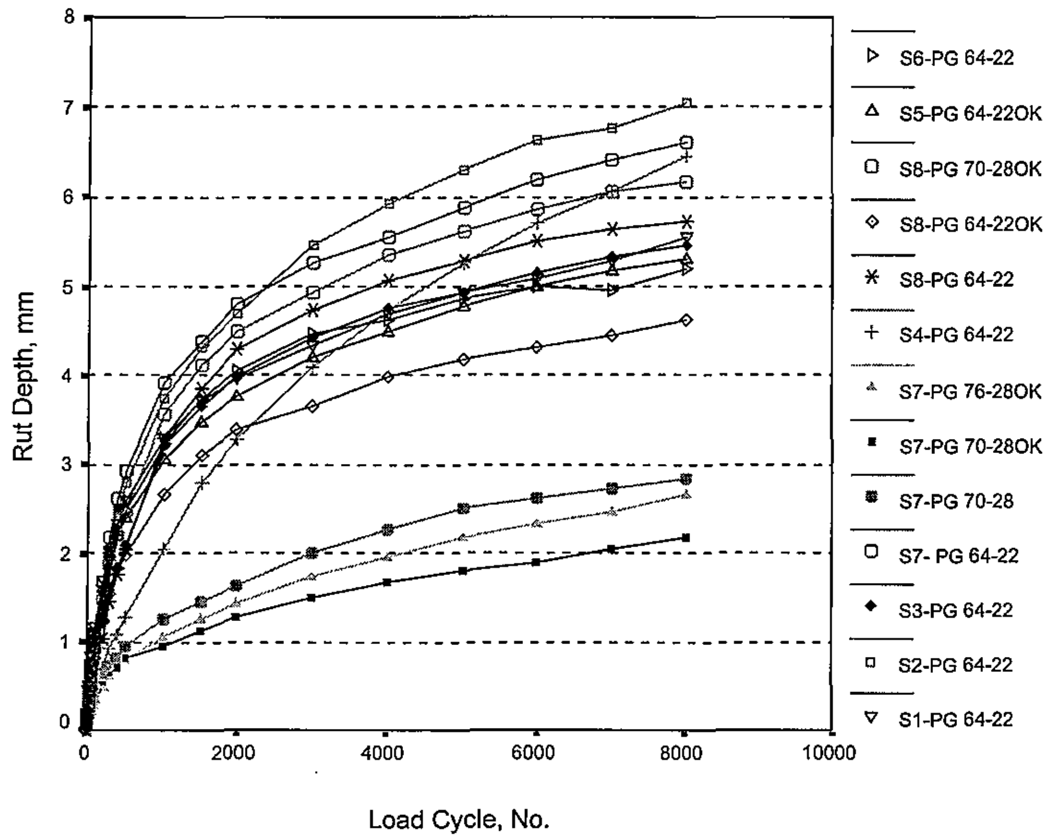


Figure 5.3 Typical Rut Depth versus Load Cycle

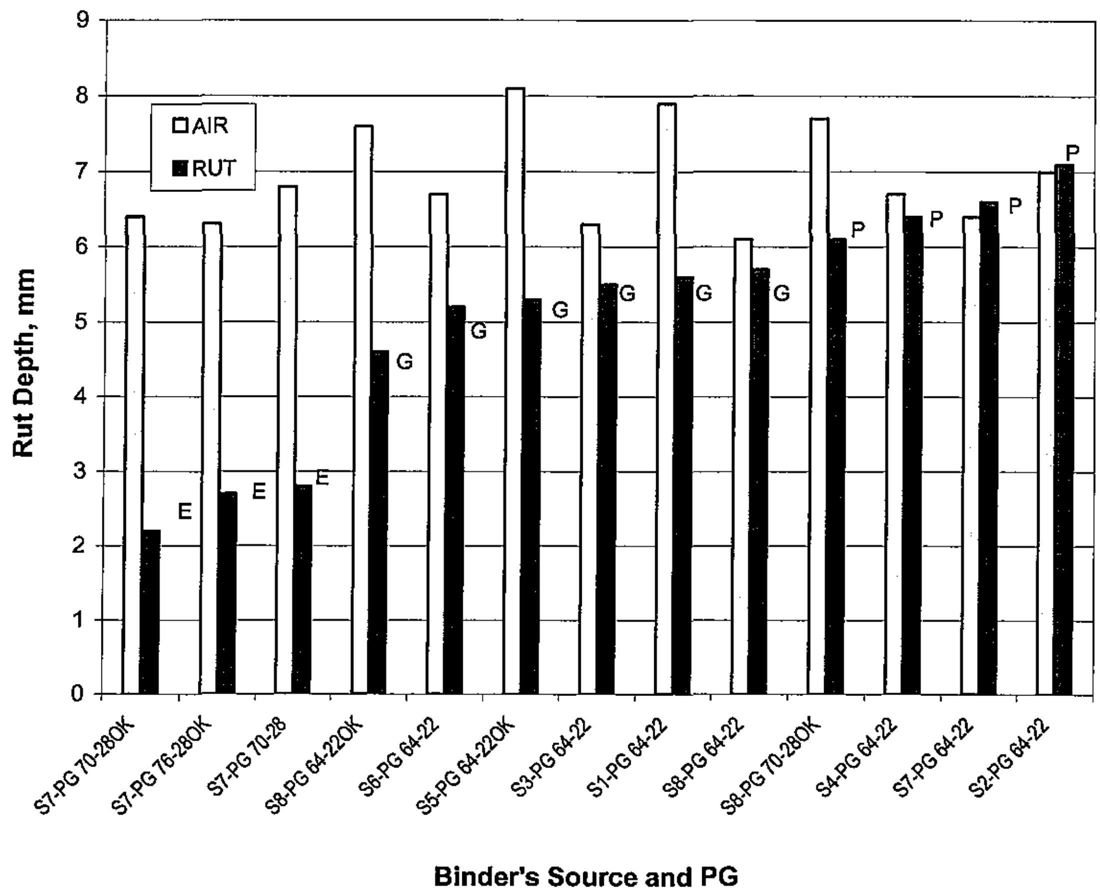


Figure 5.4 Overall Ranking of Mix

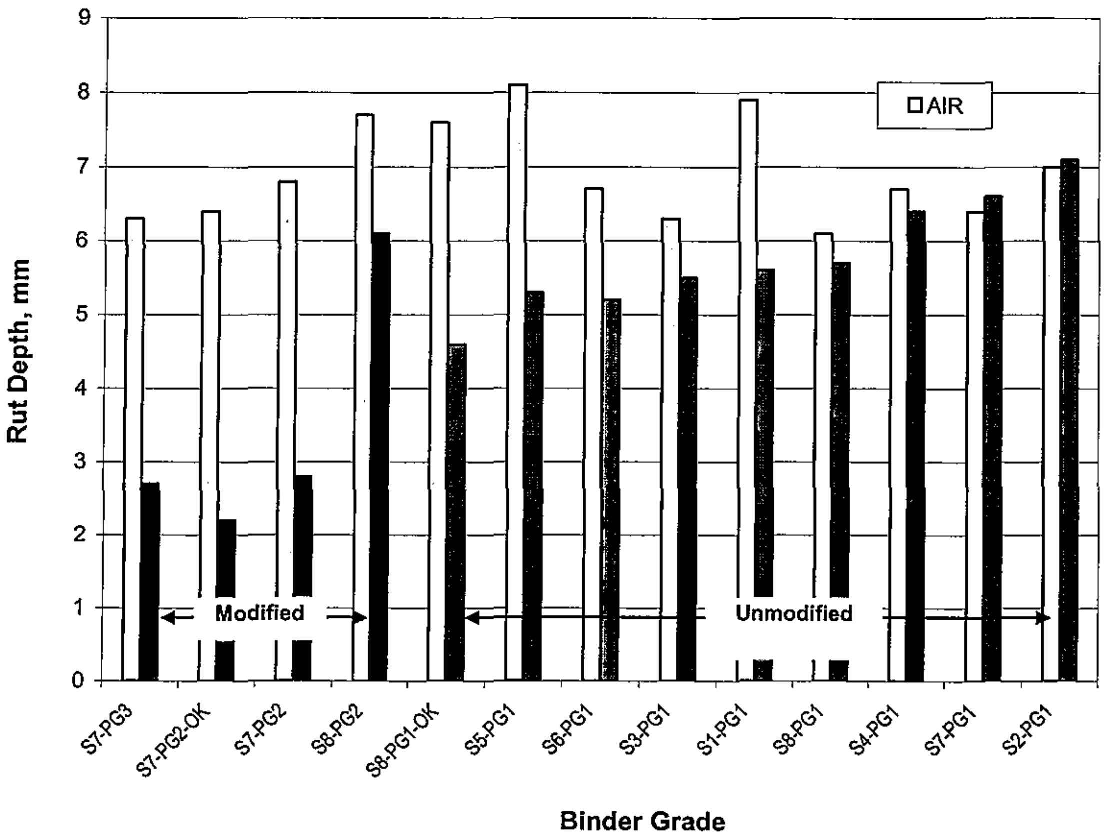


Figure 5.5 Modified and Unmodified Binders Performance

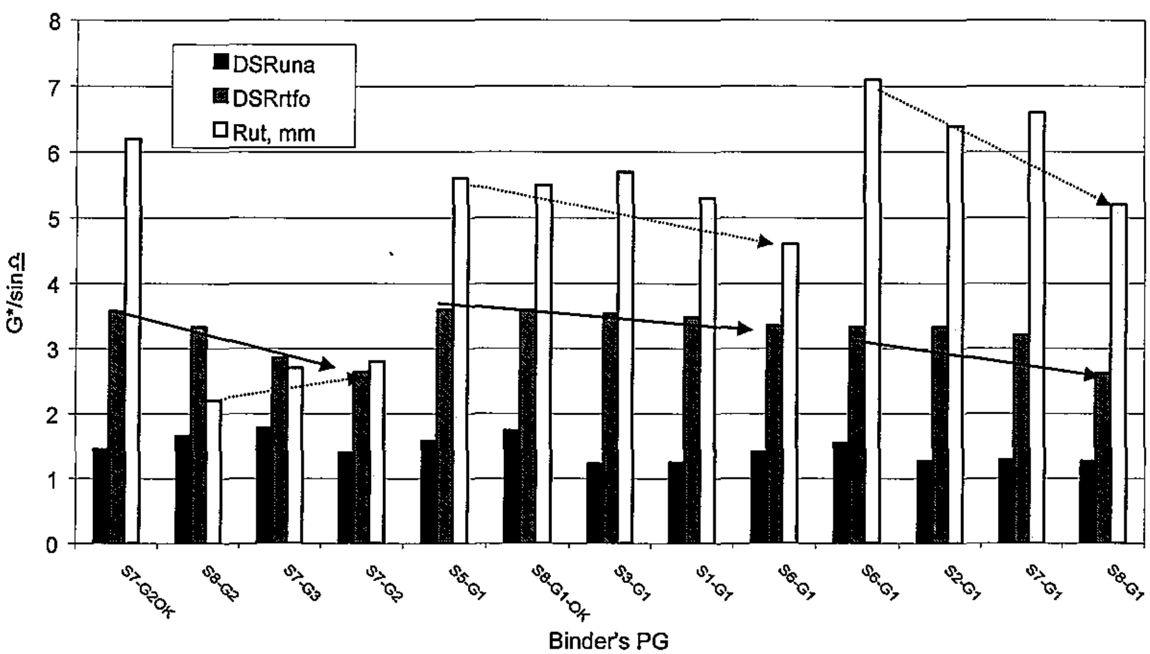


Figure 5.6 Effect of Rutting Factor on Rutting Performance

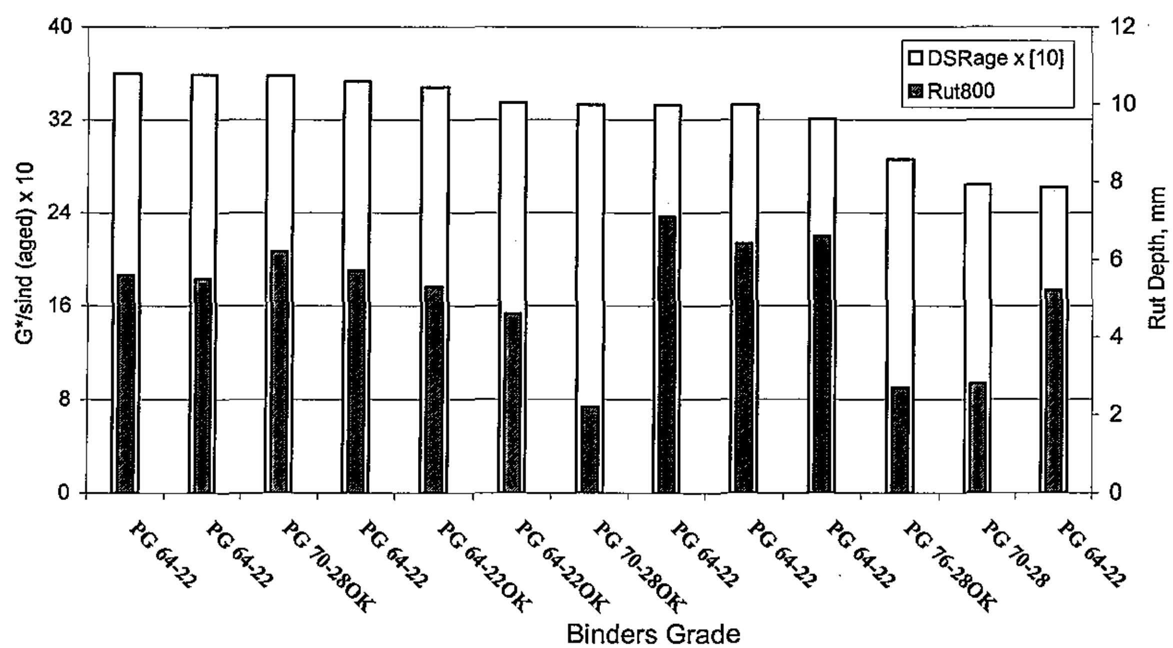


Figure 5.7 Overall Ranking of Binder based on G*/sinδ (aged) Values

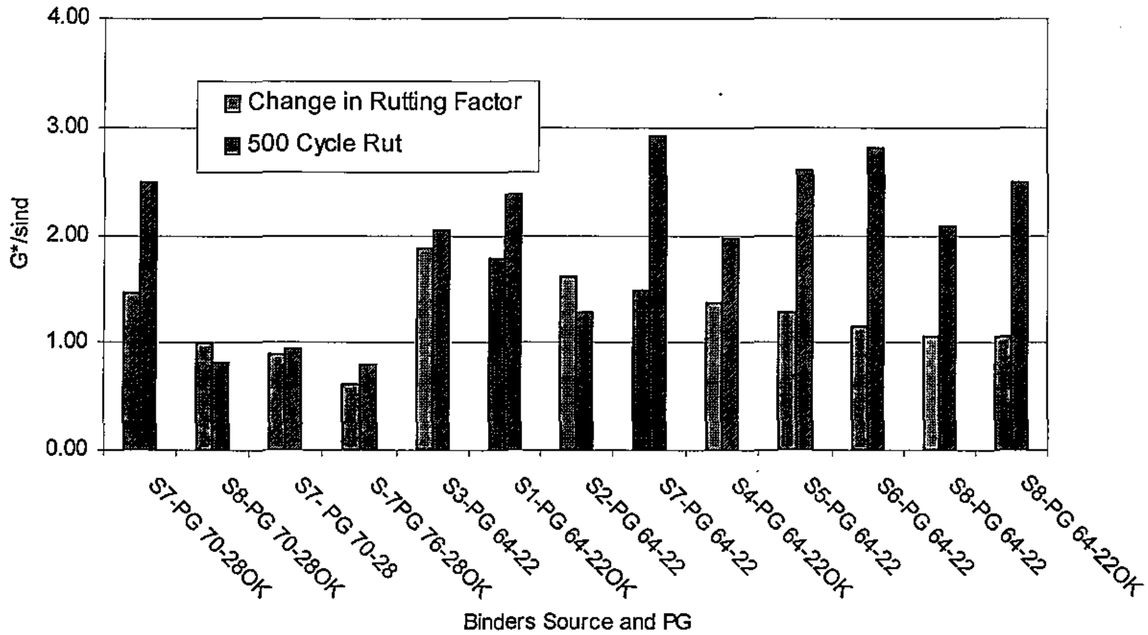


Figure 5.8 Effect of RTFO Aging on Binder's Performance

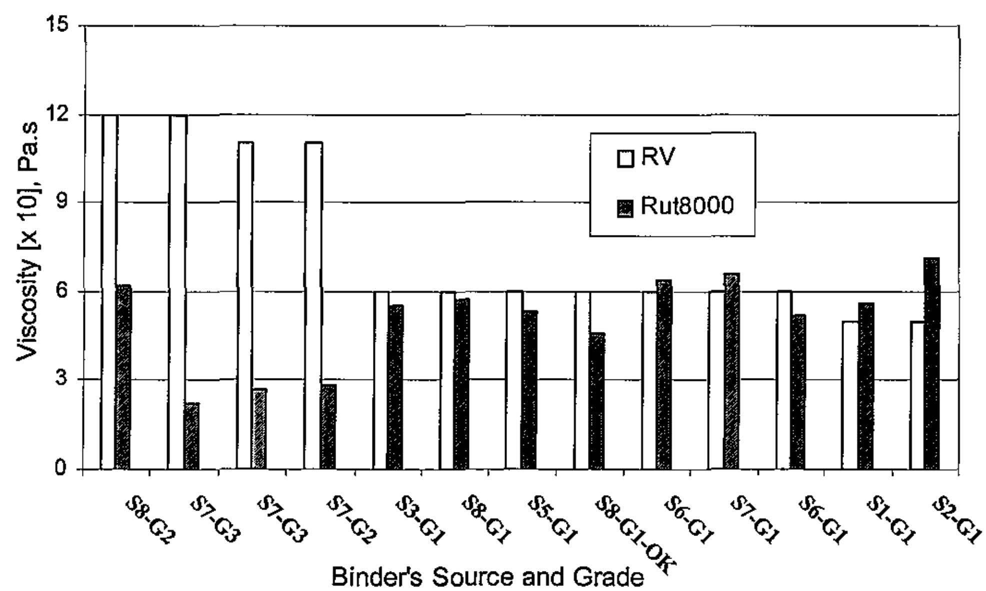


Figure 5.9 Effect of Viscosity on Rut Performance

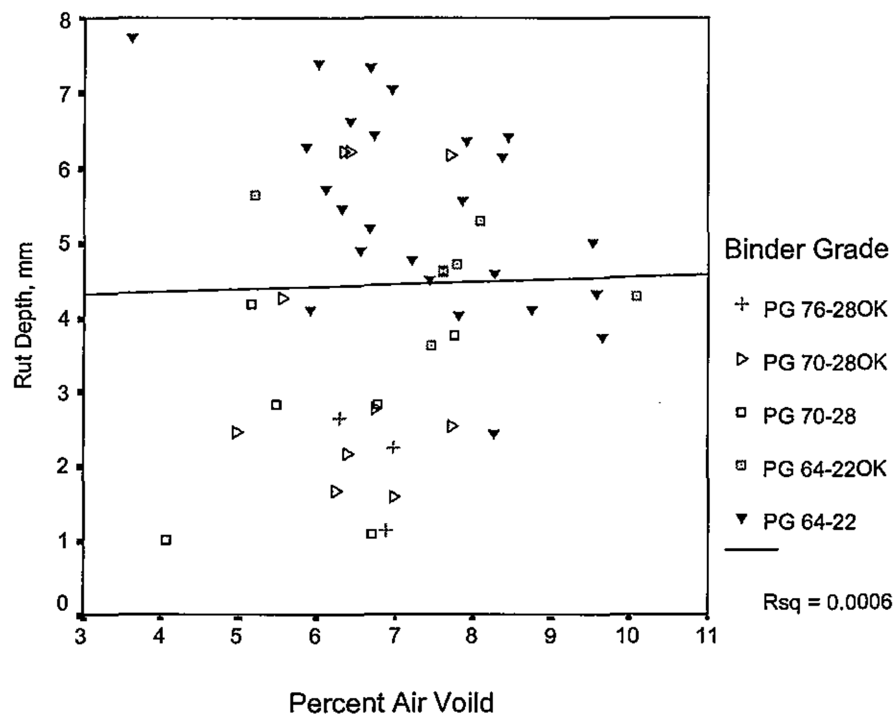


Figure 5.10 Correlation of Rut Depth with Percent Air Void

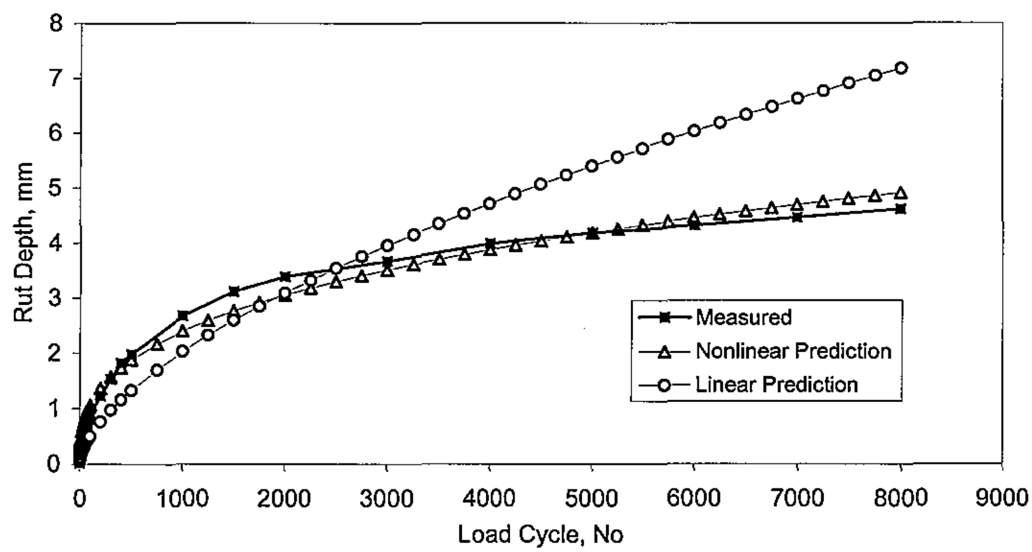


Figure 5.11 Prediction versus Measured Rut Depth for Binder S8-PG 64-22OK

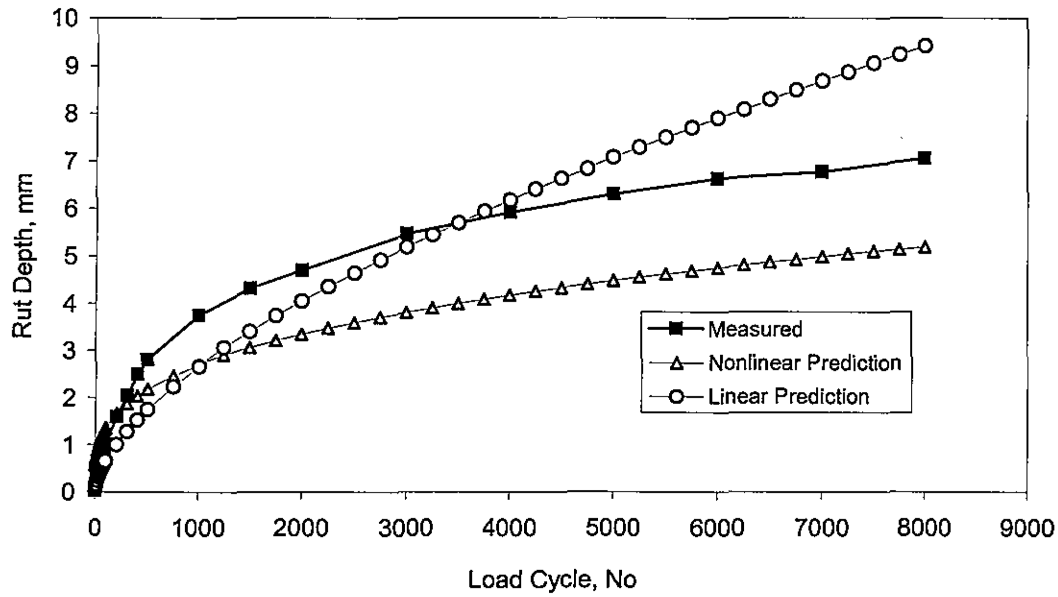


Figure 5.12 Prediction versus Measured Rut Depth for Binder S2-PG 64-22OK

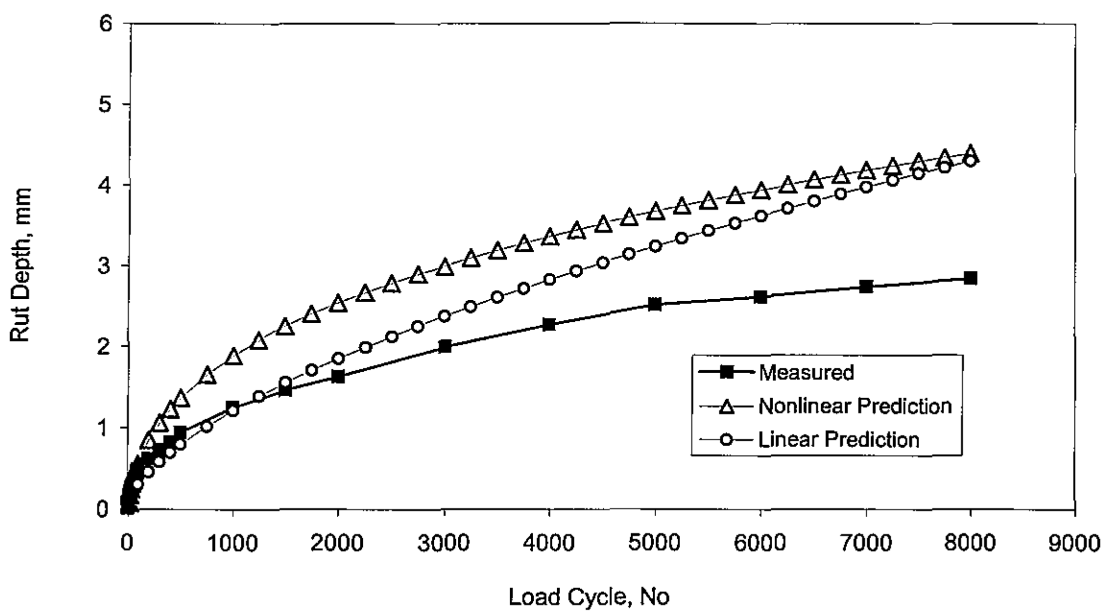


Figure 5.13 Prediction versus Measured Rut Depth for Binder S7-PG 70-28

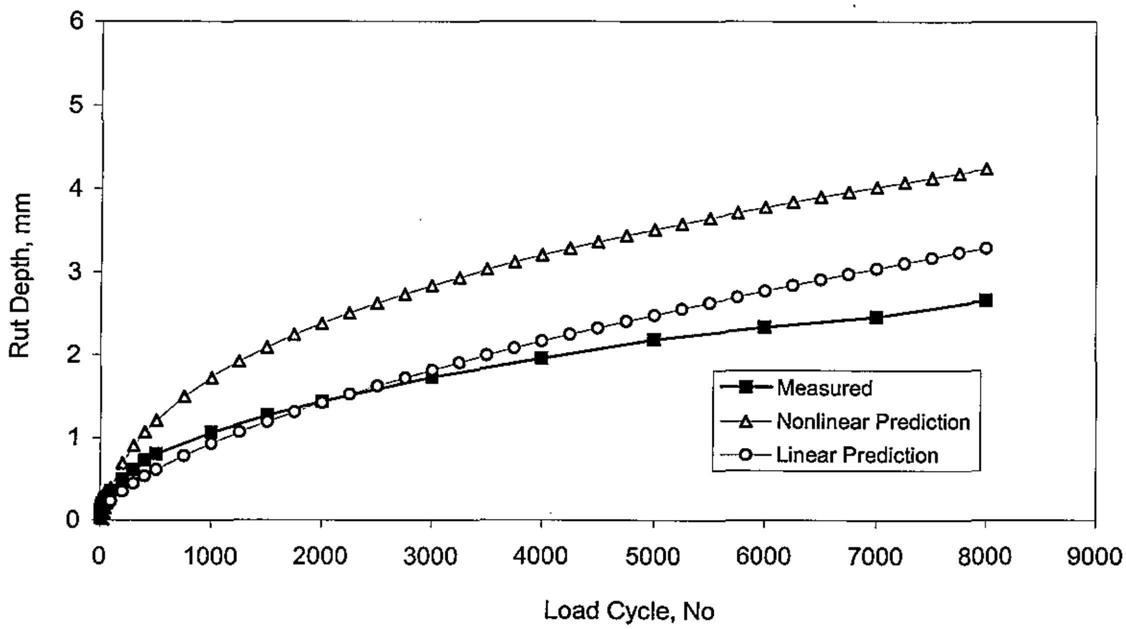


Figure 5.14 Prediction versus Measured Rut Depth for Binder S7-PG 76-28OK

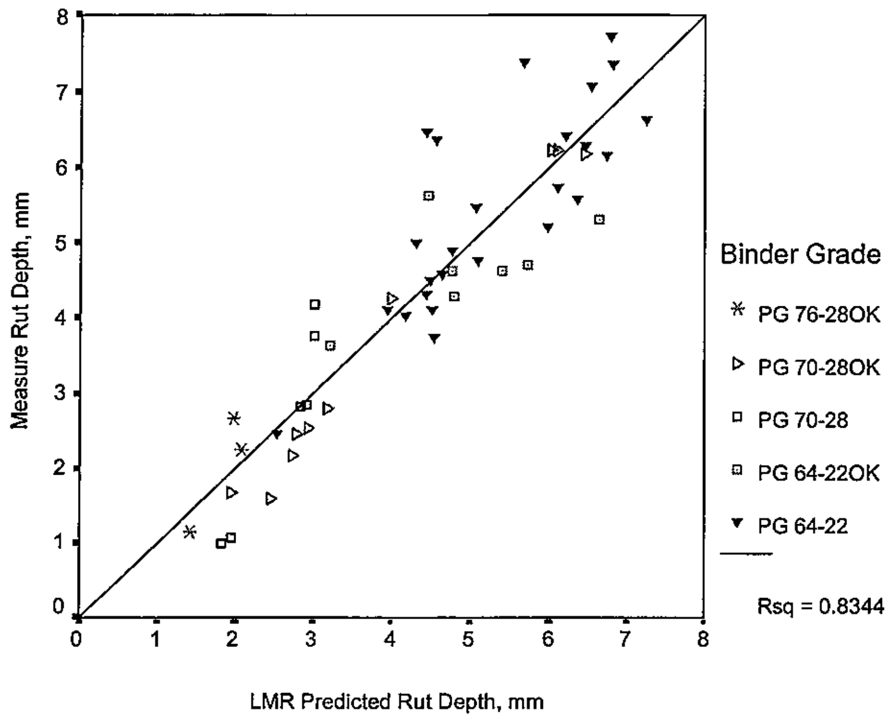


Figure 5.15 Linear Model Predicted 8000-Cycle Rut and Measured Rut

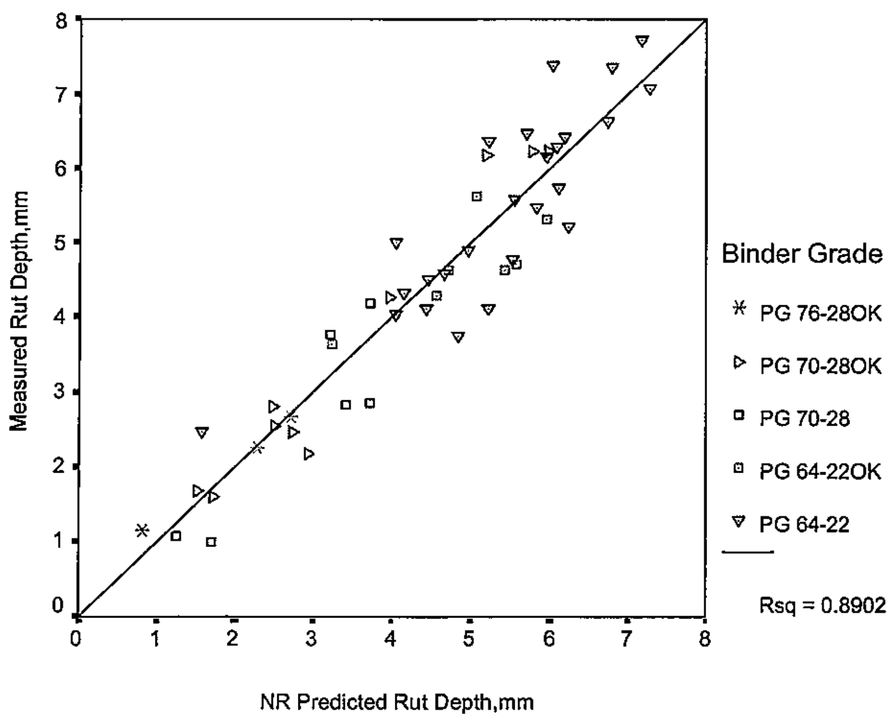


Figure 5.16 Nonlinear Model Predicted 8000-Cycle Rut and Measured Rut

Table 5.1 Properties of Unaged and RTFO Aged Binder

Binder Type	Binder Source	Binder PG	Specific Gravity	Viscosity $[R_v]^a$	$^b[G^*/\sin\delta_{unaged}]$	$^b[G^*/\sin\delta_{RTFO}]$	% Increase ($G^*/\sin\delta$)
Unmodified	S1	PG 64-22	1.0152	0.47	1.58	3.60	128
	S2	PG 64-22	1.0315	0.45	1.55	3.33	115
	S3	PG 64-22	1.0254	0.61	1.74	3.59	106
	S4	PG 64-22	1.0159	0.63	1.27	3.33	162
	S5	PG 64-22 OK	1.0103	0.64	1.25	3.48	178
	S6	PG 64-22	1.0076	0.59	1.27	2.62	106
	S7	PG 64-22	1.0151	0.60	1.29	3.21	149
	S8	PG 64-22	1.0110	0.60	1.23	3.53	187
Modified	S8	PG 64-22OK	1.0160	0.56	1.41	3.35	138
	S7	PG 70-28	1.0122	1.11	1.40	2.64	89
	S7	PG 70-28 OK	1.0150	1.20	1.66	3.33	101
	S8	PG 70-28 OK	1.0087	1.17	1.45	3.58	147
	S7	PG 76-28 OK	1.0258	1.08	1.78	2.86	61

Note: a = Test was performed 135 ^{0}C and 10 radian/second, b = The value of $G^*/\sin\delta$ is at high PG temperature

Table 5.2 Aggregate Information

Material	Source	Type	% Used
5/8" Chips	Western Rock at Davis, Oklahoma	Rhyolite	35
Screening	Western Rock at Davis, Oklahoma	Rhyolite	35
Shot	Dolese Co. at Davis, Oklahoma	Limestone	20
Sand	Dolese Co. at Oklahoma City, Oklahoma	Quartz	10

Table 5.3 Blended Aggregate Properties

Properties	Measured	Required
L.A. Abrasion, % wear	23	40 Max.
Durability Index	74	40 Min.
Insoluble Residue (%)	68.7	40 Min.
Fractured Faces (%)	100	95/90 Min.
Sand Equivalent (%)	52	45 Min.
Fine Aggregate Angularity (%)	46	45 Min.
Specific Gravity (SSD)	2.639	
Absorption (%)	0.189	

Table 5.4 Volumetric Properties for Optimum Asphalt Content

Binder	Optimum AC	% V_a at N_d	% VMA at N_d	% VFA at N_d	% G_{mm} at N_i	% G_{mm} at N_d
S3-G1	5.4	4.0	14.2	72.0	88.8	96.0
S8-G2	5.4	4.1	14.7	72.3	88.5	95.9
S7-G2	5.1	4.0	13.9	70.9	88.2	96.0
S2-G1	5.1	4.1	14.0	70.7	89.0	95.9
Superpave Requirement		4.0	14 min	65-76	Less than 89	96.0

Table 5.5 LMR Model Summary

Model	Independent Variables (Predictor)	R	R^2	Adjusted R^2	Std. Error of The Estimate
1	(Constant), LNCY	0.931	0.867	0.867	0.5989
2	(Constant), Ln (cycle), R_v	0.944	0.892	0.891	0.5409
3	(Constant), Ln (cycle), R_v, P_b	0.948	0.899	0.899	0.5219
4	(Constant), Ln (cycle), R_v, P_b P_{be}	0.950	0.902	0.902	0.5137
5	(Constant), Ln (cycle), R_v, P_b P_{be} DSR_u	0.951	0.905	0.905	0.5068
6	(Constant), Ln (cycle), P_b P_{be} DSR_u VFA	0.952	0.906	0.906	0.5038
7	(Constant), Ln (cycle), R_v, P_b P_{be} DSR_u, VFA	0.952	0.906	0.906	0.5039
