Development of Laboratory Performance Test Procedures and Trial Specifications for Hot Mix Asphalt: Final Report

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16. Abstract The implementation of Superpave mix design procedures in 1997 has continued throughout the United States, and has gained general acceptance by most State Departments of Transportation. The Michigan Department of Transportation has successfully implemented Superpave mix design procedures for all mainline paving, inline with the national trend. However there are some unresolved issues with the new mix design procedures, such as the identification and use of performance test criteria to accompany the volumetric mix design criteria. Other evolutionary issues with regard to specifications, such as quality control and quality assurance testing and performance-related specifications, are also receiving acknowledgment as viable techniques to address dwindling experience in the industry. This report summarizes the activities of all the phases of research working towards development of performance testing criteria for HMA mix design as well as construction specifications.			
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EXECUTIVE SUMMARY

Introduction

This report describes the establishment of a laboratory performance test specification and field specification in order to contribute to the asphalt materials technology in Michigan. Characterization of materials, performance testing of specimens, and statistical analyses will be used in developing specifications. These more advanced, science-based specifications should significantly improve the qualities of designed and constructed hot mix asphalt leading to improved service life for flexible pavements in Michigan.

The objectives of this study include the following: 1. field sampling of mixtures throughout Michigan, 2. characterization of materials sampled, 3. development of laboratory performance test criteria to accompany existing Superpave mix design criteria, 4. development of field specifications for acceptance of hot mix asphalt. This study involved both laboratory and field work. The following tests and evaluations were done on the laboratory prepared and field specimens:

- i.) Gradation analyses.
- ii.) Volumetric determinations and analyses of mixes (bulk specific gravity, maximum theoretical specific gravity, binder content, air voids, voids in the mineral aggregate, voids filled with asphalt, etc.).
- iii.) Asphalt binder characterization using Superpave binder tests (rotational viscosity, dynamic shear rheometer, bending beam rheometer, direct tension test, and aging of binders using a rolling thin film oven and pressure aging vessel).

iv.) Performance testing of mixture specimens (Asphalt Pavement Analyzer (APA) and Four-Point Beam Fatigue (FPBF)).

v.) Development of a pay factor system based on the APA and FPBF apparatus.

Statistically sufficient testing will be done in support of analyses of test materials.

An extensive literature review was done on field sampling, the characterization of asphalt materials, and pay factor development. The major conclusions from this study include are intended to support the following:

- Implementation of quality control/quality assurance (QC/QA) specifications. The QC/QA approach is a statistical approach to ensure the end product is of acceptable quality, but is heavily dependent upon sampling and corresponding test results.
- Performance-related specifications or performance warranties are anticipated in the future.
- Sampling can take on two forms: Truck sampling and/or roadway sampling from behind the paver.
- These two sampling techniques can be compared with statistics to see if there is any significant difference between the two techniques.
- Common errors in sampling include:
    1. Obtaining the entire sample from a single location.
    2. Not removing all the material from within a template.
    3. Contaminating the sample with underlying material.
    4. Segregating the material while sampling

- Performance testing of HMA materials sampled include asphalt pavement analyzer and four point beam fatigue testing.
- These tests are important in determining the rutting and fatigue cracking susceptibility of hot mix asphalt.
- Various models have been developed based on the above testing techniques.
- These models can be used with cost models in order to determine pay factors for the appropriate measurable or calculated material properties.

Verification of Mix Designs

Verification of the hot mix asphalt (HMA) mixture designs was undertaken for future use in performance testing of the laboratory designs as well as controlled variations in asphalt binder content and percent air voids. The Michigan Superpave mix design criteria were used for verification. Verification of the aggregate stockpiles were done in accordance with AASHTO T 27: Sieve Analysis of Fine and Coarse Aggregate and AASHTO T11: Materials Finer than 75µm (No. 200) Sieve in Mineral Aggregates. The Superpave mix design process was used to verify the submitted job mix formula's (JMF's). In conducting the Superpave mix designs, the following AASHTO test specifications were used: Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor: AASHTO T 312, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens (Method A): AASHTO T 166-00, and Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures: AASHTO T 209-99.

The volumetric calculations of voids in the mineral aggregate (VMA) and effective binder contents (P_{be}) used the aggregate specific gravity design values submitted in the JMF to MDOT. The assumed aggregate specific gravity values also are then translated into the calculation of voids filled with asphalt (VFA) as this calculation is dependent upon VMA.

Using the MDOT mix design verification tolerance of +/-0.3 within the submitted JMF, 16 of the 20 mix designs were verified with respect to the optimum binder content. The four projects that were outside the binder content tolerance were:

M-52, St. Charles: 5.5% JMF vs. 6.0% MTU,

M-50, Brooklyn: 6.8% JMF vs. 7.2% MTU,

I-75, Grayling: 6.3% JMF vs. 5.8% MTU, and

M-43, Lansing: 6.0% JMF vs. 6.5% MTU.

One important note to mention is that the mix design oven temperatures for the M-43 project were adjusted to reflect that a polymer modified asphalt binder was used based on the rotational viscosity test results. This adjustment resulted in the compaction temperature being 162°C rather than the stated 157°C on the MDOT 1911 form for this mixture.

Several mix designs were attempted for the US-31 (Elk Rapids) project and in all cases the aggregate structure was relatively insensitive to changes in asphalt binder content. Optimum binder content for this aggregate structure could not be obtained at 4% air voids was not obtained even when 0.5% more binder was added than the reported JMF optimum.

Examining the voids in the mineral aggregate (VMA) criteria, there are four mixes that do not meet the minimum VMA criteria. The following three projects failed to meet the minimum mix design VMA criteria:

> 8-Mile Road, Warren: 15.3% JMF (14.0% Design Minimum) vs. 13.9% MTU,
>
> I-94 (4E30), Ann Arbor: 14.7% JMF (14.0% Design Minimum) vs. 13.7% MTU,
>
> and I-94 (3E30), Ann Arbor: 13.7% JMF (13.0% Design Minimum) vs. 12.9% MTU.

The 0.2% drop in the MTU binder content from the JMF for the 3E30 Ann Arbor mix did place the VMA criteria out of specification.

The voids filled with asphalt (VFA) criteria have a range of 65-78% and 65-75% for E3 mixes and E10 and above mixes, respectively. Only the gap-graded Superpave mix on I-94 in Ann Arbor (78.1%) and the 5E30 mix on I-75 in Flint (78.2%) were out of specification. It is important to note that the gap-graded Superpave mix design was accepted with a VFA of 76.8%. The remaining mixes were all within the design specification ranges.

Characterization of Asphalt Binder

Asphalt binders were obtained from three sampling locations: the tank at the plants, recovered from HMA samples, and recovered from paver HMA samples. Aging processes of tank binders, rolling thin film oven and pressure aging vessel, were done for comparisons with recovered truck and paver samples. The recovered paver samples were also aged using a pressure aging vessel for comparison to tanks samples at similar simulated ages. The methods for recovering and testing all samples were the same to minimize statistical error.

Michigan Tech test results show that eight of the twenty tank binders did not meet their design grade. Upon further examination seven of the eight failing binders were not being met on the low temperature side and only one on the high temperature side.

Overall, it was generally found that the recovered binder properties of paver samples are more representative of the corresponding aged tank binder than truck samples.

Hot Mix Asphalt and Aggregate Characterization of Truck and Paver Samples

The characterization of HMA and all of its constituents were done on truck and paver samples. The constituents of the HMA analyzed were aggregate and asphalt binder. The aggregate and HMA characterization is summarized in this section, while the asphalt binder characterization was summarized previously above.

Characterization of HMA Samples

The characterization of the HMA and aggregate properties were done in accordance with AASHTO test specifications except for the crushed particle count for coarse aggregate. The test specifications followed were:

- Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens (Method A): AASHTO T 166-00,
- Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures: AASHTO T 209-99,
- Quantitative Extraction of Bitumen from Bituminous Paving Mixtures (Method A): AASHTO T 164-97,
- Recovery of Asphalt from Solution by Abson Method: AASHTO T 170-00,
- Mechanical Analysis of Extracted Aggregate: AASHTO T 30-93,

- Uncompacted Void Content of Fine Aggregate: AASHTO T 304-96, and
- Determining the Percentage of Crushed Fragments in Gravels: Pennsylvania DOT Test Method 621.

The volumetric calculations of voids in the mineral aggregate (VMA) and effective binder contents (P_{be}) used the aggregate specific gravity design values. The assumed aggregate specific gravity values also are then translated into the calculation of voids filled with asphalt (VFA) as this calculation is dependent upon VMA.

Comparison of Field Samples Characteristics to Design Values

Comparison of aggregate and mixture characteristics from the two field sampling locations were made as well as comparison of field to design characteristics. Specifically, aggregate and HMA characteristics of truck and paver samples were compared to determine whether or not there is a relationship between the two. The two sampling locations' characteristics are also compared to design characteristics and examined with the specification tolerances. The comparisons were for: asphalt binder content, aggregate characteristics, and HMA volumetrics. The reason for differentiating the asphalt binder content from the HMA volumetric comparisons is based on the binder content being a measurement, whereas the other volumetric properties are calculations based on other measurements. Furthermore the discussion follows the steps in which the materials were processed, e.g., binder content determination, gradation and aggregate characteristic measurements, and volumetric calculations.