8	(Constant), Ln (cycle), P_b P_{be} DSR_u VFA DSR_a	0.952	0.906	0.906	0.5031

Note: Dependent Variable = Ln (RD. 1000)

Table 5.6 NR Model Summary Statistics

Source	DF	Sum of Squares	Mean Square
Regression	8	6456.02	807.00
Residual	1522	867.09	0.5697
Uncorrected Total	1530	7323.11	
(Corrected Total)	1529	4473.71314	

R squared = 1 - Residual SS / Corrected SS = 0.80618

CHAPTER 6

RUTTING FACTOR

6.1 General

The work prescribed in this chapter consisted of screening and evaluating the relative weights of parameters that influence rutting performance of a mix. The objective of this work is to identify the most significant factors. A fractional factorial design was employed to implement experiments and statistical analysis considering seven influencing parameters for rutting. Mix rutting performance was determined in the Asphalt Pavement Analyzer (APA). Initially, seven rutting parameters for a Superpave mix (limestone) were investigated in a two level-design experiment in the laboratory. Parameters are asphalt content, binder grade, testing condition, temperature, compaction type, wheel load, and hose pressure. The test data was analyzed statistically. The results from this study showed that binders Performance Grade (PG), specimen testing condition (moisture sensitivity of mix), test temperature, and sample type affects a mix rutting performance significantly. Wheel load, hose pressure and percentage asphalt content at their chosen levels were shown to be less significant when compared to other factors. A most likely value of rut depth under the influence of aforementioned significant factors at a specified level was also postulated and verified by a confirmation experiments.

Next, the study investigated two gravel mixes with five rutting parameters at different levels. Identical statistical approaches were used to evaluate these parameters.

Wet condition, temperature and gradation were found to be significant. Rutting was highly affected by the introduction of moistures for all cases.

6.2 Background

Rutting is influenced mainly by loading, especially, cyclic loading, environment, and time dependent material behavior under loading. A list of factors affecting rutting in flexible pavement was shown in Table 6.1a and Figure 6.1a. A detailed discussion of the factor list was given below:

6.2.1 Loading

Loading is an important factor for rutting. Overstressing of the underlying pavement layers due to heavy loading was considered to be a significant cause of rutting. The contact area between the tire and the pavement increases with increasing wheel load and decreasing tire pressure. The average stress under the wheel was not proportional to the contact stress. Again, the actual traffic did not move in a single wheel path, but was laterally distributed over the traffic lane. Some of the material that was pushed sideward to the lateral swelling was also pushed backwards by the wheel moving along the edge of the central wheel path. Corte, et al. (1997) found that the rutting magnitude was increased from 20 to 40% going from dual wheels to the singlewide wheels. Several traffic variables can influence rutting and some of those were listed below:

- Wheel load, axle load and total vehicle load.
- Number of load applications, and their sequence

- Vehicle speed
- Lateral and lane distribution of load
- Tire pressure
- Wheel configuration

High-pressure truck tires and increased wheel loads were primary causes of increased rutting. Studies by Middleton et al (1986) and Kim et al (1988) have shown that truck tire inflation pressures have increased substantially above the 482 to 551 kPa (70 to 80 psi). Hudson et al (1988) have shown truck tire pressures to be as high as 965 kPa (140 psi). Temperature was another major factor to influence rutting.

6.2.2 Material Behavior

HMA layers contain both asphalt binder and mineral aggregate. The properties of the individual components and how they react with each other in the system affect its behavior. The rutting performance of the HMA primarily depends on the properties of the mix and to a lesser degree upon the individual properties of the binders or aggregate. There were occasions when the asphalt binder and aggregate were adequate. The mix failed to exhibit desired performance because of poor compaction, use of incorrect binder content, poor adhesion or some other problems associated with the mix. Also, mix properties alone were not sufficient to ensure satisfactory performance. The effect of the asphalt mix, asphalt binder and aggregate on rutting was discussed in this section.

6.2.2.1 Asphalt Cement Properties

Asphalt cement was a visco-elastic or thermoplastic material. Its consistency changes with temperature and rate of loading. Its properties can change during HMA production and can continue to change subsequently in service. Factors that contribute to age heardening were oxidation, volatilization, polymerization, thixotropy, syneresis, separation etc. The consistency (viscosity or penetration) of asphalt cement plays a relatively small role in the rut resistance of HMA if well graded, angular and rough textured aggregates were used. Some increased resistance to rutting can be obtained by using stiffer (high viscosity or low penetration) asphalt cements. However, stiffer asphalt cements were more prone to cracking during winter in cold regions, especially if they were used in the surface courses.

The current specification uses a performance grade (e.g., PG 64-22) or viscosity grade (e.g., AC-30) notation for the selected binder. The physical properties remain constant for all performance grades, but the temperature at which these properties must be achieved varies from grade to grade depending on the climate in which the asphalt binder was expected to perform. For example, a PG 64-28 grade was intended for use in an environment where an average seven-day maximum pavement temperature of 64 ^{0}C and a minimum pavement design temperature of –28 ^{0}C, were likely to be experienced. Some states in the southeastern portion of the US have started to use higher viscosity AC-30 grade in place of AC-20 to improve the resistance of the mix to rutting (Roberts, et al., 1996).

6.2.2.2 Mineral Aggregate Properties

Shear strength dependent on aggregate properties-such as coarse and fine aggregate angularity, elongation, flatness and clay content etc. For an example, by specifying a sufficient angularity, it was possible to achieve a high degree of internal friction and thus, high shear strength for rutting resistance. Angular-shaped particles exhibit greater interlock and internal friction; hence, result in greater mechanical stability than do rounded particles. On the other hand, mixes containing rounded particles, such as most natural gravels and sands, have better workability and require less compactive effort to obtain the require density. This ease of compaction was not necessarily an advantage, however, since mixes that were easy to compact during construction may continue to be densified under repeated traffic loading, ultimately leading to rutting due to low voids and plastic flow.

. Button et al. (1990) studied aggregate characteristic through creep-recovery performance of HMA and concluded that the rutting susceptibility of the mix increases dramatically when natural fine aggregate particles replace crushed particles for a given aggregate gradation. Aggregate gradation was perhaps the most important property of aggregate. It was the distribution of particle sizes expressed as a percent of the total weight and can be determined by sieve analysis. It affects almost all the important properties of a HMA, including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture drainage.