Asphalt Pavement Analyzer Rut Prediction Model and Results

The APA is an empirical performance test used to evaluate the rutting susceptibility of HMA. The value measured, the rut depth, cannot be used as a basis for a mechanistic model. In the past, the APA rut depth at 8000 cycles has been used as to identify rut-prone HMA mixtures before they are used in the field. This is done by establishing a pass/fail rut depth. For example, based upon past experience some state highway agencies have established a rut depth of 5mm as the dividing point between rut-prone and rut resistant HMA mixtures. Hence, no HMA mixtures with an APA rut depth of 5 mm or greater would be constructed in the field. No attempts have been identified in the literature review to use the APA to predict how many 80-kN Equivalent Single Axle Loadings (ESALs) an HMA pavement can be loaded with until failure. The following topics were evaluated:

1. A methodology of converting APA rut depth and APA cycles to field rut depth and 80-kN ESALs.
2. The creation of a performance based specification based upon the methods presented in the report.
3. A preliminary Performance Based APA Specification for Michigan.

Converting the APA Test Performance to Field Performance

The wheel load in the APA is supposed to simulate the wheel loading on an in-service pavement while the rut created is supposed to be similar to the rut created by trafficking on in-service pavements.

To determine an APA rut depth that is equal to failure on an in-service pavement, a pavement failure rut depth must first be determined. Barksdale (1972) found that for pavements with a 2% crown (typical for the United States) rut depths of 0.5 in. (12.5 mm) are sufficiently deep to hold enough water to cause a car traveling 50 mph to hydroplane. The rut depth referred to by Barksdale is the total rut depth, not the downward rut depth. According to pavement rut depth taken from Westrack (FHWA, 1998) a 12.5 mm total rut depth is approximately equivalent to a downward rut depth of 10 mm. From APA data also taken from Westrack pavements it can be determined that a 10 mm downward rut depth on an in-service pavement correlates well with a 7 mm rut depth in the APA. Based upon these correlations, an APA failure rut depth of 7 mm will be used in establishing an empirical model.

The WesTrack experiment provided a unique opportunity to compare APA results with a full-size pavement testing facility where both the loading and temperature were known. APA test specimens were taken directly from the wheel paths of the test track before truck loading and were tested at 60 °C - nearly the same as the average high pavement temperature of 57.53 °C @ 12.7 mm depth (Williams and Prowell, 1999). The WesTrack pavement rut depths correlated very well with the APA test specimens taken from WesTrack.

Although the Westrack and APA test temperatures are nearly the same, the number of ESALs per APA cycle cannot be found simply by dividing 582,000 ESALs by 8000 cycles. This is because the trucks that loaded WesTrack traveled slower then ordinary trucks on highways and the wheel wander of the WesTrack trucks was tighter then ordinary truck traffic. Both truck speed and wheel wander have to be corrected as follows before the amount of rutting ESALs per APA cycles can be determined.

The Development of an Empirical Rut Prediction Model for Michigan

Since asphalt binder viscosity decreases with increasing temperature, HMA rutting occurs when pavement temperatures are above average, particularly in the summer months. More specifically, work done by Mahboub and Little (1988) stated the following assumptions could be made based on Texas HMA pavements:

- Permanent deformation occurs daily over the time interval from 7:30 a.m. to 5:30 p.m.,
- Permanent deformation occurs only in the period from April to October, inclusive, and
- Measurable permanent deformation does not occur at air temperatures below 50 °F (10 °C).

The Superpave 20-year design life includes all ESAL loadings during the entire 20-year design life. Based on the above assumptions, the number of ESALs in the 20-year design life needs to be adjusted to only the ESALs when rutting occurs, or “rutting ESALs”, if a PBS using the APA is to be developed. This process of making this conversion was developed and the steps are summarized below

1. Divide Michigan into six regions and retrieve weather data for each region.
2. Establish the length of the rutting season for each region.
3. Find the daily effective pavement temperature.
4. Find an average effective pavement temperature based on five years of weather data from each region.
5. Establish a rut factor during a Superpave 20-year design life.
6. Establish the amount of rutting during a Superpave 20-year design life.

A Preliminary PBS for Michigan

As stated in the previous section a PBS based upon APA data must include an APA rut depth failure criterion as well as the test length representing the HMA pavements design life, in terms of ESALs. Based upon these findings, Performance Based APA Specifications were created for all six Michigan regions. As mentioned, a PBS based on APA data must include both a test length (in terms of APA cycles) and a failure rut depth criterion. The rut depth criterion is summarized first, followed by the test length.

The failure criterion for an APA specimen was set at 7 mm based upon data gathered at WesTrack, but this criterion should be adjusted to consider APA testing variability. This rut criterion adjustment is based upon the following factors (Williams and Prowell, 1999):

- The level of confidence,
- The variance or standard deviation,
- The sample size and
- The specification limit.

A method established by Williams and Prowell (1999) to develop an APA pass/fail rut depth criteria taking the preceding factors into account. The rut depth criterion is set using the small-sample confidence for a one-tail test (Mendenhall and Sincich, 1989).

Asphalt Pavement Analyzer Results

Ten separate 9.5-mm nominal maximum aggregate wearing course mixtures were sampled during the 2000 construction season. The HMA was sampled from throughout the State of Michigan and included all four Superpave traffic levels (i.e. E1, E3, E10, and E30). These project’s mix designs were recreated in the laboratory and tested in the APA to determine the following:

- To determine the usefulness of the empirical model developed in the previous section,
- To determine the effect that changing asphalt content and air voids has on APA performance,
- Develop a regression model to predict APA rut depth, and
- Perhaps most importantly, the APA data presented will be correlated with future in-service pavement performance to assess the APA's usefulness in predicting the performance of Michigan HMA pavements.

Two types of APA data were analyzed. The first, the APA rut depth at 8000 cycles, is used industry wide as a indication of whether or not an HMA mixture will perform in the field. The second is the amount of APA cycles needed to achieve a rut depth of 7 mm. As shown previously, a 7 mm APA rut depth correlated with an in-service HMA rutting failure. The previous section also presents a method of converting APA cycles to 80-kN ESALs. Based on this, it is thought that the number of APA cycles needed to achieve a 7 mm rut depth can be converted to how many ESALs an in-service pavement could withstand before failure.

The results are of the above analysis were statistically analyzed. Specifically, the following was done:

1. The results were analyzed to determine whether or not changes in asphalt content and air voids result in statistically different APA rut depths at 8000 cycles and APA cycles until failure. Past experience has shown that changing asphalt content and air void content does change rutting performance of in-service pavements. Because of

this, it would be beneficial to know that the APA is sensitive to changes in these properties.

2. The average APA rut depths and standard deviations for each Superpave design level were analyzed. It is of interest to know if HMA mixtures designed at different Superpave levels perform differently in the APA.
3. Lastly, regression models were constructed to predict the APA rut depth using potentially ten different HMA material properties as predictor variables.

In this model, ten HMA properties were chosen as predictor variables to predict the dependant variable, which in this case is the APA rut depth at 8000 cycles. Two separate models were developed, one intended for use in research and one intended for practitioners using only HMA mixture properties typically found on a Job Mix Formula (JMF). A description of the ten HMA properties used as dependent variables as well as the properties effect on APA performance are as follows:

- Superpave Mixture Design Level: This property was included in the regression model as a classification variable. An increase in the Superpave mixture design level (i.e. from an E3 HMA mixture to an E10 mixture) would be expected to increase APA performance.
- Is the HMA mixture a coarse or a fine mixture (i.e. does the gradation curve pass above or below the Superpave Restricted Zone). This was a classification variable in the model; 0=Coarse, 1=Fine. Prior research has shown that coarse mixtures are more susceptible to changes in HMA mixture properties and thus can be more susceptible to rutting.

- Was the asphalt binder bumped?: In practice, asphalt binders are typically “bumped” a performance grade (PG) above the PG required for a project. For example, the M-43 HMA project in Lansing used a PG 70 binder when the climate required that only a PG 64 binder be used. Binder bumping is typically done on high stress pavements where the truck traffic is very high or traffic is moving slowly, such as at intersections. This practice is intended to increase the rut resistance of HMA pavements. The APA test settings presented in this report require that APA testing be done at the high temperature of the PG grade. The APA test temperature does not include bumps in the binder grade. For example, the Lansing M-43 project was tested in the APA at 64 °C, not 70 °C. The binder bump was included in the model as a classification variable; 0= binder was not bumped, 1=binder is bumped. A binder bump is expected to result in better APA performance.
- Fine aggregate angularity (FAA): FAA is a measure of the angularity of the aggregate passing the No. 8 sieve. The FAA used in this model was taken from the JMF of the project. The FAA is determined by AASHTO TP33. It should be noted that the FAA of the laboratory HMA mixtures may have varied slightly from the JMF FAA values. An increase in FAA is thought to increase APA performance.
- $G^*/\sin\delta$: $G^*/\sin\delta$, the complex modulus, is the asphalt binder property used in the Superpave Performance Grade Binder Specification to assess a binder’s susceptibility to rutting (SHRP,1997). G^*, the complex shear modulus, is a measure of the asphalt binder’s resistance to deformation while the phase angle, δ,

is a measure of the relative amounts of elastic and inelastic deformation. $G^*/\sin\delta$ was determined from rolling thin film oven (RTFO) aged binders sampled from each project. This was done in accordance with AASHTO TP5 at the Michigan Technological University asphalt binder lab. Higher values of $G^*/\sin\delta$ is thought to increase APA performance.