Hughes and Maupin (1987) reported that the binder type of asphalt concrete mixes does not appear to be as important as the gradation of aggregates and possibly the type of aggregates in minimizing the early rutting of pavement. Aggregate gradation provides more sufficient aggregate interlock. That was an effective way to improve the rutting response of the asphalt concrete pavements.

6.2.2.3 Mix Properties

The properties of an asphalt mix depend on percent air voids, asphalt content, asphalt to dust content and compaction effort. The Strategic Highway Research Program (SHRP) recommended that asphalt concrete mixes be designed based on maximizing the overall mechanical properties of the mix (Sherif, 1997). Air voids in asphalt concrete cannot bear stress. Lower air void content result in greater stiffness because it reflects a more homogeneous structure with better stress distribution. The fine-graded, 50-blow Marshall-designed mixes have experienced a significant number of failures due to rutting (Musselman, 1998). Aggregate properties had little effect on rutting when the void contents were low. When the voids were above 2.5%, mixes with higher fractured face counts and more fine angular aggregate showed more resistant to premature rutting (Cross and Brown, 1992).

The density of an HMA is usually expressed as a percent of theoretical maximum density. Increased compaction, asphalt content, filler content or any method that reduces the voids can achieve the required density. When voids filled exceed approximately 80% to 85%, the asphalt mix typically became unstable and rutting was likely to occur. Therefore, the method used to achieve density is important. Satisfactory compaction effort on a properly designed mix produces a mix with shear strength. While modifying the mix to reduce in-place voids will provide a mix with low shear strength and a tendency for high permanent deformation.

Filler materials (passing No. 200 sieve) fill the voids in an asphalt mix and lower the optimum asphalt content. Some fillers were necessary to obtain the desired stability, but excess fillers result in a mix at optimum asphalt content that was brittle and which tends to crack. The asphalt content must be adjusted for higher filler contents; otherwise, rutting will occur. Filler characteristics also vary with the gradation of the filler. Filler smaller than 10 microns act as an extender of the asphalt cement. Since the thickness of most asphalt films in dense-graded HMA was less than 10 microns. The filler, larger than 10 microns act as an aggregate. If an excessive amount of this larger sized mineral filler was present, the asphalt demand may increase because of increased VMA. Certain mineral fillers can increase the apparent viscosity of asphalt cement at 60 °C and thus make the mix more resistance to rutting. Therefore, care must be taken to consider not only the amount of mineral filler, but also its type and size in evaluating design mix (Anderson, 1987).

Asphalt cement content is probably the single largest contributor to rutting in HMA. Higher asphalt content increase the percent density and the thickness of the binder film between aggregates, which results in lower stress in the binder. Yet it is not good for rutting. A high asphalt content in HMA results in insufficient compaction during mix preparation. Barksdale (1973, 1987) concluded that the permanent deformation in dense-graded asphalt concrete, caused by both densification and shear distortion, is directly related to the asphalt content and is not sensitive to the material types, the gradation of aggregate and the level of compaction used in mix design.

Test parameters that significantly affect test results are the type and compaction method of test samples (West 1999). The two predominant "types" of test specimens are cylinders and beams/slabs. Cooley et al (1999) evaluated the density gradients in terms of variation in air voids within samples common to the APA and compared the two types of compactive effort used for THE APA samples: vibratory and gyratory compaction. Vibratory compaction tends to result in more compaction at the top and less compaction at the bottom of samples. Gyratory samples showed less compaction in the top and bottom of samples and significantly more compaction were noted in the middle. The vibratory specimens exhibited greater variability throughout a given specimen than was observed in gyratory specimens (Cooley et al., 1999 and Masad et. al., 1999). They found that the sample type could also influence THE APA rutting.

6.2.3 Environment

Temperature, moisture, water table, and frost can influence rutting. Among these, temperature has the greatest effect on rutting of HMA pavement. It was verified that if the temperature in the asphalt did not reach 30°C, no rutting was produced. When the temperature was close to 60°C to 65°C, the rut depth was doubled compared to the rut depth at 40°C to 45°C (Corte', et al., 1997). At high temperatures (e.g., > 100°C), asphalt cement acts almost entirely as a viscous fluid. At low temperature (e.g., < 0°C), asphalt cement behaves mostly like an elastic solid.

Brown and Snaith (1974) studied the effect of stress, strain and temperature on the rutting of asphalt concrete triaxial specimen subjected to dynamic loading for both deviatoric and the confining stress. They reported temperature as a major rut causative factor. Moisture was another factor that contributes to rutting performance. Rutting rates accelerate when moisture-induced damage was observed. Moisture susceptibility of a mix can be determined by conducting tests for rutting susceptibility on both dry and preconditioned specimens. The precondition was achieved by vacuum saturating a sample and then subjecting the sample to static saturation under water for at least 10 hours. The preconditioned specimens were then tested under water in the APA.

6.3 Experimental Design

Four mineral aggregates consisting of 16 mm chips (5/8 inch), screenings, shot and sand were incorporated in a Superpave method of mix design to produce specimens for testing in this study. The aggregate information was listed in Table 6.1. In the experimental procedure, aggregates were evaluated and gradation tests were performed to obtain the desired blend that met all of the Superpave gradation criteria. The final blend gradation was plotted on the 0.45 power chart, as shown in Figure 6.1, which passes Below the Restricted Zone (BRZ) with a nominal maximum size (NMS) of 12.5 mm (1/2 inch). The blended aggregate properties were summarized in Table 6.2. Two different binders, PG 62-22 and PG 70-28, were used in this study. The Superpave method of mix design was used with roadway traffic levels of more than 3 and less than 30 million Equivalent Single Axle Loads (ESAL).

The maximum number of gyrations, N_{max} was chosen to be 160 and the design number of gyrations, N_{design} was 100 (ODOT, 1999). Mixing temperature was kept at 163° C (325° F). Mixes were aged at 149° C (300° F) for minimum of two hours but less than four hours. The optimum asphalt contents were determined. Table 6.3 summarizes the optimum asphalt content of the two binders used in this study and volumetric properties as well as the Superpave volumetric criteria (AASHTO D PP3-00, 1998).

Two gravel mixes consisting of 25 mm (1 inch) rock, 19.0 mm (3/4 inch) chips, screenings and crushed gravel were also designed by varying the gradations as shown in Figure 6.2. Other design criteria such as average daily traffic, average high air temperature, mixing temperature etc. were the same as mentioned above. Cylindrical samples of 75 mm (3 inch) height were compacted with the SGC at target air voids of 6 to 8%. Beam samples of the same height were prepared with the AVC at the same target air voids. Samples were preconditioned either dry or wet for 10 hours before rut testing. For rut testing under water, samples were vacuum saturated to 55-75% saturation.

6.4 Identification of the Rutting Factors

HMA was a composite material composed of graded aggregates embedded in a matrix of asphalt cement that fills part of the space between the aggregate particles and binds them together. The properties of the individual components and how they react with each other in the system affects the behavior of a mix. There were occasions when asphalt binders and aggregates were adequate but the mix fails to exhibit a desired level of performance because of poor compaction, use of incorrect binder content, poor adhesion or some other problem or combination of problems associated with the mix. Again, mix properties alone were not sufficient to ensure satisfactory performance. A pavement material was subjected to three dimensional stress induced by repeated loads. This stress-response depends on the time or rate of loading, temperature, and material properties.

6.5 Selection of the Factor's Levels

Aggregate affects almost all the important properties of a HMA, including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture drainage. The aggregate factor includes aggregate size (i.e., NMS of 12.5 mm [1/2 inch] or 19.0 mm [3/4 inch]), type (i.e., limestone or gravel), and shape (i.e., rounded or angular). The properties of an asphalt mix depend on percent air voids, asphalt content, asphalt to dust content and compaction effort. Mix factors includes percent air voids (i.e., 4% or 7%), percent asphalt content (i.e., optimum, more or less than optimum), Voids in Mineral Aggregate (i.e., VMA of 15 or 18), mix gradation (above, through or below the restricted zone).

Asphalt cement is a visco-elastic or thermoplastic material. Its consistency changes with temperature and rate of loading. Binder factors include stiffness (i.e., soft or stiff binder), source (source A or Source B) and Performing Grade (i.e., PG 64-22 or PG 70-22). Load was also an important factor for rutting. Overstressing of the underlying pavement layers due to heavy loading was considered a significant cause of rutting. The contact area between the tire and the pavement increases with increasing wheel load and increasing tire pressure. The average stress under the wheel was not proportional to the contact stress. Load factor includes wheel load (i.e., 100 or 110 lb), hose pressure (i.e., 100 or 110 psi), and load repetition (i.e., 8000 cycles or 10000 cycles). Temperature, moisture, water table, and frost can also influence rutting. Among these, temperature had the greatest effect on rutting of HMA pavement. Environmental factors include temperature (i.e., 60° C, 62° C or 64° C, 66° C), testing condition (i.e., wet or dry), and aging (i.e., no aging, short term aging or long term aging). The sample type (i.e., AVC for beam samples or THE SGC for cylindrical samples) also influences laboratory rutting.

6.6 Optimization of the Test Matrix

Seven factors were incorporated into the orthogonal arrays of L_8 balanced design (Kyle, 1995). Designations for orthogonal arrays include the letter 'L' first then the subscript number second. The subscript after the L denotes the number of trials that must be executed in a given design. For example, in an L_4 array, four trials would be required to complete the experiment. It was decided to explore the following seven factors: wheel load, hose pressure, test temperature, test condition, sample type, asphalt content and binder grade (see Table 6.4 for details). Each factor in the array was compared to all other factors in equal number of times. The selected factors were assigned to the designed array, as shown in the Table 6.5 to develop an experimental matrix. Table 6.6 summarizes the rutting averages for two selected experiments. A total of 8 beam samples and 16 cylindrical samples were tested in accordance with the test matrix. Table 6.7 summarizes the rutting averages achieved over two trial experiments. Table 6.7 also summarizes the trials that needed to be added together to obtain the Level 1 and Level 2 totals for each factor. This parameter was needed to calculate the sums of squares.

6.7 Analysis of Data

In an L_8 array, sample type (denoted by factor F) was set at level 1 in trials 1, 4, 5, and 8. The calculation of F_1 (factor F at level 1) at level sum was accomplished by adding together the totals for each of these trials as shown in Table 6.8. Level sums for other factors could be performed in a similar way.

Table 6.9 shows the level sum for each factor. The totals for each factor was also calculated and recorded in this table. The sum of level 1 and level 2 were equal to the total of the experiment. The next step was to perform the sums of squares (SS_x) calculation. The modified sums of squares were calculated by the following formula:

$$SS_x = \left(\frac{Level\ Sums_{Level1}^2 + Level\ Sums_{Level2}^2}{n} \right) - \left[\frac{\left(\sum X_i\right)^2}{N} \right] \qquad (1)$$

where

SS_x = Sum of squares for factor x,

Level $Sum_{Level\ 1}$ = Level sum for factor x at level 1,

Level $Sum_{Level\ 2}$ = Level sum for factor x at level 2,

n = Sum of data points used in calculating the level sums for either level 1 or level 2, and

N = Total number of data points in the experiment.

Table 6.10 summarizes the sum of squares calculations for each factor and the total variation, SS_{Total}, in the experiment. This study adopted the simplest way of making a significant plot as shown in Figure 6.3. The SS_x for each factor was plotted in descending order of magnitude from the left to the right and points were connected by a solid line. It was evident from the plot which factors were expected to have the greatest effect on the quality characteristic (i.e., the dependent factor) and which would not. The factors along the steepest section of the graph were the more important ones and those along the flat portion or the bottom of the slope were the least important. From Figure 6.3, factor A (i.e., binder's grade) was considered the most significant followed by factors F (sample type), C (test temperature), and B (sample conditioning). All remaining factors, D (wheel load), G (percentage asphalt), and E (hose pressure), were not considered significant.

An Analysis of Variance (ANOVA) calculation was also performed. The premise of an ANOVA calculation was to compare the contribution by each factor to the explained variation to that of the unexplained variation (i.e., experimental error). The factors that had little or no effect on rutting were grouped. Factors that were grouped together were those calculated to have the smallest sums of squares. The factor that resulted from the grouping was represented by error term. Factors D, E and G were grouped together as error term and were summarized in Table 6.11. The degrees of freedom, df_x, and variance, V_x of each factor was calculated in ANOVA, Table 6.11. The expected sum of squares, SS'_x and the percent contribution, P, were calculated and incorporated in the ANOVA table. The expected sums of squares were calculated to compensate for any experimental error that influenced the calculation of the sum of squares. The percentage contribution, P, was used to estimate the portion of the variation that could be attributed to a specific factor in the experiment. The following formulas were used for calculations of P and SS'_x:

$$P=\left(\frac{SS'_x}{SS_{total}}\right)\times 100 \qquad (2)$$

$$SS'_x = SS_x - (V_{err} - df_x) \qquad (3)$$

The percent contribution due to error, P was an important factor as it offers a quantitative evaluation of experimental results. There were also situations where a factor may be determined to be statistically insignificant according to the F statistic, but that it still had a sizable percent contribution. A comparison of the calculated F statistics obtained from F-table suggests that the binder grade, sample type, temperature and sample conditioning were statistically significant factors. These four factors contributed about 93%. There were also 7% of the percent contributions that could not be attributed to any of the factors examined in this investigation. This suggests that there may be other factors or possibly an interaction between factors, not yet identified, which could also influence the APA rutting average.

6.8 Estimation of Rut Interval

An experiment could be set using above four factors at a desired level. An estimation of the expected results would be obtained by using the average values for the trial containing the recommended factor levels. The mean response was estimated by the following equation:

$$\overline{\mu} = \overline{T} + \sum_{i=1}^{n} (LS_{xi} - \overline{T}) \qquad (4)$$

where

$\overline{\mu}$ = estimate of the mean response,

$\overline{T}$ = mean of all experimental data, and

LS_{xi} = optimal level sum response for thssignificnt facto athe levl of interest.

The key factors and their respective level sum response were summarized in Table 6.12. Using these values in Equation (4), the predicted mean response was found to be 9.18 mm. Based on this prediction, a beam sample prepared by binder of PG 64-22 could be expected to rut about 9.18 mm under wet condition at a temperature of 64^o C (147.2^o F). However, the estimation of the mean response was meaningful only if there was some idea of the spread that could be expected in the data. An estimation of the spread in the data was obtained by calculating a confidence interval or CI. The error of the estimate was defined as:

$$Error = \sqrt{\left(\frac{F_{x,err} \; x V_{err}}{n_{eff}} \right)} \tag{5}$$

where

$F_{x,err}$ = F statistic associated with the specified ✓-risk and the degrees of freedom for each factor in the experiment, x and the degrees of error term, err,

V_{err} = Variance for the error term, and

n_{eff} = effective number of degrees of freedom for the error.

Using Equation (5), the error for the estimate was calculated to be ± 3.36 mm. Therefore, the confidence interval could be expressed as: CI = 9.18 ± 0.56 mm. This means that the predicted rutting value will be between 8.6 mm to 9.7 mm if the level of parameters in Table 12 was used in the APA rut testing.

6.9 Confirmation of Factor Levels

It becomes obvious to determine whether the additional tests, performed with parameter levels as in Table 6.12, would show a rut value in the predicted range of rut value (i.e. 8.6-9.7 mm). This was accomplished by conducting what was commonly referred to as a confirmation experiment. A confirmation experiment consisted of adopting the recommended levels of the key factors (i.e., binder grade 64-22, wet condition, 64 ° C temperature and beam sample) and the most favorable settings of all remaining factors investigated in the experiment. Figure 6.4 shows the typical test result for a confirmation experiment. The final value of rut depth from APA tests (rut depth at 8000 cycles) was found to be 9.29 mm and 9.69 mm, which was within the predicted range. Therefore, the predicted range of the rut depth, for the mentioned conditions using APA, was considered satisfactory.

6.10 Gravel Mix

Five factors: wheel load, hose pressure, test temperature, test condition, mix gradation were selected for evaluation in two-designed experiments of gravel mix. The selected factors were assigned to the designed array, as shown in the Table 6.13 and Table 14 to develop experimental matrices. Statistical analysis as described above was performed. From significance plot of Figure 6.5 and Figure 6.6, it was evident that the effect of wheel load and hose pressure at the selected range can be neglected. The gradation had the second highest effect on rutting among these five parameters. Results of the statistical analysis were summarized in Table 6.15.

The confidence interval for a gravel mix with temperature level 60-64° C was, CI = 11.55 ± 1.79 mm, where as CI = 11.53 ± 3.47 mm. This means that the predicted rutting value will be between 8.0 mm to 15.0 mm if the level of parameters in Table 6.14 were used in the APA rut testing. Predicted value will be 9.7 to 13.3 mm if the parameters in Table 13 were used. It was evident that gravel mix has higher rut potential compared to the Superpave Mix. It was noticed that the gravel mix during testing under water created large amounts of uncoated fines or dust.

6.11 Conclusions

This study employed a factorial design for screening several APA rutting factors. Knowledge of underlying physics was used to choose the levels of factors. Seven factors were chosen to examine their relative effect on the APA rutting. It was found that four factors out of nine had important effects on laboratory prediction of rutting for the case of the limestone mix used in this study. Based on the results of this study, it was evident that the beam samples yielded collectively higher rutting in the APA under wet conditions. A testing temperature of 64° C (147.2° F) with a PG 64-22 binder showed the highest average rut depth. Wheel load, hose pressure and asphalt content at their chosen level did not show any significant effects on rutting. However, the estimate of the effect of these insignificant factors on rutting was associated with setting the low and the high value of that factor. A prediction of rut depth for the limestone mix using these significant factors was found to predict a rut depth between 8.6 mm to 9.7 mm and was verified by confirmation experiments. Two other test matrices were covered for gravel mix.

Gravel mix passing through the restricted zone showed higher rut potential when compared to the rut potential of gravel mix passing below the restricted zone. From the test result, it was found that temperature has a significant effect for the case of gravel mix. For the case of gravel mix, a considerable amount of dust or fines was produced during rut tests under water. For some cases, aggregates under the APA hose showed no coating when gravel mixes were tested under water testing. Therefore, stripping has to be investigated carefully before using any gravel mix in the field. This study considered selected parameters only at two different levels. However, rutting can be affected by other parameters such as aggregates type, size, boundary and loading conditions in the test set-up as well as other factor-levels not considered here.

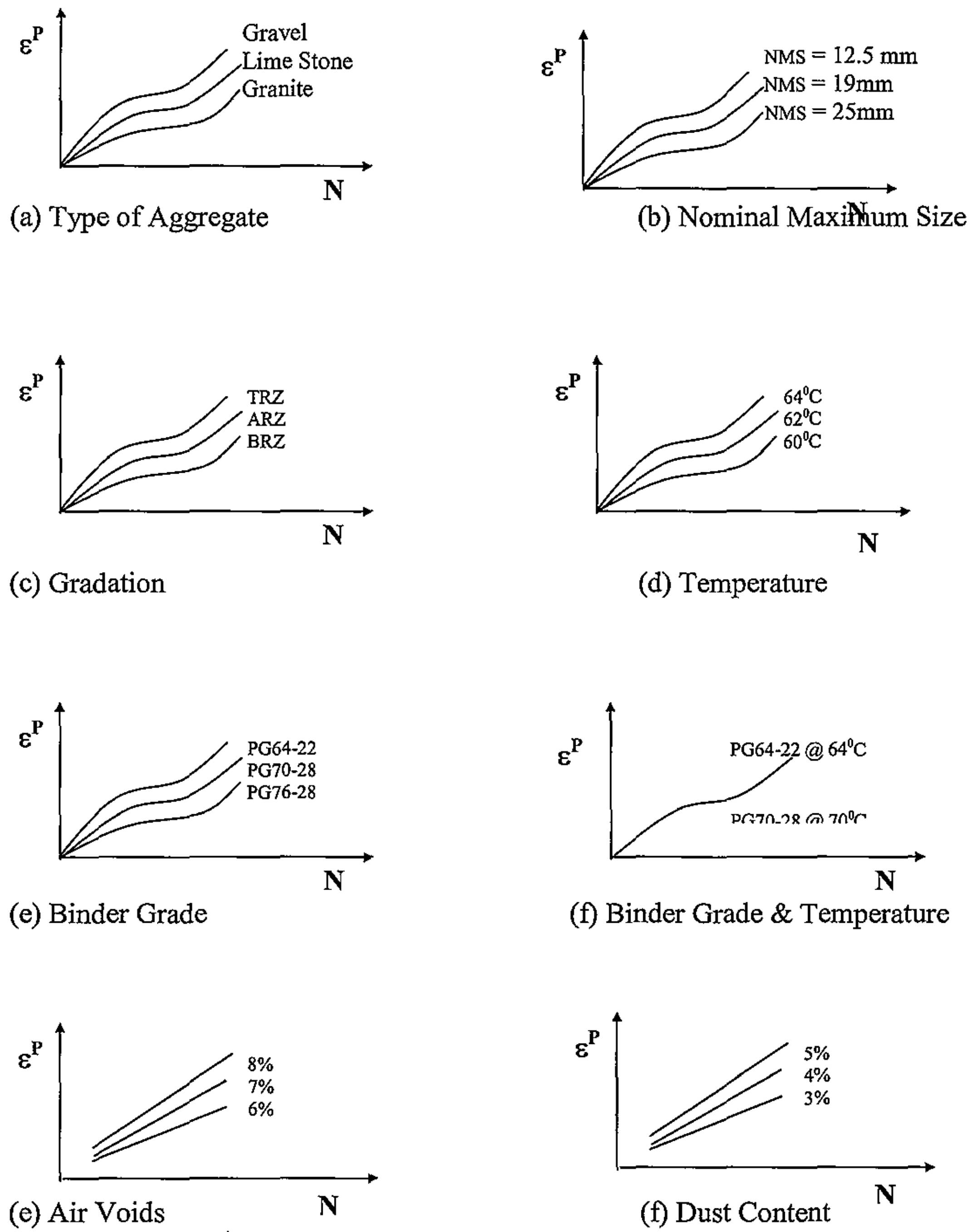
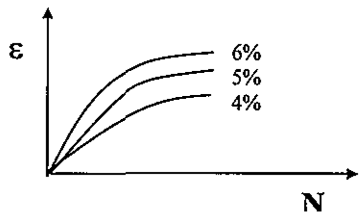
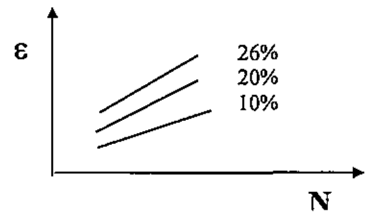


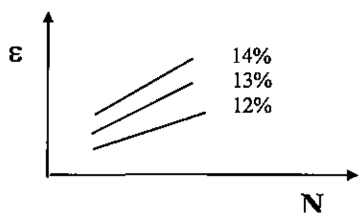
Figure 6.1a Factors Affecting Rutting (Fig. 6.1a continue)



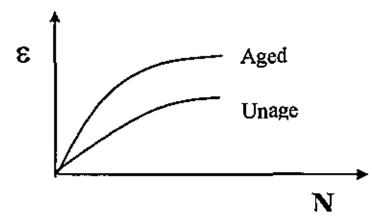
(g) Asphalt Content



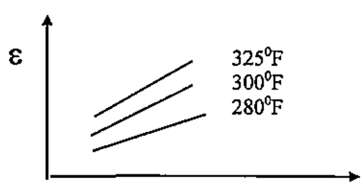
(h) Natural Sand Content



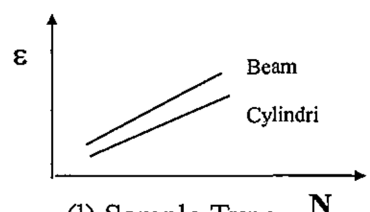
(i) VMA



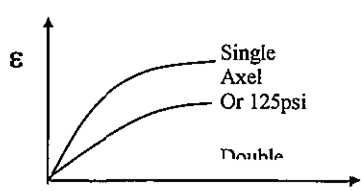
(j) Short Term Aging



(k) Compaction Temperature

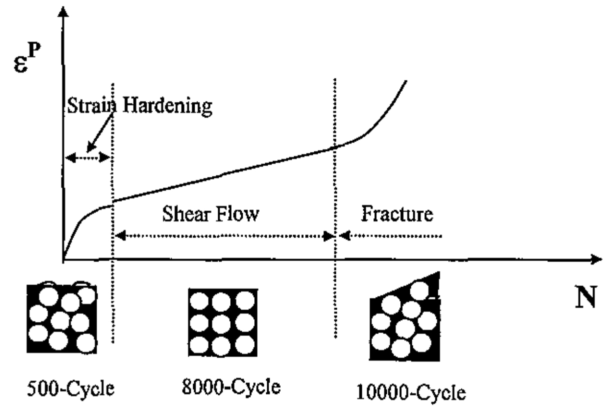


(l) Sample Type



(m) Wheel Load

Figure 6.1a Factors Affecting Rutting (Fig. 6.1a continue)



(n) Density gradient Analysis

1. Particle orientation
   (Image Analysis)
2. Mold Effect
3. Specimen Geometry

(o) Other Factors

- VFA *f*(Traffic level)
- Effective Asphalt Content
- Effective of Absorptive Aggregate
- Effect of incorporating RAM
- Compaction Energy (NG or vibration-time)
- Hose Pressure
- Specimen Conditioning
- Manual measurement vs. Auto measurement
- Seating Cycle
- Fractured face
- Fine Aggregate Angularity
- Percent Passing Sieve No. 4 and No. 40
- Specific Gravity of Binder (Source)
- Liquid Antis trip (w/o lime)