- Asphalt Film Thickness: This property is the measure of the thickness of the asphalt binder film surrounding the aggregate in an HMA mixture. The asphalt film thickness is dependant on the amount of asphalt content, the aggregate gradation, and the aggregate particle shape. The method of calculating the asphalt film thickness used in this study was developed by the National Stone Association Aggregate Handbook (1991). It assumes that all of the aggregate particles are round or cubical, thus it does not consider aggregate shape or texture in its estimation of asphalt film thickness. Based upon the literature, the relationship between asphalt film thickness and APA performance is unclear.
- Fines to Binder Ratio: The fines to binder ratio (F/B ratio) is simply the ratio of mass of the material passing the No. 200 sieve divided by the mass of total asphalt binder in an HMA mixture. The mass of the asphalt binder used in the ratio was the total asphalt mass, not just the effective asphalt mass. The F/B ratio was calculated based upon the laboratory HMA mixture. The F/B ratio in this study changed due to changes in the asphalt content, not by changing the amount of material passing the No. 200 sieve, which remained constant for each project. It is believed that fines can become embedded fully into the asphalt binder and act

as an asphalt binder extender. As more fines are included into the asphalt binder the asphalt becomes stiffer and will improve APA performance.

- Asphalt Content: The asphalt content was varied throughout the tests. An increase in binder content above the optimum asphalt content normally results in a loss of mixture stability and a decrease in APA performance.
- Air Voids: Air voids were calculated and varied throughout the tests. An increase in air voids leads to an increase in consolidation rutting as well as lower shear resistance and consequently decreases APA performance.
- Voids in the Mineral Aggregate: Voids in the Mineral Aggregate (VMA) is "the volume of interangular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume of the sample"(SHRP, 1996). Based upon the literature, the effect of VMA on APA performance is unclear.

In addition to describing the ten HMA properties included in the regression models, it is necessary to establish the range of each property

Four-Point Beam Fatigue Results

The four-point beam fatigue apparatus is a laboratory testing device used to evaluate the fatigue life of HMA. It estimates the number of loading cycles the pavement can endure before fracture occurs. The method used here can be seen in its entirety in AASHTO TP8, Standard Test Method for Determining the Fatigue Life of Compacted Hot Mix Asphalt Subjected to Flexural Bending. This test system is similar to a system developed by Deacon for his doctoral research in 1964. The beam fatigue apparatus subjects asphalt beams that are 50 mm high (+/-5 mm) x 63 mm wide (+/-5 mm) x 400 mm long to a constant rate of microstrain. The microstrain refers to the magnitude that the beam is deflecting (e.g. 600 microstrain means 0.0006 in/in or mm/mm). The load the sample is subject to decreases as the test progresses with time and increasing cycles. Both the stiffness and the number of cycles are recorded. The failure point is defined as when the beam reaches 50 % of its initial stiffness. A plot of log of cycles versus microstrain is then made. From this information the maximum flexural stress a pavement can endure is calculated. By knowing the relationship between the stresses or strains for a pavement and the repeated cycles to failure, the number of traffic loads to failure can be estimated. The test itself is typically used to compare different mixtures to give an indication of relative performance (Roberts et al. 1996).

The main purpose of the statistical analysis was to quantify the effects of certain variables and to model their effects on the fatigue life of hot mix asphalt. Each job listed in the experimental plan was tested in the four point beam fatigue apparatus. The results for each job were compiled and statistically analyzed using regression analysis. A regression equation allows for the relationship between a variable of interest (the dependent variable) to be explained as a function of other factors (the independent variables). A regression equation is commonly referred to as a prediction equation. Also, this type of analysis allows one to see what factors are statistically significant in explaining the relationship between the dependent and independent variables. The level of significance chosen for all regression modeling was 90%. This means that there is a 10% chance of making a Type I error. A Type I error occurs when the null hypothesis is rejected; when in fact it is true (McClave and Sincich, 2000). The dependent variable in the analysis was the natural log of the cycles to failure (Log Cycles). The independent variables were: microstrain (MS), air voids (AV), initial stiffness (IS), initial modulus (IM) and asphalt content (AC). These variables were the main effects in the model. Microstrain had no units, air voids was entered as a percentage (e.g., 4.00%), initial stiffness and modulus had the units of megapascals (MPa) and asphalt content was entered as a percentage by weight of the total mix (e.g., 5.0%). Nominal maximum aggregate size (NMAS 3, 4 and 5) and traffic level (E3, E10 and E30) were later added to the regression model as simple dummy variables.

Pay Factor Development for the Four-Point Beam Fatigue and Asphalt Pavement Analyzer

For the purposes of this research it was decided to base the rutting and fatigue cracking pay factors on parameters that can be readily measured either in the field or shortly thereafter in a lab. Two main parameters that can be directly related to rutting and fatigue life are the air void content, asphalt content, and voids in mineral aggreage. As mentioned previously, air voids cannot transfer a load. The relative compaction is a measure of the air void level in a pavement. The higher the target air voids level, the lower the fatigue life and rutting. The target air void level in the field is usually 7%, with a standard deviation of about 1.2% (Deacon et. al., 1997). As asphalt content is increased, the fatigue life is also increased but the rutting susceptibility increases.

The steps taken to develop pay factors for the State of Michigan with respect to fatigue cracking and rutting will be discussed in the report. Another statistical procedure called Monte Carlo simulation will be discussed. The Monte Carlo simulations were performed using a Microsoft Excel® based software package called @Risk© prepared by Palisade in February, 2001. The pay factors for MDOT were developed using many of the same techniques and equations that were presented by Deacon et al. in 1997. The work herein diverges from the aforementioned research in that aggregate size and traffic level design were taken into consideration when developing the proposed pay factors.

Since the air voids consistently yielded a broader range of pay factors, and hence, a more revealing prediction of the fatigue life (for the ranges of air voids experimented with in this research project), an owner/agency may consider using this as the more important factor for determining the pay factor. Another possibility would be to use a weighted average of the pay factors developed with respect to air voids and asphalt content. In terms of rutting, voids in mineral aggreage yielded a broader range of pay factors and hence some insight into the rutting susceptibility of that mixture.

It has been shown that the statistically based pay factors developed herein can be generated with target values for air voids, asphalt content, and voids in mineral aggregate. In addition, a certain level of reliability can be used to predict the pavement performance and to assign a pay factor accordingly. If an owner agency wants to be conservative with respect to air voids or asphalt content when awarding a bonus or a penalty, the 99.9% confidence band pay factor can be used. The confidence bands merely serve as a boundary for the pay factors as opposed to capping the pay factors at some percentage. This decision is always up to the owner/agency paying for the work to be built. As always, the owner has the right to stipulate that if the performance parameters are gravely out of specification, the HMA material can be removed and replaced.

It should be noted that the developed pay factors do not increase in numerically equal increments when deviating away from the target value. This is due to the way the change in present worth is calculated since it uses an exponential function to calculate the change in present worth. Therefore, the change resulted in a linear relationship when regressed against either asphalt content or air voids, but the incremental change away from the target value is not numerically equal.

The proposed pay factors developed herein have been developed with respect to fatigue cracking only. Separate pay factors for rutting need to be developed to account for an excessive use of asphalt content that would typically lead to a permanent deformation problem.

Summary

This project has successfully completed the three phases of the research project encompassing a literature review, development of APA specifications and its use to determine rut potential of MDOT mixes, development of experimental plans and field sampling of materials on projects throughout Michigan, and the use of the four point beam fatigue to determine the fatigue life of MDOT mixes.

PRODUCTS AND DELIVERABLES

The products and deliverables for the project are as follows:

- A series of Michigan maps detailing the high and low temperature contours corresponding to varying levels of reliability and as a function of pavement depth. The integration of these maps into selecting laboratory test temperatures are critical to the success in identifying appropriate testing specifications for laboratory equipment. The maps can be used to select the APA test temperature for permanent deformation.
- Evaluation of HMA sampling techniques on HMA volumetrics and asphalt binder. The research team determined the sampling method that is most consistent with the current Michigan DOT QC/QA specification is either truck or paver sampling since the measured volumetric properties and aggregate characteristics are close to each other and that a statistical analysis would indicate no statistical difference based on a 95% confidence limit.
- Volumetric determinations and analyses of mixes (bulk specific gravity, maximum theoretical specific gravity, binder content, air voids, voids in the mineral aggregate, voids filled with asphalt, etc.) sampled for this project.
- Asphalt binder characterization using Superpave binder tests (rotational viscosity, dynamic shear rheometer, bending beam rheometer, direct tension test, and aging of binders using a rolling thin film oven and pressure aging vessel) of tank binders and binders recovered from mixtures sampled from trucks and behind the paver.