Figure 6.1a Factors Affecting Rutting

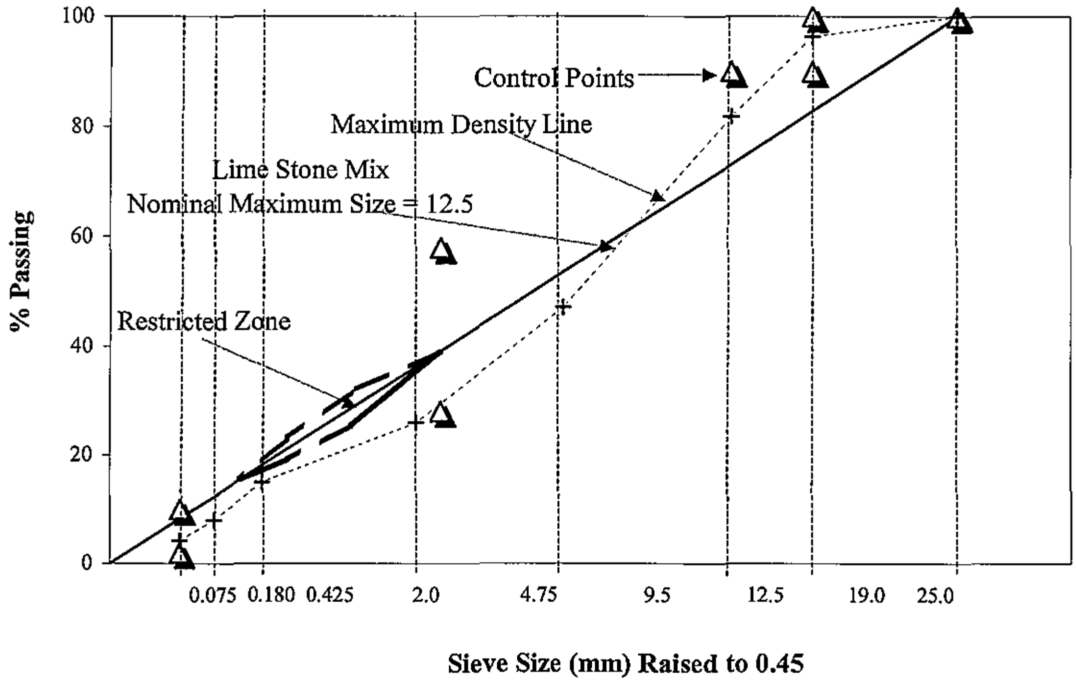


Figure 6.1 Blended Aggregate Gradation for Limestone Mix

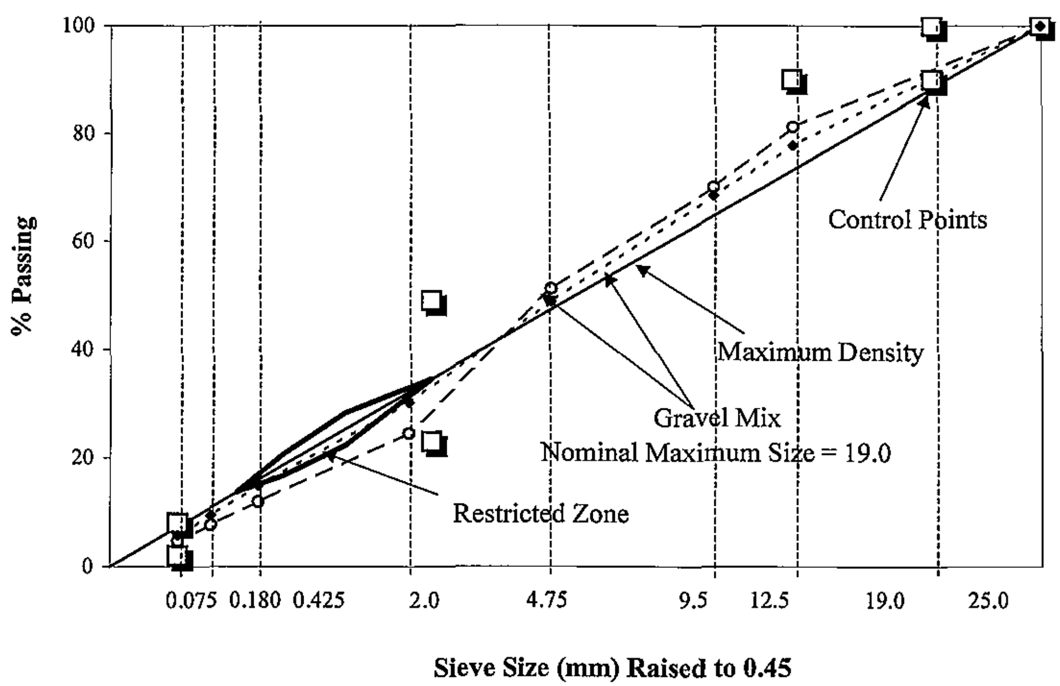


Figure 6.2 Gradation Curves for Gravel Mixes

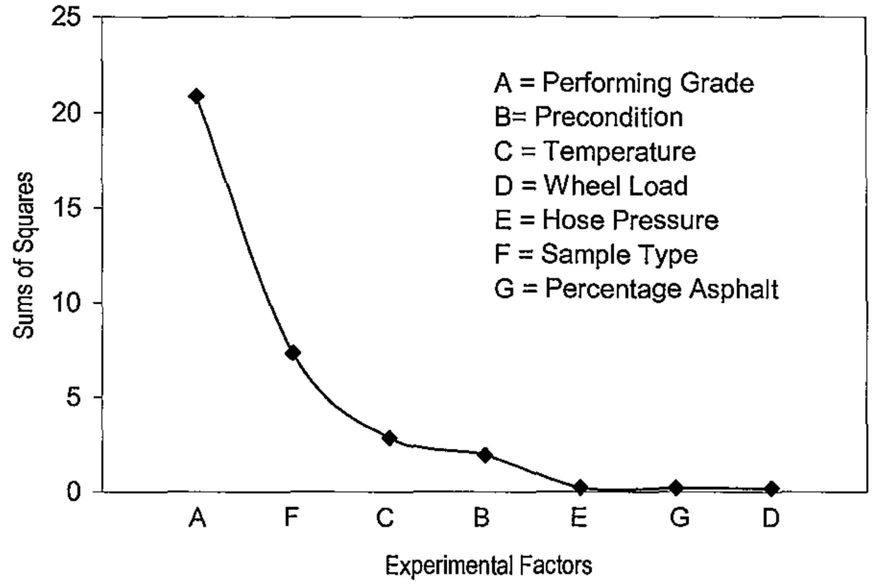


Figure 6.3 Significance Plot

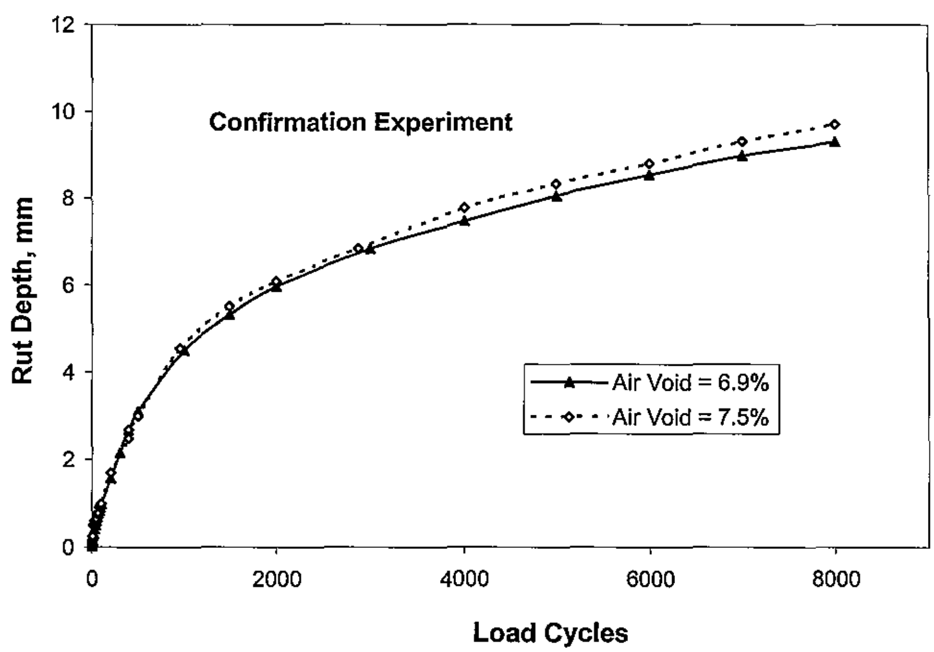


Figure 6.4 Typical Rut Depths versus Load Cycles in Confirmed Experiment

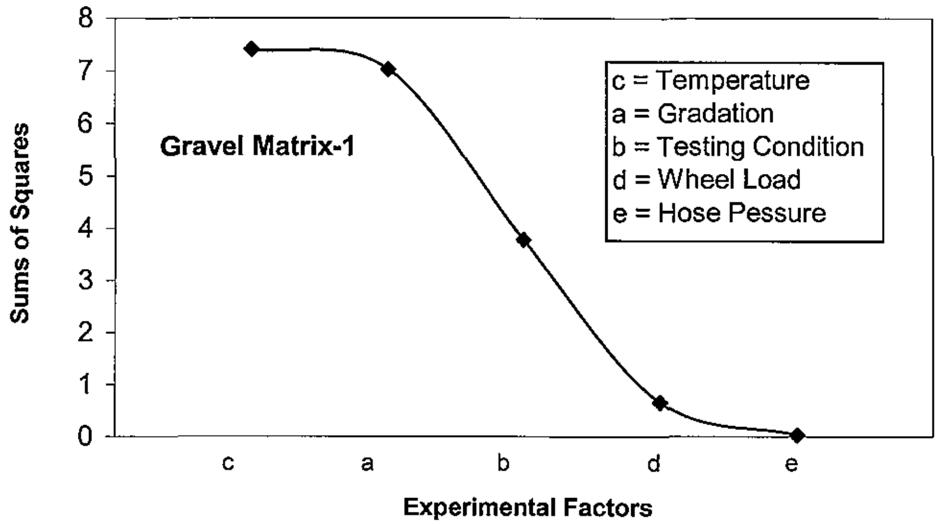


Figure 6.5 Significance Plot for Gravel in Matrix-1

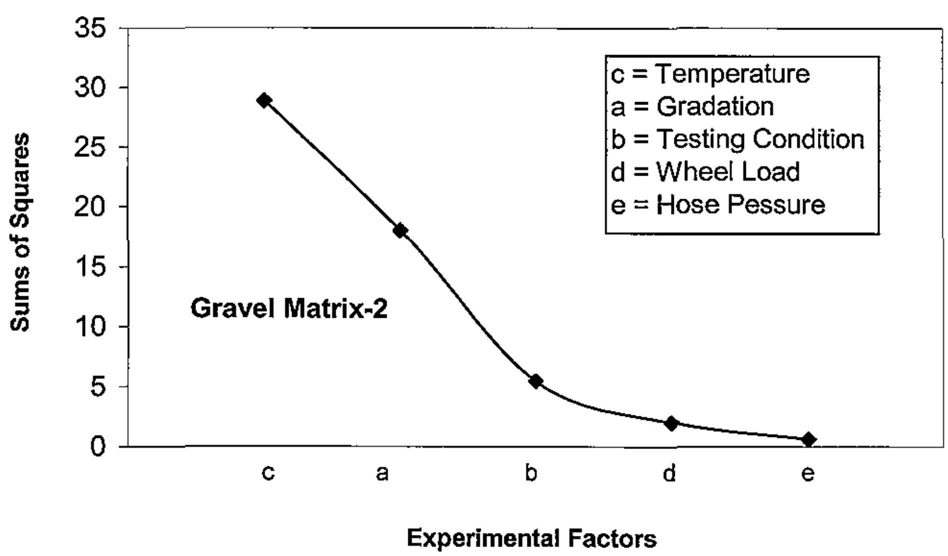


Figure 6.6 Significance Plot for Gravel in Matrix-2

Table 6.1a Factors Affecting Rutting

Traffic	Material	Environment
• Wheel load • Axel load • No of load repetitions • Tire pressure • Speed of vehicle • Lateral distribution of load • Wheel configuration	• Aggregate angularity, fractured face, gradation, type, specific gravity • Asphalt Cement grade, Type • Mix air void, asphalt content, dust content, VMA, VFA, compaction	• Temperatu re • Moisture • Frost • Water table

Table 6.1b Limestone Mix's Aggregate Information

Material	Source	Type	% Used
5/8" Chips	Western Rock at Davis, Oklahoma	Rhyolite	35
Screening	Western Rock at Davis, Oklahoma	Rhyolite	35
Shot	Dolese Co. at Davis, Oklahoma	Limestone	20
Sand	Dolese Co. at Oklahoma City, Oklahoma	Quartz	10

Table 6.2 Blended Aggregate Properties (Limestone Mix)

Properties	Measured	Required
L.A. Abrasion, % wear	23	40 Max.
Durability Index	74	40 Min.
Insoluble Residue (%)	68.7	40 Min.
Fractured Faces (%)	100	95/90 Min.
Sand Equivalent (%)	52	45 Min.
Fine Aggregate Angularity (%)	46	45 Min.
Specific Gravity (SSD)	2.639	
Absorption (%)	0.189	

Table 6.3 Volumetric Properties for Optimum Asphalt Content

Binder	Optimum AC	% Air at N_d	% VMA at N_d	% VFA at N_d	% G_{mm} at N_i	% G_{mm} at N_d
PG 70-28	5.4	4.0	14.2	72.0	88.8	96.0
PG 64-22	5.1	4.0	14.0	70.9	88.2	96.0
Superpave Requirement		4.0	14 min	65-76	Less than 89	96.0

Table 6.4 Factor and Levels

Factor Number	Factors	Level 1	Level 2
1	Binder's PG	PG 64-22	PG 70-28
2	Sample Conditioning	Dry	Wet
3	Temperature	60^0 C	64^0 C
4	Wheel Load	100 lb	110 lb
5	Hose Pressure	100 lb	110 lb
6	Specimen Type	SGC cylinder	AVC beam
7	Percentage Asphalt	5.1	5.4

Table 6.5 Test Matrixes

Trial Number	Grade	Conditioning	Temperature (^{0} C)	Wheel Load (lb)	Hose Pressure (psi)	Sample	% Asphalt	% Air
1	PG 64-22	Dry	60	110	110	Cylinder	5.1	7.5
2	PG 64-22	Dry	60	100	100	Beam	5.4	7.3
3	PG 64-22	Wet	64	110	110	Beam	5.4	7.2
4	PG 64-22	Wet	64	100	110	Cylinder	5.1	7.0
5	PG 70-28OK	Dry	64	110	100	Cylinder	5.4	6.3
6	PG 70-28OK	Dry	64	100	110	Beam	5.1	8.0
7	PG 70-28OK	Wet	60	110	100	Beam	5.1	7.5
8	PG 70-28OK	Wet	60	100	110	Cylinder	5.4	6.3

Table 6.6 Experimental Total and Average Rut Depth

Trial Number	Grade	Conditioning	Temperature	Wheel Load	Hose Pressure	Sample Type	% Asphalt	Average Rut Depth (mm)
Factors	A	B	C	D	E	F	G	
1	1	1	1	1	1	1	1	4.952
2	1	1	1	2	2	2	2	6.823
3	1	2	2	1	1	2	2	9.426
4	1	2	2	2	2	1	1	6.960
5	2	1	2	1	2	1	2	2.677
6	2	1	2	2	1	2	1	5.177
7	2	2	1	1	2	2	1	4.235
8	2	2	1	2	1	1	2	3.174
							Total =	43.4

Note: 1, 2 were factor levels (Table 4)

Table 6.7 Trial Combinations for Factor in an L_8 Array

Factor	Level 1	Level 2
A	1,2,3,4	5, 6, 7, 8
B	1,2, 5, 6	3,4,7,8
C	1,2,7,8	3,4,5,6
D	1,3,5,7	2,4,6,8
E	1,3,6,8	2,4,5,7
F	1,4,5,8	2,3,6,7
G	1,4,6,7	2,3,5,8

Table 6.8 Level Sums for Factor F at Level 1

Trial Number	Grade A	Conditioning B	Temperature (64^0 C) C	Wheel Load (Lb) D	Hose Pressure (psi) E	Sample Type F	% Asphalt G	Average Rut Depth (mm)
1	1	1	1	1	1	1	1	4.952
2	1	1	1	2	2	2	2	
3	1	2	2	1	1	2	2	
4	1	2	2	2	2	1	1	6.960
5	2	1	2	1	2	1	2	2.677
6	2	1	2	2	1	2	1	
7	2	2	1	1	2	2	1	
8	2	2	1	2	1	1	2	3.174
								Total = 17.8

Table 6.9 Level Sums Table

Factor	Level 1	Level 2	Total (level1+level2)
Performing Grade (A)	28.2	15.2	43.40
Pre-conditioning (B)	19.6	23.8	43.40
Temperature (C)	19.2	24.2	43.40
Wheel Load (D)	21.3	22.1	43.40
Hose Pressure (E)	22.7	20.7	43.40
Sample Type (F)	17.8	25.6	43.40
Percent Asphalt (G)	21.3	22.1	43.40

Table 6.10 Sums of the Squares Calculations

Factor	SS	Significance
Grade	20.86	Significant
Sample Type	7.34	
Temperature	2.86	
Conditioning	1.94	
Hose Pressure	0.23	Insignificant
Percentage Asphalt	0.19	
Wheel Load	0.19	
SS_{Total}	34.64	
Error	0.61	