- Performance testing of mixture specimens APA and FPBF. Development of a rut prediction model using an APA and a preliminary performance based specification using the APA. A fatigue life prediction model using the FPBF apparatus was also developed.

$$
\begin{aligned}
APA\,Rut\ Depth_{8000\,Cycles} = &-14.357 - 5.900\,\mathrm{E3} - 3.153\,\mathrm{E10} - 3.095\,\mathrm{E30} \\
&+ 2.598\,Grad + 1.591\,Bump + 0.760\,VMA \\
&+ 1.365\,AC
\end{aligned}
$$

where :

E3	= Is the HMA a Superpave E3 Mixture? Yes = 1, No = 0
E10	= Is the HMA a Superpave E10 Mixture? Yes = 1, No = 0
E30	= Is the HMA a Superpave E30 Mixture? Yes = 1, No = 0
Bump	= Was the Upper PG Bumped? 0=Yes, 1=No
Grad	= What Kind of Aggregate Gradation? 0 = Fine, 1 = Coarse
AC	= Asphalt Content (% of Mass of Mixture)
VMA	= Voids in the Mineral Aggregate (% by Mixture Volume)

Preliminary APA Machine Settings and Test Methods

Parameter	Specification
Test Temperature, (°C) [*1]	Upper Performance Grade of HMA Mixture Being Tested
Environmental Condition	Dry
Specimen Size, mm	Cylindrical Specimens with 150 mm diameter and 75 mm height
Load, N (lb)	445 (100)
Hose Pressure, kPa (psi)	689 (100)
Wheel Speed, m/sec	0.61
Number of Test Wheel Load Cycles	8000
Laboratory Compaction Device	Superpave Gyratory Compactor
Pretest Specimen Conditioning	4 hours @ Test Temperature
Number of Seating Cycles	50 Cycles

APA Failure Criterion Based on the Mean APA Rut Depth of 3 APA Specimens

Trafficking Level (ESALs)	**Allowable APA Rut Depth, mm**
E1	7.0
E3	5.0
E10	3.5
E30	3.0

$$
\begin{aligned}
&\text{Log Cycles} = 17.07054 - 0.00853MS - 0.19451AV \\
&-7.7309\times10^{-4} IS + 0.21630AC + 3.26\times10^{-6} MS^2 \\
&+9.407489\times10^{-8} IS^2 - 7.62028\times10^{-7} MSxIS \\
&+6.631\times10^{-5} AVxIS - 0.18715(Traffic = \text{E10 or E30}) \\
&-0.28706(\text{NMAS=3 or 4})
\end{aligned}
$$

Where:
Log Cycles is the natural log of the cycles to failure for fatigue life,
MS and MS^2 are the microstrain level and the microstrain level squared, respectively,
AV is the air voids content, expressed as a percentage,
IS and IS^2 are the initial stiffness and the initial stiffness squared, respectively, in MPa,
AC is the asphalt content, expressed as a percentage,
MSxIS is the interaction term for microstrain and initial stiffness,
AVxIS is the interaction term for air voids and initial stiffness,
Traffic = E10 or E30 is equal to 1 if the traffic level is E10 or E30 and is equal to 0 if the traffic level is E3, and
NMAS = 3 or 4 is equal to 1 if the NMAS is 3 or 4 and is equal to 0 if the NMAS is 5.

- Development of pay factor table using the APA and FPBF apparatus using MDOT parameters of asphalt content (AC), air voids (AV), and voids in mineral aggregate (VMA). Other mixture characteristics were statistically evaluated, however, only AC, AV, and VMA were found to be statistically significant.

CHAPTER 1 INTRODUCTION

1.1 Background

The implementation of Superpave mix design procedures in 1997 has continued throughout the United States, and has gained general acceptance by most State Departments of Transportation. The Michigan Department of Transportation (MDOT) has successfully implemented Superpave mix design procedures for all mainline paving, inline with the national trend. However there are some unresolved issues with the new mix design procedures, such as the identification and use of performance test criteria to accompany the volumetric mix design criteria. Other evolutionary issues with regard to specifications, such as quality control and quality assurance testing and performance-related specifications are also receiving acknowledgment as viable techniques to address dwindling experience in the industry. Coupling laboratory performance testing specifications and field specifications into one project allows for development of trial laboratory and field specifications and minimizes statistical bias of laboratory design criteria on field specifications.

1.2 Scope

Establishment of laboratory performance test specifications and field specifications would be a significant contribution to asphalt technology in Michigan. The continued decrease in expertise in the hot mix asphalt industry merits an examination of improved processes in laboratory and field specifications. Characterization of materials, performance testing of specimens, and statistical analyses will be used in developing specifications. These more advanced, science-based specifications should significantly improve the qualities of designed and constructed hot mix asphalt leading to improved service life for flexible pavements in Michigan.

1.3 Objective

Objectives of this study include the following: 1. field sampling of mixtures throughout Michigan, 2. characterizations of materials sampled, 3. development of laboratory performance test criteria to accompany existing Superpave mix design criteria, 4. development of field specifications for acceptance of hot mix asphalt. This study will involve both laboratory and field work. The following tests and evaluations are planned on the laboratory prepared and field specimens:

- i.) Gradation analyses.
- ii.) Volumetric determinations and analyses of mixes (bulk specific gravity, maximum theoretical specific gravity, binder content, air voids, voids in the mineral aggregate, voids filled with asphalt, etc.).
- iii.) Asphalt binder characterization using Superpave binder tests (rotational viscosity, dynamic shear rheometer, bending beam rheometer, direct tension test, and aging of binders using a rolling thin film oven and pressure aging vessel).
- vi.) Performance testing of mixture specimens (Asphalt Pavement Analyzer, beam fatigue, indirect tensile test, etc.).
- vii.) Development of a pay factor system based on the APA and beam fatigue apparatus.

Statistically sufficient testing will be done in support of analyses of test materials.

1.4 Report Organization

Chapter two discusses the literature review on performance tests and their use in evaluating hot mix asphalt (HMA). The literature review also examines different types of specifications' methods such as quality control/quality assurance, performance-related specifications, and performance-based specifications. Chapter three outlines how the asphalt mixes were verified. Chapter four characterizes the asphalt binder. Chapter five outlines the experimental plan and how the statistical analyses will be conducted. Chapter six outlines the HMA specimen preparation in the Asphalt Pavement Analyzer and the Four Point Beam Fatigue Apparatus. Chapter seven gives the results and analysis of the three phases of this study: 1) field sampling that was performed for the project; 2) Asphalt Pavement Analyzer (APA) specifications and 3) Four-Point Beam Fatigue apparatus. Chapter eight summarizes this report.

1.5 Implementation

Implementation of the results and recommendations in this study is expected to assist the Michigan Department of Transportation in advancing specifications in several areas. These areas are as follows: sampling for quality assurance testing, identification of performance test criteria to accompany Superpave mix design, and provide a framework for furthering specifications using performance-related specifications and mechanistic pavement design. Overall, the research project's objective is to provide MDOT with implementation tools to optimize HMA performance and costs.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

Sampling of bituminous hot-mix asphalt and subsequent testing is the only method to evaluate the quality of a production process. Sampling is equally as important as the testing. It must be done in an unbiased manner to represent an acceptable estimate of the nature and conditions of the material. The increasing tonnage of HMA and corresponding reduction in agency personnel has led to more innovative processes being implemented in the HMA industry. The traditional approach of certifying the quality of the HMA constituents and HMA at steps throughout the production process by agency personnel is called method specifications. Although the method specification approach has generally been effective, it has also been reported that it does not promote innovation (Anderson and Russell). This combination of events and circumstances has led to the implementation of quality control/quality assurance (QC/QA) specifications. The QC/QA approach is a statistical approach to ensure the end product is of acceptable quality, but is heavily dependent upon sampling and corresponding test results. The continued research thrust in the areas of performance-related and performance-based specifications has demonstrated the dependence of the process on representative samples, e.g. samples that are representative of the material in-place. Performance warranties, contract processes in place in the European HMA industry, will not be as important to agencies, but will be for the contracting entity to ensure long-lasting quality. Figure 2.1 demonstrates the evolution of specifications from method specifications to performance warranties.

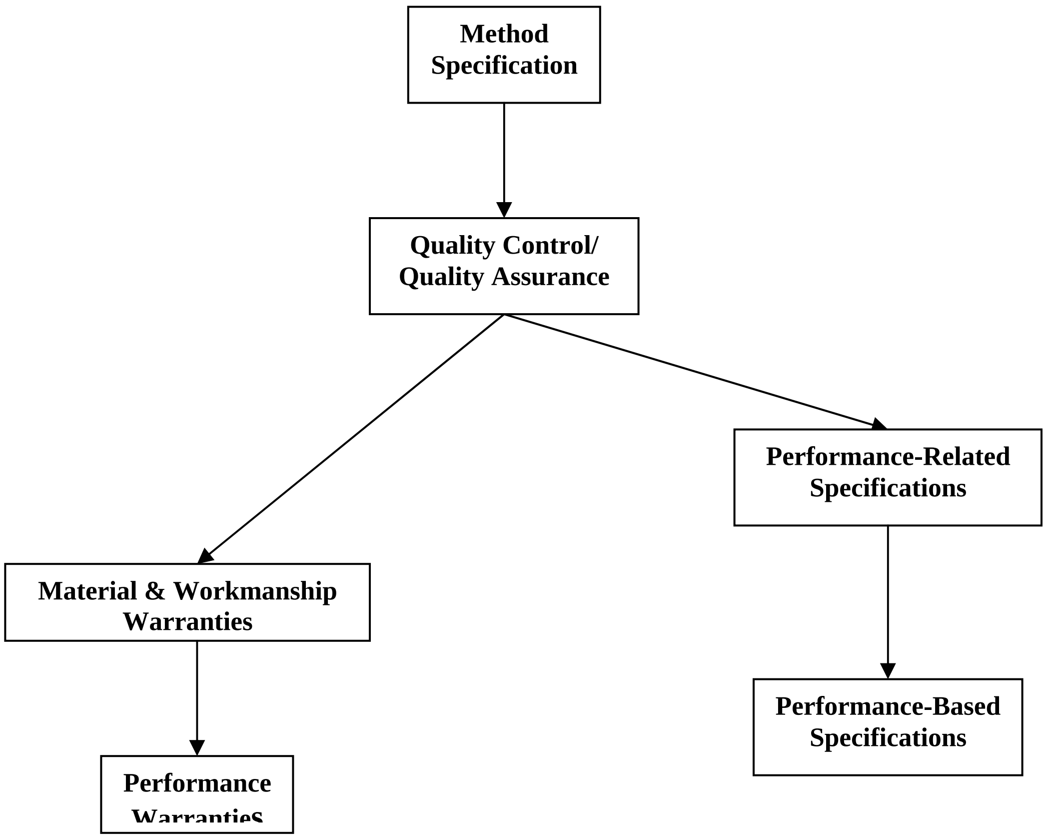


Figure 2.1 Evolution of Specification

Most State Highway Agencies (SHA) currently employ method specifications or quality control/quality assurance, some have implemented performance-related specifications (PRS) and material and workmanship warranties. California has developed a PRS specification which payment is dependent upon HMA lift thickness, air voids and binder content as compared to the structural and mixtures designs and is in the early stages of implementation (Deacon, Monismith, and Harvey). Michigan has employed a material and workmanship warranty process where the responsible contractor will provide a three to five year warranty for the materials and workmanship. However, the contractor in Michigan is not responsible for elements related to the structural design nor materials below the placed HMA for the specific pavement. It is anticipated that the next type of specifications to be implemented will be performance-based specifications and/or performance warranties.

2.2 Significance and Use

There are several motives for sampling. The three primary reasons are: 1) Samples are taken for the development of preliminary data by parties responsible for the development of the specification data, 2) The manufacturer, contractor, or other parties responsible for accomplishing the work, take them for control of the product at the source of manufacture or storage, or at the site of use, and 3) Samples are also taken for use in an acceptance or rejection decision-making process by the purchaser or an authorized representative via testing. An example of the first reason would be for field verification of a plant-produced mix at the beginning of a project to ensure it is within Superpave Laboratory Trial Mix Formula (LTMF) Tolerances. The second reason demonstrates the Quality Control Plan where the contractor is responsible for the development and formulation of the Superpave mix design and the State Highway Agency (SHA) verifies the mix. The Quality Acceptance Plan is an example of the third reason where the Contractor tests more frequently on a series of lots and the SHA decides whether to accept of reject the mix if it is within specification limits. The purpose of inspection and testing is to provide reasonable assurance to the purchaser that the quality of component materials comply with the standards specified, and that the manufactured asphalt is in accordance with the designated job mix design (National Asphalt Specification). A more recent survey of SHA found a number of different properties that are being used to measure/ensure HMA quality (Burati and Hughes). The results of this survey are summarized in Table 2.1.

ASTM D 979-96 gives detailed procedures for sampling from a conveyor belt, truck transports, roadway prior to compaction, skip conveyor delivering mixture to bin storage, and the roadway after compaction. For the purpose of this report two common methods, truck sampling and sampling from behind the paver are explained in more detail.

2.3 Sampling

Sampling of produced HMA can occur at any one of four locations: truck, paver, behind the paver prior to compaction, and core sampling of the roadway (after compaction). The Federal Highway Administration published a regulation on quality control/quality assurance testing of highway materials in 1995 (Federal Highway Administration). The ensuing implementation of QC/QA on a broad basis has led to testing differences between SHA and contractors in some instances. Resolution of these differences has led to split sampling processes where a sample is being split into three approximately equal amounts: one for the contractor, one for the SHA and one for a third party in case of a dispute by either the contractor or SHA.

Table 2.1 Statistical Quality Measures Used by States (Burati and Hughes)

State	Gradation	% AC	Air Voids	VMA	Density	Thickness	Bulk Spec. Gravity	Max. Spec. Gravity
AK	X	X			X			
AR		X	X	X	X			
CO	X	X			X			
CT	X	X			X			
IA					X	X		
ID	X	X			X			
IL						X		
LA	X	X	X	X	X			
MD					X			
ME	X	X	X	X	X			
MI		X	X	X	X			
MN	X	X	X	X				
MS	X	X	I	X	X			
MT	X				X			
NC	X	X	X	X	X			
ND	X	X			X			
NE					X	X		
NJ			X		I	X		
NM	X	X	X	I	X			
NV	X	X	X	I	I			
NY	X	X	X	I	I		X	X
OH		X			X			
Ontario	X	X	X	I	X			
OR	X	X			X			
PA	X	X			X			
SC		X	X	X	X			
TX					X			
VA	X	X						
WA	X	X			X			
WI		X	X	X	I			X
WY	X	X			X			
31	20	24	13 (14)[1]	9 (13)	24 (28)	4	1	2

X: Property is measured/calculated.
I: Property can be calculated from other property measurements.
1: Sum of X (X&I).

2.3.1 Truck Sampling

The Michigan Department of Transportation explains the following procedures for truck sampling. The procedures require two people, an inspector and an observer. Whenever sampling, always make sure the conditions are safe. Wear appropriate safety equipment including a hardhat, safety glasses, long sleeves, pants, and thick-soled insulated shoes.

2.3.1.1 Procedure 1: Sampling in a Haul Truck

The following equipment for sampling in a haul is needed: shovel/scoop, thermometer, and appropriate container to hold the sample. In this procedure, the inspector will take samples while in the haul truck. While both the inspector and observer are on the ground, the truck pulls up next to the station. The driver will then exit the truck. The inspector and observer after making sure the platform is safe will climb up onto the platform and undo the safety chain connected on the platform. The inspector will proceed to climb into the back of the truck. The inspector will then insert a thermometer into mixture leaving it until later. The surface material is cleared and with the shovel or scoop the inspector obtains a HMA material below the surface material and places the material into appropriate containers. Samples should be taken in at least 6 different locations throughout the truck. Each sample should provide more than enough for testing in case there is an error. After all samples are obtained, retrieve the thermometer, check the temperature for compliance, and record on the sample sheet. Hand over equipment and samples so that both hands are free when exiting the truck. Replace the safety chain and carefully exit the platform.

2.3.1.2 Procedure 2: Sampling a Haul Truck from a Platform

The alternative way to obtain truck samples is by not climbing into the truck. This procedure will require the same equipment as outlined above. Climb up the platform as in the previously described procedure. This time, the samples will be taken from the platform by reaching in with a scoop. Take at least three samples from the side of the truck. It may be necessary for the truck to move up. Whenever the truck needs to be moved, exit the platform first. The truck will need to turn around in order to obtain at least three more samples from the opposite side of the just sampled truck (Michigan Department of Transportation).

2.3.2 Roadway Sampling

According to Sampling Bituminous Paving Mixtures AASHTO T 168 (Federal Highway Administration Multi-Regional Asphalt Training & Certification Group) the procedure for roadway sampling is as stated. "Sample the uncompacted mat by placing a template through the entire lift of HMA (hot mix asphalt), or using a square pointed shovel to create a sample area with vertical faces. Remove all material from within the template or between the vertical faces and place in a clean sample container. Avoid contaminating the sample with any underlying material. At least three increments should be obtained for each sample."

2.4 Sampling Behind the Paver

The Ontario Ministry of Transportation and Communications in Ontario, Canada developed a technique to obtain samples from behind a paver prior to compaction. The process involves using a metal plate and a locating wire attached to the plate. A metal plate is placed in front of a laydown machine with the locating wire placed to beyond the paving lane. The laydown machine paves over the plate and the portion of locating wire in the paving lane. The locating wire is pulled up through the uncompacted mix to locate the plate. The plate with the HMA is lifted out of place and represented as a sample. The mix was left undisturbed on the plate and the plate was sent to the laboratory for evaluation.