Table 6.11 Calculations of Variance, F Statistic and Percent Contribution

Factor	df_x $(n_x\text{-}1)$	SS_x	V_x	F (V_x/V_{err}) (Statistics)	F $(1,3)_{0.05}$ (Table)	SS'_x	P
Grade	1	20.86	20.86	417.20	10.1	20.71	59.79
Sample type	1	7.34	7.34	146.80	10.1	7.19	20.76
Temperature	1	2.86	2.86	57.20	10.1	2.71	7.82
Conditioning	1	1.94	1.94	38.80	10.1	1.79	5.17
Error (err)	3	0.15	0.05			Sum =	93.53
SS_{Total}	7	34.64	4.95				

Table 6.12 Parameters for Calculation of Predicted Results

Factor	Significance Level	Level Sum	Level Sum response
Grade	1	28.20	7.05
Sample Type	2	25.60	6.4
Temperature	2	24.20	6.05
Sample Conditioning	2	23.80	5.95
Total			43.4

Estimated Mean Response = 9.18

Table 6.13 Test Matrix-1 for Gravel Mixes

Trial Number	Asphalt Content	BRZ/TRZ	Condition	Temperature	Load	Hose	Air	8000
1	4.5	BRZ	Dry	60	100	100	6.2	6.0
2	4.5	BRZ	Dry	64	110	110	8.7	7.1
3	5.5	BRZ	Wet	60	100	110	5.5	7.6
4	5.5	BRZ	Wet	64	110	100	5.5	11.4
5	4.3	TRZ	Wet	60	110	100	7.4	8.3
6	4.3	TRZ	Wet	64	100	110	7.1	11.3
7	5.3	TRZ	Dry	60	110	110	6.1	10.1
8	5.3	TRZ	Dry	64	100	100	7.0	9.9

Table 6.14 Test Matrix-2 for Gravel Mixes

Trial Number	Asphalt Content	TRZ/ARZ	Condition	Temperature	Load	Hose	Air	8000
1	4.9	BRZ	Dry	62	100	100	6.1	4.8
2	4.9	BRZ	Dry	66	110	110	7.7	10.6
3	5.0	BRZ	Wet	62	100	110	7.9	5.3
4	5.0	BRZ	Wet	66	110	100	6.7	9.1
5	4.5	TRZ	Wet	62	110	100	7.6	7.6
6	4.5	TRZ	Wet	66	100	110	7.9	10.5
7	4.8	TRZ	Dry	62	110	110	6.3	10.5
8	4.8	TRZ	Dry	66	100	100	6.3	13.2

Table 6.15 Significant Parameters in Gravel Mixes

Factor	Significance
Gradation Temperature Conditioning	Significant
Hose Pressure Wheel Load	Insignificant

CHAPTER 7

REPEATABILITY AND REPRODUCIBILITY

7.1 General

An identical result cannot obtain from the tests performed under presumably identical circumstances. The differences in results were due to unavoidable random errors inherent in every test procedure. In other words, the factors that influence the outcome of a test cannot all be completely controlled. For practical interpretation of test results, this inherent variability must be accounted for. Several factors may contribute to variability associated with the application of a test method. They include, the operator, the equipment, equipment calibration; and the environment.

An inter-laboratory study was undertaken to determine whether the data collected were adequately consistent, to investigate data considered to be inconsistent and to verify precision statistics. In the case of THE test procedure, the primary factor of concern was the sample preparation at a target level of air void. Other factors such as temperature, wheel load, and tire pressure could be controlled by proper calibration. A measure of the greatest difference between two test results would be considered acceptable when properly conducted repetitive determination were made on the same material by a competent operator. This was defined as "repeatability" or within laboratory precision (ASTM 670). It was the square root of the pooled average of within laboratory variances.

'Reproducibility' was a measure of the greatest difference between two tests. The tests were usually made by two different operators in different laboratories on portions of a material that were identical, or nearly identical as possible. Repeatability would be considered acceptable when the difference in test results was negligible. The reproducibility was the square root of the pooled average of between laboratory variances. The fundamental statistics underlying repeatability and reproducibility was the standard deviation (one sigma limit, 1s or difference two-sigma limit, d2s) of the population of measurements. In some cases, it was appropriate to use the coefficient of variation in place of the standard deviation as the fundamental statistic. The results of two properly conducted tests from two different laboratories on samples of same material should not change the value obtained from multiplying 1s or d2s by 2.828 (ASTM C 670).

7.2 Outlier

An outlier can be defined as discarding individual test results that appear to differ by suspiciously large amounts from the others. However, discarding of suspicious test results should be avoided unless there is a clear physical evidence to consider the result faulty. In particular, laboratories should be asked to report all results in their proper place and include notes describing the conditions surrounding those results that were suspected of being faulty. Sometimes if a test really went wrong, a laboratory should discard the results and repeat the test. Tests should not be repeated, however, just because the results don't look good. The consistency statistics generated through the method may assist in the detection of outlying data (ASTM E691). For a single APA rut test, there were 3 sets of rut results from six samples. An outlier was imposed to these 3 sets according to OHD L-43 method. If the difference between any set and average of the set divided by the standard deviation of that set exceeds 1.155 then the result of that particular set was rejected.

7.3 Test Results

One of the limestone aggregates (T.J.Campbell materials) was used for a variability study. It was an aggregate batched from OU laboratory. The designed optimum asphalt content 5.1% was set by the design laboratory. Batching was not performed from both laboratories because it might produce a number of variables for the limited number of mixes. However, mixing was performed in both the OU laboratory as well as the ODOT laboratory. A total of 24 samples were prepared; half of them were prepared in the OU laboratory and half of them were prepared in ODOT's laboratory. Four combinations of samples were tested, namely, OU-ODOT, OU-OU, ODOT-OU, ODOT-ODOT for packing purposes. Half of the samples prepared at OU were tested at ODOT (OU-ODOT) and another half was tested at OU (OU-OU). Similarly, half of the samples prepared at ODOT were tested at OU (ODOT-OU) and another half was tested at ODOT (ODOT-ODOT). The test results were plotted in Figure 7.1.

It can be seen that one result (average of two samples) for the case of (OU-OU) with air void of 6.9 % showed higher rut depth. Similarly, one result (average of two samples) for the case of (ODOT-ODOT) with air void of 7.5 % showed higher rut depth. A sample calculation for outlier was shown in Table 7.1. The critical value for student test (T-statistic) was taken as 1.155. If the calculated t-statistic value was greater or equal to this value, then one chance in one hundred the value was from the same population (OHD L-43, 2001). No set was rejected as an outlier for all combination of tests.

7.4 Data Analysis

Table 7.2 shows that the results between and within analysis for the various samples tested. The table shows the average and standard deviation for each combination tested. It was evident that the results of samples prepared at OU and tested at ODOT (combination, OU-ODOT) differ radically when compared to the other combinations. The combination OU-ODOT had 10 times the second highest variance. Therefore, the data obtained from this combination was excluded. Therefore, outlier applied in OHD 43 has to be reinvestigated. Table 7.2 also shows one sigma limit (1s) or coefficient of variation, which was an indication of variability.

The value of repeatability (1s%) within laboratory ranges from 2.6 to 5.5. Therefore, results of two properly conducted tests by the same operator on the same material should not differ by more than 7% to 15% (second last column of Table 7.2). The multilaboratory coefficient of variation had been found to be 15% to 45%. The results of two different laboratories differ from each other by more than 45% of the average.

7.5 Conclusion

The APA induced rutting at OU was compared to the APA induced rutting at ODOT on the SGC samples representing common HMA designs. It was evident that the actual variability in measured rutting seemed to be a function of variability in air voids for the sample set. Results generated with the APA were actually more consistent when test specimens were compacted to uniform air voids. Essentially, there were no significant difference in final rut depths obtained from the OU and the ODOT laboratory. It was found that the test results were repeatable and reproducible.

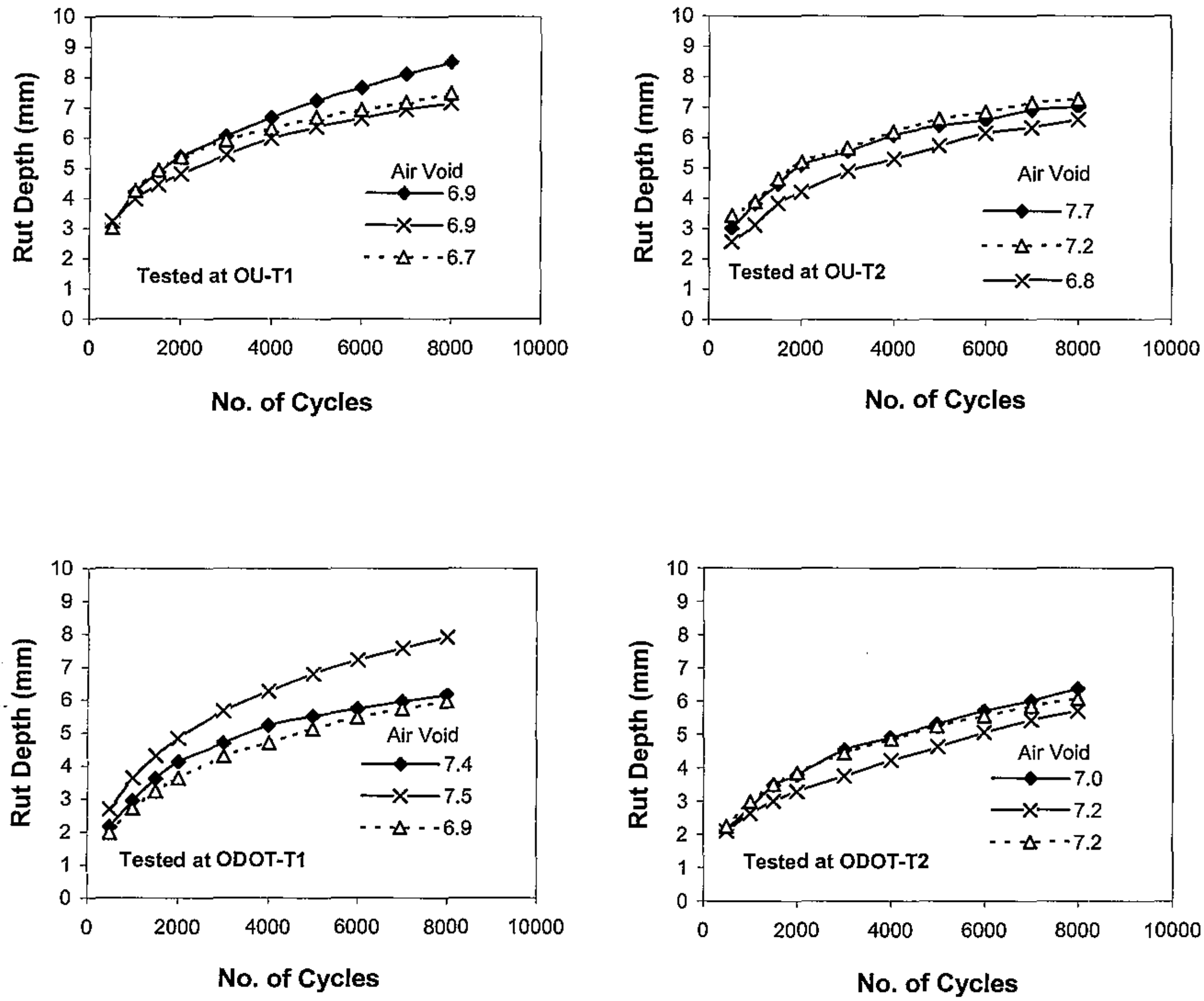


Figure 7.1 Rut Depths Versus Number of Cycles

Table 7.1 Outlier for Rut Depth Calculation

Sample	Rut (mm)	m-values	Outlier	Average Rut (mm)
1	8.5033	1.124	8.5033	
2	7.1522	0.791	7.152	7.71
3	7.4755	0.333	7.4755	
Average	7.710	Note: m= (x-average)/stdev		
Standard Deviation	0.705	If m> 1.155 then throw		

Table 7.2 Between and Within Analysis for Rut Tests

Within Laboratory	Specimen No. 1	Specimen No. 2	Specimen No. 3	Average	Standard Deviation	Variance	Standard Deviation	1s%
OU-OU	7.503	7.152	7.475	7.377	0.195	0.038	7.484	2.644
ODOT-ODOT	6.371	5.699	6.074	6.048	0.337	0.113	15.757	5.568
ODOT-OU	7.012	7.265	6.596	6.958	0.338	0.114	13.740	4.855
OU-ODOT	6.162	7.92	5.961	6.681	1.078	1.161	45.650	16.131

Note: OU-OU means sample prepared at OU and tested at OU
OU-ODOT means sample prepared at OU but tested at ODOT
Average = sum of n tests results for a particular combination divided by the specimen number
Variance = sum of the squares of n test results for a particular combination minus n times the square of the average for that combination, divided by one less than the number of replicate test results
1s% = (Standard Deviation x 100)/Average

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This study evaluated rutting potential of Hot Mix Asphalt (HMA) concrete by laboratory predicted value of rut depth. The APA in conjunction with the SGC was capable of determining the rutting potential of HMA. Rutting is a complex phenomenon, as evident from the literature. Many variables contribute to rutting and no one variable can adequately predict rutting. Much of the rutting can be attributed to improper mix design (mix gradation, binder grade and content, amount of filler material, aggregate shape and texture). Temperatures play a significant rule in rutting in HMA. Each of these variables was considered in evaluating the rutting potential of HMA mixes. A series of tests were conducted considering the practical ranges of properties such as aggregate size, type, shape, texture, binder grade, mix gradation, density and temperature, etc. The tested data was analyzed using correlation analysis, linear regression analysis methods, and stepwise multiple variable analysis methods.

The parameters, which have the greatest influence on rutting, were categorized. The laboratory testing suggested criterions for rank a HMA mix as poor, fair or good depending on the rutting magnitude. Binder's performance was evaluated by the corresponding mix rut performance. A linear and non-linear statistical model was developed for rut prediction. Nonlinear model showed better prediction compared to the linear model. The issue of repeatability and reproducibility was analyzed. The APA test showed almost no variability between OODT and OU laboratory. The study developed a database for future model development.

8.2 Recommendations

Considering the complexity of the rutting problem, from the viewpoint of physics and mechanics involved, this study developed regression models based on laboratory test results. A simple model may lacks from capturing all the fundamental behavior of HMA pavement with sufficient accuracy. Advanced models are need for a realistic assessment of material properties and predict rutting. It is recommended that neuron-based model will be an educated approach for including numerous parameters involved in the rutting of HMA.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), (1999). "Sieve Analysis of Fine and Coarse Aggregates," AASHTO T 27, AASHTO *Standard Specifications for Transportation Materials*, Part II, Washington, D.C.

AASHTO MP1-98, (2000). "Standard Specification for Performance Graded Asphalt Binders," *AASHTO Provisional Standards*, Washington, D.C.