2.5 Visual Inspection

Each loaded truck or paving mat can be visually inspected for segregation, uncoated particles, excess bitumen or overheating, before dispatch from the plant. Warning signs of insufficient hot mix include the following. If blue smoke is seen escaping the hot mix, it is most likely too hot. If the mix is lumpy it is a sign of the mix not being hot enough. Mixes that are soupy, stiff or dull may be indicators of inappropriate asphalt content. Finally, pockets of coarse or fine aggregate indicate segregation (Michigan Department of Transportation).

2.6 Comparison of Sampling Techniques

The Ministry of Transportation and Communications conducted a statistical analysis of three different sampling techniques. The first two methods involved truck sampling. The first method the researchers termed the “scoop method”.

The scoop method requires a trench be scraped in the mix from the truck side to the top of the pile using a garden trowel. From this trench one scoop is taken from the top, center and bottom sections and combined to form one sample. This is done twice per truck.

The second method, termed the shovel method, involved plunging a shovel into the pile randomly. Again two samples were taken from each truck and appropriately packed for later testing. The third method developed and evaluated by the Ontario Ministry of Transportation and Communications was for sampling behind the paver. The sampling behind the paver procedure, detailed in the section 2.4, was the third sampling technique evaluated.

The results of the tests showed that the effect of sampling methods on the overall average value of the property being tested was not significant. However, there was considerable difference noticed in the individual interactions such as in samples compared from day to day or batch to batch. It also showed that truck sampling methods are more operator-sensitive than the developed plate method. The operator may not dig as deep a trough or separate his scoop holes properly in the scoop method. In the case of the shovel method, he may take a shovel of mix very shallow and close to the truck sides where the coarse mix is segregated rather than digging deeply into the center of the mix pile. In the plate method, the operator has little control on the sample taken. A significant conclusion taken from the Ontario study is that the shovel method showed about twice the variation then both the plate and scoop methods.

This study does acknowledge the effects of segregation. The plate sampling method gave the most consistent results because it is less subject to segregation. The material undergoes extra mixing when it passes through the paver. A steel belt draws it from the bottom of the pile in the paver hopper. The material on the sides of the paver hopper slowly flows down, becoming mixed in the process, and is picked up by the belt. The material at the end of the belt is then picked up by an auger blade and distributed across the width of the road being paved and immediately held in place and compacted by the paver screed. The auger blade also serves to further mix the material. The sample taken, then, is the full depth of the material laid on the road. A major disadvantage of sampling behind the paver is that it is more time consuming. The scoop method better estimates the plate method than the shovel method because it provides additional mixing by taking mix from the bottom, middle and top (Fromm).

2.7 Sampling Errors

Every effort should be made to avoid sampling errors as this could effect test results and subsequently effect payment to a contractor. Although there are nearly an unlimited number of sampling errors that can occur, the following a few common ones:

- Obtaining the entire sample from a single location.
- Not removing all the material from within a template.
- Contaminating the sample with underlying material.
- Segregating the material while sampling. (Federal Highway Administration Multi-Regional Asphalt Training & Certification Group)

Equally important to avoiding sampling errors are errors in handling the material prior to testing such as storage or maintaining uniformity when splitting sampling if necessary.

2.8 Segregation

One of the largest problems in sampling HMA is segregation or maintaining HMA uniformity during the production, construction, and testing process. Segregation is the separation of the coarse and fine aggregate particles in an asphalt mix. Segregation can occur at several locations prior and during the mix production, hauling, and placing operation. Segregation of aggregate stockpiles or bridging effects in coldfeed bins can result in segregated HMA being placed. Segregation can also occur as the mix is delivered from the asphalt plant to a surge silo or loaded into the haul truck from the silo. It can also occur when the mix is discharged from the truck to the paver hopper as well as during inconsistent paver operations (US Army Corps of Engineers). Well-maintained equipment and checks of equipment operations can usually minimize HMA segregation. If segregation is encountered with a particular mixture, one option is to switch mix designs to a different gradation.

2.9 Shipping Samples

Samples should be packaged in containers that will avoid contamination or damage to the sample. Adequate identification should be attached to the sample to provide clear information about the status of the sample. Possible useful information to be recorded include the following:

- Job project number
- Highway route number
- Owner or operator of plant
- Location of plan
- Quantity represented
- Identification of bitumen and mineral aggregates used
- Point at which sampled, both by station number and location transversely in pavement
- By whom sampled and title
- Date of most recent mixing
- Date sampled
- By whom submitted and address
- Purpose for which sample was taken
- To whom report is to be made

Other useful information such as sampling conditions could also be recorded as well. There is generally a section for comments to recorded information that is not typical about the sampling circumstances.

2.10 Selecting Sampling Sites in Accordance with Random Sampling Techniques

It is essential that the sample location be chosen in an unbiased manner. In order to obtain sampling locations, a common set of definitions for lot and sublot must be used. The Transportation Research Board has developed the following definitions for lot and sublot as follows:

- *Lot:* An isolated quantity of a specified material from a single source or a measured amount of specified construction assumed to be produced by the same process.
- *Sublot:* A portion of a lot, the actual location from which a sample is taken. The size of the sublot and the number of sublots per lot for acceptance purposes are specified in the specifications.

Lot size, sample size, and the testing frequency for the control and acceptance of HMA are highly variable from agency to agency. Some states use an area or length basis as a unit for determining lot size, whereas others use a day's production basis or a tonnage basis. Typically, lot sizes defined by some SHA's range from 500 to 4,000 tons (Transportation Research Board).

There are different approaches used in order to determine the lot and sublot size. For example there is sampling by time sequence, material tonnage or by area. All of these examples involve using a table of random numbers. This table can be used when single dimensions (e.g., time, tonnage, and units) or two dimensions are required (e.g. left or right edges of the pavement).

2.11 Quality Acceptance Sampling

Quality acceptance sampling and testing of HMA is a prescribed procedure, usually involving stratified sampling and is applied to a series of lots. The acceptance sampling and testing enable the SHA to decide on the basis of a limited number of tests whether to accept a given lot of plant mix or construction from the Contractor. A stratified random sampling plan shall be followed to obtain a minimum of five samples per lot. The lot shall be at least 1,000 tons or one day's production of HMA. It emphasized that the objective of acceptance sampling and testing is to determine a course of action (accept or reject). It is not an attempt to "control" quality. Briefly, in terms of acceptance sampling, the Acceptance Plan for HMA defines the following:

- Lot size,
- Number of samples or measurements,
- Sampling or measuring procedure,
- Point(s) of sampling or measurement,
- Method of acceptance, and
- Numerical value of specification limits.

Acceptance can be dependent upon sliding scale of a few different specification limits. One example may be the binder content must be within 0.5% of the target if the other quality parameters are met, e.g. the percent of the maximum specific gravity of the compacted mix is 96%. However, a sliding scale could be that the binder content must be within 0.3% if the percent of the maximum specific gravity of the compacted mix is with 92%.

2.12 Quality Assurance Plan Approach for HMA

Establishment of a quality assurance plan is done prior to any material being produced. The acceptance plan involves evaluating the percentage of material within the HMA specification limits. A table for the quality index values for estimating percent within limits is set up. N sampling positions on a lot are located using a table of random numbers. Measurements are taken on the test portions with averages and standard deviations of the quality parameters being calculated. From here the two quality indexes, average and standard deviation, are used to calculate the upper and lower specification limits. Using the table for the quality index values, the upper and lower tolerance limits are obtained and the percent within limits is calculated.

The current quality assurance practice is to take five cores per lot for each of the following measures: asphalt content, percent compaction (percent of the maximum specific gravity of the compacted mix) and aggregate gradation. However, studies have shown that the sample size may be reduced to four samples per lot (McCabe, AbouRizk, and Gavin).

2.13 Quality Control Sampling

For quality control (QC) purposes, the testing frequency used by the contractor is more frequent to ascertain that the process variation is within specification tolerances. Since the contractor tests more frequently to ascertain that the process variation is within specification tolerances, the SHA needs only to carry out additional work in accordance with the specification acceptance plan to ensure the degree of the HMA with the mix design specification. If differences between the SHA and contractor test results are encountered, a third party may be asked to test additional samples in order to resolve differences in test results or act as a referee.

2.14 Quality Control Plan Approach for HMA

The quality control (QC) plan is based on a concept of continuous sampling of HMA at the plant. Lots and sublots are considered in the QC plan only for in-place compaction. The QC sampling will progress continuously as long as the target values are within the LTMF tolerances and do not change substantially as monitored by the control chart values. The objective of sampling and testing associated with a QC plan is to ensure conformance of the mean properties of the "plant-produced" mix with the "target" mix and to minimize variability in the HMA.

The contractor's QC plan shall be based on random sampling and testing of the HMA at its point of production to determine compliance with the LTMF tolerances. The contractor shall measure by means approved by the SHA and record a daily summary including the following:

- Quantities of asphalt binder, aggregate, mineral filler, and (if required) fibers used;
- Quantities of HMA produced; and
- HMA production and compaction temperatures.

The QC plan shall include a statistically sound, randomized sampling plan to provide samples representative of the entire HMA production and to ensure that all sampling is conducted under controlled conditions (Transportation Research Board).

2.15 Introduction to Performance Testing of Sampled HMA

The need for the implementation of a laboratory performance test specification and field specification for Hot Mix Asphalt (HMA) production in the state of Michigan has been established. This literature review is being conducted so that any recent and significant developments in this research area can be utilized in this project. In particular, current practices and new developments in mix design, quality management, and performance tests and their use in evaluating HMA. The performance tests included in this literature review are the Asphalt

Pavement Analyzer, the Superpave Shear Tester, the Flexural Bending Beam Test, and the Indirect Tensile Test.

This chapter explains and summarizes literature that is relevant to the issues rutting, fatigue cracking, and the development of a pay factor system in HMA pavements. Also, this chapter will provide information on types of asphalt mix design methods and specifications that are currently used in the HMA industry. This chapter defines and addresses some of the past and current problems with rutting and fatigue cracking and what research has been done concerning this problem.

2.16 Mix Design

An asphalt concrete is a “high-quality, carefully controlled hot mixture of asphalt cement and well-graded, high-quality aggregate thoroughly compacted into a uniform dense mass” (The Asphalt Institute, 1989). The first step in obtaining this “high-quality mixture” is known as the mix-design. The mix design is the process of choosing adequate materials for the HMA being designed and the proportioning of these materials. The materials in HMA include the mineral aggregate, asphalt binder, and any modifiers or additives that may be used to enhance the pavement’s performance. In addition to choosing an adequate aggregate source (i.e., an aggregate consisting of angular particles), the aggregate gradation must also be considered in the mix design.

2.16.1 Mix Design Factors

It is important that the environmental conditions that the pavement will be subjected to in its service life are taken into account while designing an HMA. Some of the factors that must be considered are the temperatures the pavement will encounter as well as the amount of traffic and load level (i.e. passenger cars vs. trucks). In addition to environmental considerations a number of objectives should be kept in mind. These objectives are (Roberts, et al, 1996 and The Asphalt Institute, 1989):

- Resistance to Permanent Deformation,
- Fatigue Resistance,
- Resistance to Low Temperature Cracking,
- Durability and Resistance to Moisture Induced Damage,
- Permeability,
- Skid Resistance,
- Flexibility, and
- Workability.

2.16.1.1 Resistance to Permanent Deformation

Permanent deformation (or rutting) is the appearance of longitudinal depressions in the wheel paths of a roadway. Permanent deformation is the result of small deformations within the pavement that accumulate with each passing truck (The Asphalt Institute, 1996). These deformations are formed when a pavement does not have adequate shear strength (The Asphalt Institute, 1989). Resistance to permanent deformation is controlled in the mix design by selecting quality aggregates with proper gradation and the proper asphalt content (The Asphalt Institute, 1996). The proper amount of asphalt ensures that an adequate amount of air voids will be present within the HMA. This will ensure that an excessive amount of asphalt is not being used. Excessive asphalt tends to act as a lubricant and may result in rutting (The Asphalt Institute, 1989). An effective mix design provides enough internal friction to resist permanent deformation.

2.16.1.2 Fatigue Resistance

Fatigue resistance is the ability of asphalt pavement to withstand repeated flexing caused by the passage of wheel loads (The Asphalt Institute, 1989). When the internal tensile stresses at the bottom of the surface layer become greater than then the pavement's tensile strength, a tensile crack results (The Asphalt Institute, 1996). As a rule, the higher the asphalt content, the greater the fatigue resistance of a pavement (The Asphalt Institute, 1989). There is a running debate on whether cracks are initiated at the top or the bottom of a pavement layer. Over time these strains cause cracks to develop and propagate up to the surface of the pavement, which results in damage throughout the entire asphalt layer. Figure 2.2 below shows where fatigue cracking typically originates.

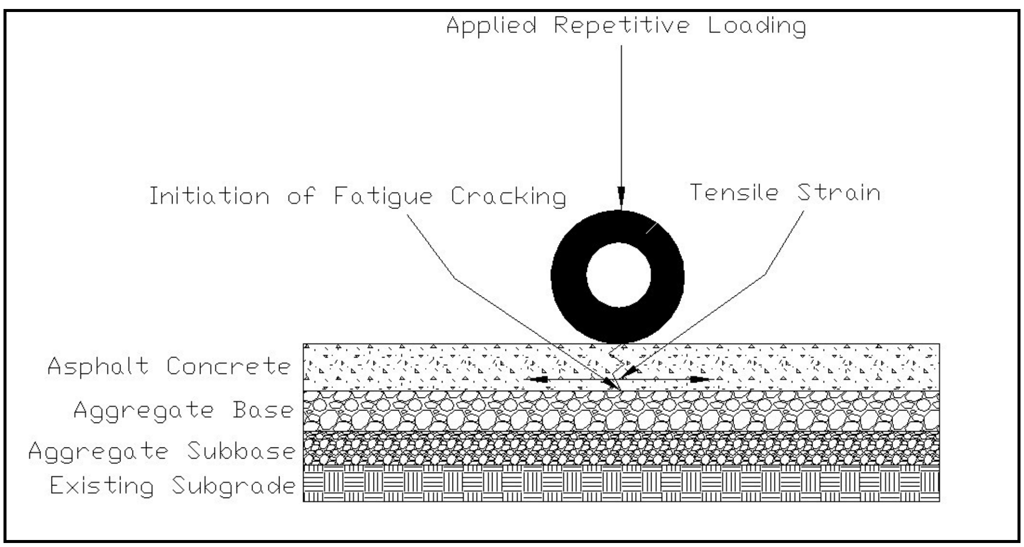


Figure 2.2: Fatigue Cracking in HMA Pavements

Fatigue cracks can have just one fissure longitudinally with traffic, or they can spread about in a shape that resembles the pattern on an alligator's back, hence fatigue cracking is also referred to as "alligator" cracking.

It is believed that pavements can become prone to fatigue cracking by not having the correct combination of the asphalt content and air void content in the mix. If the correct asphalt content is not used in a mix, the pavement being placed can be susceptible to fatigue cracking. Usually, it is when too little asphalt cement (AC) is used that these cracks can develop. Pavements with not enough AC in them can be stiffer or more brittle than intended and not able to flex with the loading of traffic. This can cause cracks to develop. Pavements containing too high of an air void content are also prone to fatigue cracking. This is because an air void cannot transfer a load. It is thought that as the thickness of the binder increases between the aggregates, the stresses in the mix dissipate. Decreasing the air voids will generally make a more uniform mixture, which allows for stress to be dispersed more effectively. This can be done while not increasing the binder content so as to have a pavement that is prone to permanent deformation.

2.16.1.3 Resistance to Low Temperature Cracking

Low temperature cracking occurs when an HMA contracts as it is exposed to cold weather. This causes tensile stresses to develop. If these tensile stresses exceed the tensile strength, a low temperature crack develops. Since the low temperature performance of an HMA is primarily governed by the low temperature properties of the asphalt binder, proper selection of asphalt binder during the mix design process is critical (Roberts, et al., 1996).

2.16.1.4 Durability and Resistance to Moisture Induced Damage

Durability is the property of an asphalt paving mixture that describes its ability to resist the detrimental effects of air, water, temperature, and traffic. Durability is generally enhanced by high asphalt contents, dense aggregate gradations, and well-compacted, impervious mixtures. High asphalt content results in thicker films of asphalt around the individual asphalt particles.

This increased film thickness makes the asphalt concrete more resistant to age hardening. Hydrophobic ("water-hating") aggregate should be used because water can weaken the adhesion of the aggregate to the asphalt binder and result in raveling. All of these factors, and also the consideration of an anti-stripping additive, should be taken into account in the mix design (Roberts, et al., 1996, The Asphalt Institute, 1989).

2.16.1.5 Permeability

Permeability is the resistance that an asphalt pavement has to the passage of air and water into or through a pavement. The permeability of an asphalt pavement is related to the durability in that an impervious surface helps safeguard against age hardening of HMA deeper in the pavement layers and stripping. The amount of air voids in an asphalt pavement is an indication of the permeability of the pavement. Due to this fact the initial in-place (immediately after construction) air voids should never be over 8% of the total pavement volume (Roberts, et al., 1996). In the mix design, an adequate amount of asphalt binder and a dense gradation of aggregate will lower the amount of air voids.

2.16.1.6 Skid Resistance

If the asphalt pavement is to be a surface mix, the mix should provide sufficient resistance to skidding to permit normal turning and braking movement. Aggregate that is resistant to polishing should be used to ensure an adequate amount of friction between automobile tires and the road surface. Also, if too much asphalt binder is present flushing may occur and be the cause of a lower friction pavement (Roberts, et al., 1996). These factors need to be taken into account in the selection of aggregate prior to the mix design.

2.16.1.7 Flexibility

Since a pavement may settle differentially (i.e. some portions of the pavement settle more than the others do), an amount of flexibility should be provided for in the pavement design to ensure against cracking. A high asphalt content and an open-graded aggregate (i.e. an aggregate with less fine aggregate) should be used (The Asphalt Institute, 1989). The open-graded aggregate requirement is in contradiction to the dense-graded aggregate needed for mixture durability and a balance should be provided. This calls for engineering judgment.