AASHTO PP3-00, (1998), "Standard Practice for Superpave Volumetric Design for Hot-Mix Asphalt (HMA)," *AASHTO Provisional Standards*, Washington, D.C.

AASHTO T 166, (1999). "Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens." AASHTO *Standard Specifications for Transportation Materials*, Part II, Washington, D.C.

AASHTO T 209, (1999). "Maximum Specific Gravity of Bituminous Paving Mixtures" AASHTO *Standard Specifications for Transportation Materials*, Part II, Washington, D.C.

AASHTO T 269, (1999). "Standard Test Method for Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures." AASHTO *Standard Specifications for Transportation Materials*, Part II, Washington, D.C.

AASHTO T 283, (1996). "Resistance of Compacted Bituminous Mix to Moisture Induced Damage" AASHTO *Standard Specifications for Transportation Materials*, Part II, Washington, D.C.

AASHTO TP1-98, (2000). "Method for Determining the Flexural Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR), AASHTO *Provisional Standards*, Washington, D.C.

AASHTO TP3-00, (2000). "Method for Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT)", *AASHTO Provisional Standards*, Washington, D.C.

AASHTO TP5-98, (2000). "Method for Determining the Rheological Properties of Asphalt Binder Using Dynamic Shear Rheometer (DSR)", *AASHTO Provisional Standards*, Washington, D.C.

AASHTO TP7-98, (2000). "Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt (HMA) Using the Simple Shear Test (SST) Device, *AASHTO Provisional Standards,* Washington, D.C.

AASHTO TP9-96, (2000). "Method for Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device", *AASHTO Provisional Standards,* Washington, D.C.

American Society for Testing and Materials (ASTM), (1999). "Standard Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials", ASTM C 670, Vol. 03.03, *Annual Book of Standards*, Philadelphia, PA.

ASTM D 3381, (1998). "Standard Specification for Viscosity-Graded Asphalt Cement for use in Pavement Construction", *Annual Book of Standards*, Vol. 04.03, Philadelphia, PA.

ASTM D 946, (1998). "Standard Specification for Penetration Graded Asphalt Cement in Pavement Construction", *Annual Book of Standards*, Vol. 04.03, Philadelphia, PA.

Anderson, D.A., Christensen, D.W. and Bahia, H., (1991). "Properties of Asphalt Cement and the Development of Performance Related Specifications," Proceedings of the *Association of Asphalt Paving Technologists*, vol. 60, pp. 437-475.

Bahia U. H., Zhai H., Bonnetti K., and Kose. S., (1999) "Non-Linear Viscoelastic and Fatigue Properties of Asphalt Binders," Proceedings of the *Association of Asphalt Paving Technologists*, AAPT, Vol. 68, pp.1-34.

Barksdale, R.D., (1993). "Test Device for Evaluating Rutting of Asphalt Concrete Mixes," *Transportation Research Record 1418,* National Research Council, Washington, D.C.

Bonaquist, R. F., and Witczak, M. W., (1997), "A Comprehensive Constitutive Model for Granualr Materials in Flexible Pavement Structures." *Proceedings of Eighth International Conference on Asphalt Pavements,* Volume 1, August 10-14, Seattle, Washington, pp.783-802.

Brock J.D., Collins R., and Lynn C., (1999). "Performance Related Testing with Asphalt Pavement Analyzer," Technical Paper T-137, *Pavement Technology Inc.* (PTI).

Brown, E. R. and Cross, S. A., (1989). "A study of in-place rutting of asphalt pavements", *Proceedings Association of Asphalt Paving Technologists*, AAPT, Volume 58, 1-39.

Brown, E.R., and Cross, S. A., (1992). "A National Study of Rutting in Asphalt Pavement," Proceedings of the *Association of Asphalt Paving Technologists*, AAPT, Vol. 61.

Button, J. (1990). "Influence of Aggregates on Rutting in Asphalt Concrete Pavements." *Transportation Research Record 1259,* National Research Council, Washington D.C.

Chen, D.H., and Lin, (1998). "Predictive Equation for Permanent Deformation," *Texas Department of Transportation*, TX, pp. 1-26.

Choubane, B., G. C. Page, and J. A. Musselman (1998). "Investigation of the Asphalt Pavement Analyzer for Predicting Pavement Rutting", *Research Report FL/DOT/SMO/98-427*, Florida Department of Transportation.

Collins, R., Watson, D.E., and Cambell, B., (1995). "Development and Use of Georgia Loaded Wheel Tester," *TRB*, No. 1492, *National Research Council*, Washington, D.C.

Cooley L. A., Kandhal, P S., and Buchanan, M. S., (2001). "Loaded Wheel Testers in the United States: State of the Practice," *National Center for Asphalt Technology*, Auburn University, Alabama, http://www.nas.edu/trb/publications/

Cooley, L.A. and Kandhal, P.S., (1999). " Evaluation of Density Gradients in the APA Samples," *National Center for Asphalt Technology*, Auburn University, Alabama.

Corté, J. F., Y. Brosseaud, J. P. Simoncelli, and G. Caroff. Investigation of Rutting of Asphalt Surface Layers: Influence of Binder and Axle Load Configuration. In *Transportation Research Record 1436,* TRB, National Research Council, Washington, D.C., Oct. 1994, pp. 28-37.

Cross, S.A. and Brown, E. R. (1992). "Selection of Aggregate Properties to minimize Rutting of Heavy Duty Pavement," *ASTM Technical Publication Symposium on Effect of Aggregate and Mineral Fillers on Asphalt Mix Performance*, Dec. 10, No. 1147, San Diego, CA.

Finn, F.N., Saraf, C.L., Kulkarni, R., Nair, K., Smith, W., and Abdullah, A., (1986). "Development of Pavement Structural Subsystem," *The National Cooperative Highway Research Program* (NCHRP), Report No. 291.

Ford M.C., (1988). "Pavement Densification related to Asphalt Mix Characteristics," 67th *Annual Transportation Research Board Meeting*.

Gillespie, T.D., Karamihas, S. M., Sayers, M. W., Nasim, M. A., Hansen, W., and Ehsan, N., (1993). "Effect of Heavy-vehicle Characteristics on Pavement Response and Performance." *National Highway Cooperative Research Program*, Report 353, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C.

Gramling, W. L., Suzuki, G. S., and Hunt, J. E., (1991). "Rational Approach to Cross-Profile and Rut Depth Analysis," *Transportation Research Record,* No. 1311, National Research Council, Washington, D.C., pp. 173-179.

Groenendijk, J., Vogelzang, C. H., Miradi, A., Molenaar, A. A., and Dohmen, L. J. M., (1996). "Rutting Development in Linear Tracking Test Pavements to Evaluate Shell Subgrade Strain Criterion," *Transportation Research Record 1570,* National Research Council, Washington, D.C., pp.23-29.

Harvey, J., Lee, T., Sousa, J., Pak, J., and Monismith, C. L., (1995). "Evaluation of Fatigue and Permanent Deformation Properties of Several Asphalt-Aggregate Field Mixes Using Strategic highway Research Program A-003A Equipment," *Transportation Research Record 1454,* National Research Council, Washington, D.C., pp.123-133.

Hudson, S.W. and S.B. Seeds, (1988). "Evaluation of Increased Pavement Loading and Tire Pressures," *Transportation Research Record* 1207, Transportation Research Board, Washington, D.C., pp. 197-206.

Isacsson U. and Lu X., (1999). "Laboratory Investigation of Polymer Modified Bitumens," *Journal of the Association of Asphalt Paving Technologists*, AAPT, 1999, Vol. 68, pp.35-63.

Jackson, N.M. and Ownbby, E.A., (1998). "Evaluation of Laboratory Compaction of HMA," *Transportation Center,* The University of Tennessee, Final Repot.

Kandhal, P.S. and Mallick, R.B., (1999). "Evaluation of Asphalt Pavement Analyzer for HMA Mix Design," *National Center for Asphalt Technology,* Report No. 99-4, Auburn University, Alabama.

Kandhal, P.S., Cross, S.A., and Brown, E.R., (1993). "Heavy-Duty Asphalt Pavements in Pennsylvania: Evaluation for Rutting," *Transportation Research Record 1384,* National Research Council, Washington, D.C., pp. 49-58.

Kim, O. and C.A. Bell, (1988). "Measurement and Analysis of Truck Tire Pressures in

Kyle B. (1995). "Successful Industrial Experimentation", *VCH Publishers Inc.,* ISBN 1-56081-050-5, pp. 1-131.

Lai J.S., (1996). Development of a Simplified Test Method to Predict Rutting Characteristics of Asphalt Mixes," Final Report, Georgia DOT Project No. 8503.

Lai, J. S. (1986). "Evaluation of Rutting Characteristics of Asphalt Mixes Using Loaded Wheel Tester", Project No. 8609, Georgia Department of Transportation.

Lekarp, F., Richard, I. R., and Dawson, A. (1996). "Influences on Permanent Deformation Behavior of Unbound Granular Materials." *Transportation Research Record 1547,* National Research Council, Washington D.C. pp.68-75.

Majidzadeh, K., Bayomy, F., and Khedr, S., (1978). " Rutting evaluation of Subgrade Soil in Ohio," *Transportation Research Record,* No. 671, TRB, pp. 75-84.

Masad, E., B. Muhunthan, N. Shashidhar, and T. Harman (1999). "Quantifying Laboratory Compaction Effects on the Internal Structure of Asphalt Concrete," *Transportation Research Record: Journal of the Transportation Research Board No. 1681,* TRB, National Research Council, pp. 179-185.

Memorandum to the Asphalt Pavement User Group and new APA Owners. Georgia, Final Report, May 1999.

Middleton, D.R., F.L. Roberts and T. Chira-Chavala, (1986). "Measurements and Analysis of Truck Tire Pressures on Texas Highways," *Transportation Research Record* 1070, Transportation Research Board, Washington, D.C., pp. 1-8.

Miller, T., K. Ksaibati, and M. Farrar, (1995). "Utilizing the Georgia Loaded-Wheel Tester to Predict Rutting", Presented at the *74th Annual Meeting of the Transportation Research Board*, Washington, D.C.

Musselman, J.A., Couhaane, B., Page, G.C. and Upshaw, P.B. (1998). "Superpave Field Implementation: Florida's Early Experience," *Transportation Research Record* 1609, National Research Council, Washington D.C. pp. 51-60.

National Asphalt Pavement Association (NAPA), (1995). "An Industry Discussion on Superpave Implementation," *NAPA Special Report 174*, National Asphalt Paving Association.

Natu, G.S and Tayebali, A. A., (1999). " Mixture Design and Accelerated Laboratory Performance Evaluation of Unmodified and Crumb Rubber Modified Mixes," *Journal of the Association of Asphalt Paving Technologists*, AAPT, Vol. 68, pp.193-221.

Oklahoma Department of Transportation (ODOT), (1999). "Special Provision for Plant Mix Bituminous Bases And Surfaces", Section 708-1(a), *The Standard Specifications for Highway Construction*, 1999 Edition, Oklahoma City, Oklahoma.

Oklahoma Highway Department (OHD), (2001). "Method of Test for Determining Rutting Susceptibility Using the Asphalt Pavement Analyzer", Oklahoma Highway Department Laboratory Test No. 43 (OHD L 43), Oklahoma City, Oklahoma.

Pidwerbesky, B.D., Steven, B.D., and Arnold, G., (1997). "Subgrade Strain Criterion for Limiting Rutting in Asphalt Pavements," *Proceedings of Eighth International Conference on Asphalt Pavements*, Volume 2, August 10-14, Seattle, Washington, pp.1529-1544.

Ramsamooj, D.V., Ramadan, J., and Lin, G.S., (1998). "Model Prediction of Rutting in Asphalt Concrete," *Journal of Transportation Engineering, Vol. 124, No. 5,* September-October, American Society of Civil Engineers, pp. 448-456.

Reese, R. E. and Goodrich, J. L., (1993). "California Desert Test Road – A step Closer to Performance Based Specifications," AAPT, vol. 62, pp. 247

Roberts, F. L., Kandhal, P. S., and Brown, E. R., (1996). "Hot Mix Asphalt Materials, Mixture Design, and Construction." *NAPA Education Foundation*, Lanhamn Maryland, 1996.

Sherif, A., El-Samny, M. K., Zahw, M., and El Halem, A. O. (1997). "Laboratory and Statistical Evaluation of the Influence of Mix parameters on Surface Rutting of Asphalt Pavement," *Proceedings of the 1997 Annual Canadian Society for Civil Engineering,* Sherbrooke, Canada, pp. 351-362.

Simpson, A.L., Daleiden, J.F. and Hadley, W.O., (1995). "Rutting Analysis From a Different Perspective," *Transportation Research Record 1473*, National Research Council, Washington, D.C., pp.9-16.

Sousa, J., and Solaimanian, M., (1994). "Abridged Procedure to Determine Permanent Deformation of Asphalt Concrete Pavements," Presented at the *73rd Annual Meeting of the Transportation Research Board*, Washington, D. C.

Sousa, J., Weissman, W.S., Sackman, J.L., and Monismith, C.L., (1992). "Nonlinear Elastic Viscous with Damage Model to Predict Permanent Deformation of Asphalt Concrete Mixes," *Transportation Research Record 1384,* National Research Council, Washington, D.C., pp.80-93.

Van de Loo, P. J., (1978). "The Creep Test: A Key Tool in Asphalt Mix Design and in the Predcition of Pavement Rutting," *Proceedings, Association of Asphalt Paving Technologists*, Vol. 47, pp. 522-554.

Von Quintus, H. L., (1991). "Asphalt-Aggregate Mixture Analysis System: AAMAS," *NCHRP Report 338, Transportation Research Board,* National Research Council, Washington, D.C.

West, R. C., (1999). "A Rugged Study of the Asphalt Pavement Analyzer Rutting Test," *Memorandum to the Asphalt Pavement User Group and new APA Owners*. Georgia, Final Report, May 1999.

West, R. C., G. C. Page, and K. H. Murphy (1991). "Evaluation of the Loaded Wheel Tester", *Research Report FL/DOT/SMO/91-391*, Florida Department of Transportation, Florida.

Williams, C. R. and B. D. Prowell (1999). "Comparison of Laboratory Wheel-Tracking Test Results to WesTrack Performance", Presented at the 78th Annual Meeting of the *Transportation Research Board,* Washington, D.C.

Zaghloul, S., and White, T., (1994). "Use of Three Dimensional, Dynamic finite Element Program for Analysis of Flexible Pavement," *Transportation Research Record 1388,* National Research Council, Washington D.C. pp.60-69.