2.16.1.8 Workability

Workability is the ease in which an HMA can be placed and compacted. The angular aggregate needed for mix stability sometimes render the asphalt mix unworkable and adjustments to the design mix may need to be made during paving operations (Roberts, et al., 1996).

2.16.2 Mix Design Methods

The objective of HMA mix design is to determine the combination of asphalt cement and aggregate that will provide long lasting pavement performance. A mix design method is a series of laboratory tests used to choose a mixture that will perform satisfactorily in expected service conditions (The Asphalt Institute, 1996). The results of these laboratory tests have been correlated with actual performance of asphalt pavements in the field. The agency or the authority responsible for the paving construction (i.e. a state Department of Transportation) usually will establish which mix design method is to be used and the design requirements (or specifications) (The Asphalt Institute, 1989). These design requirements are based on the results of laboratory tests and are used to characterize HMA. For example, one design requirement an agency may specify is a minimum theoretical density (i.e. the density of an HMA compressed to the point that there are no air voids). A mix design method will have a test established to measure the theoretical density of a trial mix to compare with an agency's specification. There are three mix design methods that are generally used in the United States today: The Hveem method of mix design, the Marshall method of mix design, and the Superpave mix design method. Both the Hveem and Marshall mix design methods are empirical methods of mix design. This means that specifications written for these methods are based solely upon experience (The Asphalt Institute, 1996). For example, A Hveem Stabilometer specification of 37 has no theoretical significance. It has just been observed previously that HMA mixes with this Stabilometer value has performed well in the past. Even with proper adherence to an empirical mix design method, good performance of an HMA cannot be assured (The Asphalt Institute, 1996). For this reason the Superpave mix design method has been developed. The Superpave system is a performance-based specification system. The tests and analyses have direct relationships to field performance (The Asphalt Institute, 1996). The Superpave mix design method is quickly becoming the method of choice and the other methods have been used less frequently. The Superpave mix design is currently being implemented in Michigan. For these reasons this literature review will touch lightly on the empirical methods of mix design. The Superpave mix design method will be discussed more thoroughly, as well as the changes that have been made to this mix design method since it first was implemented.

2.16.2.1 Hveem Mix Design Method

Francis Hveem of the California Department of Transportation developed the Hveem mix design method. The method has been refined since its initial development by both Hveem and others and has been standardized by the American Society for Testing and Materials (ASTM) and published as ASTM D 1560 and ASTM D 1561. It has not been used extensively outside the western United States (The Asphalt Institute, 1996), with Maine being the exception. The Hveem mix design method does not specify any tests for aggregate. It simply states that the aggregate source and gradation should meet project specifications (The Asphalt Institute, 1989). The principal features of the Hveem method of mix design are the surface capacity and Centrifuge Kerosene Equivalent (CKE) test on the aggregates to estimate the asphalt requirements of the mix, followed by a stabilometer test, a cohesiometer test, a swell test, and a density/voids analysis (The Asphalt Institute, 1996). Hveem noticed that the asphalt film thickness decreased as the aggregate particle diameter decreased with a constant mass of asphalt. This discovery lead to the development of the CKE. This test, which estimates appropriate asphalt content, takes into account aggregate surface texture and absorption of binder into the aggregate (The Asphalt Institute, 1996). Hveem realized that the asphalt content found using the CKE did not guarantee that an HMA would be stable (rut resistance). This lead to the development of the Hveem stabilometer (Figure 2.3), a triaxial testing device in which vertical loads are applied and resulting lateral pressures are measured (The Asphalt Institute, 1989). The Hveem Stabilometer value is dependent on the ratio of vertical and horizontal stresses when the vertical compressive stress is 400 psi (Hughes, et al., 1997).

Figure 2.3 Hveem Stabilometer

Hveem also developed a device called a cohesiometer that measured a tensile property of oil mixes that could be used to combat raveling of aggregate under traffic. It proved to be of little use and fell out of favor (Romero, et al., 1997). Hveem developed a test to gauge a trial mix's permeability. This test is known as the swell test and consists simply of leaving a compacted trial mix briquette in contact with 500 ml of water. The amount that the briquette "swells" vertically is representative of the trial mix's permeability (The Asphalt Institute, 1989). The Hveem mix method calls for the determination of the bulk specific gravity via ASTM D 1188 and ASTM D 2726 (4). The Asphalt Institute has presented some advantages and disadvantages of the Hveem mix method (The Asphalt Institute, 1996).

Advantages:

- The kneading method of laboratory compaction is thought to better simulate the densification characteristics of a compacted HMA.

- The Hveem stability is a direct measurement of the internal friction component of shear strength.

Disadvantages:

- The test equipment is expensive and not very portable.
- Some important mixture volumetric properties related to durability are not routinely determined.

As mentioned, the Hveem mix design method does not guarantee good performance. For this reason, agencies have designated performance tests in the past to supplement the Hveem mix design method (The Asphalt Institute, 1996). These performance tests can be used to predict how well a pavement will perform in service. An example of a supplemental test is the Asphalt Pavement Analyzer, which will be examined later in this literature review.

2.16.2.2 Marshall method of mix design

In 1939, Bruce Marshall of the Mississippi Highway Department developed the earliest version of the Marshall mix design method. Motivated by the increase of aircraft wheel loads during World War II, the Corps of Engineers Waterways Experiment Station (WES) began experimenting with Bruce Marshall's method. Experiments were done to develop a method of compaction that would match the densities of the asphalt pavements subject to aircraft loadings. It was desirable to adopt a procedure to select an optimum asphalt content that was portable and fast. Based on the results of their experiment a mechanical hammer was developed (Figure 2.4) to compact specimens and the 50 Blow Marshall Criteria for Surface Mixes was developed. As aircraft size increased the 75 Blow Marshall Criteria for Surface Mixes was developed (Roberts, et al., 1996).

Figure 2.4 Marshall Mechanical Hammer and Pedestal

In addition to these developments, in the 1950’s WES noted rutting problems in a number of Marshall projects and raised the Marshall Stability to 1800 lb. and also lowered the amount of natural sands that could be present in an HMA. The Marshall method of compaction has been standardized and published as ASTM D 1559.

The aggregate requirements of the Marshall method are that the materials meet the physical and gradation requirements of the project specifications and that the bulk specific gravity of all the aggregate types as well as the specific gravity of the asphalt binder be determined (The Asphalt Institute, 1996).

In the Marshall method, the trial mix samples compacted with the compaction hammers are subjected to tests and analysis in the following order (The Asphalt Institute, 1996):

1. Bulk specific gravity determination,
2. Stability and flow test, and
3. Density and voids analysis.

As soon as compacted specimens cool to room temperature the bulk specific gravity of the mix is computed according to ASTM D 1188 (The Asphalt Institute, 1996). The Marshall Stability and Flow values are determined using the apparatus in Figure 2.5. A testing load is applied at constant strain to the specimen. The Marshall stability value is the maximum load (lbs. or N) subjected to the specimen before failure. The amount of vertical deformation of the specimen at failure is the Marshall flow value and it is measured in 1/100ths of an inch (The Asphalt Institute, 1989). The Density and Voids Analysis is performed as outlined by the Asphalt Institute (The Asphalt Institute, 1989).

Figure 2.5 Marshall Stability and Flow Test

To estimate the appropriate asphalt content specimens with asphalt contents both above and below an expected design asphalt content are subjected to the above tests and analysis.

The Asphalt Institute has stated some advantages and disadvantages of the Marshall mix design method (The Asphalt Institute, 1996).

Advantages:

1. The attention to density and voids properties of the asphalt mix ensures the proper volumetrics for a durable HMA.

2. The required equipment is both inexpensive and portable which lends itself good to quality control operations.

Disadvantages:

1. Many engineers believe that the impact used with the Marshall method does not simulate mixture densification as it occurs in a real pavement.
2. Marshall stability does not adequately estimate shear strength of HMA.

The result of these disadvantages is difficulty in accessing a mixture's susceptibility to rutting. Another disadvantage is that the Marshall mix design specifications are based on the density of a pavement under actual traffic. If the density of a pavement after trafficking is incorrectly predicted, the mix will not be adequately designed (Roberts, et al., 1996).

Similarly to the Hveem mix design, the Marshall mix design can be supplemented with performance tests to predict a pavement's in-service performance (The Asphalt Institute, 1996).

2.16.2.3 Superpave Mix Design

In 1988, the Strategic Highway Research Program (SHRP) was initiated with a primary goal of developing an improved mix design program. At the conclusion of the SHRP program in 1993, a system was developed that contained the following elements: a new grading system for asphalt binder, consensus properties for aggregate, a new volumetric mix design procedure, and mixture analysis procedures to estimate a pavements future performance. This system is referred to as the **Su**perior **Per**forming Asphalt **Pave**ment System (Superpave) (Roberts, et al., 1996). The unique feature of the Superpave system is that it is a performance-based specification system. The tests use physical properties that can be directly related to field performance (The Asphalt Institute, 1996).

The Superpave mix design system contains three distinct levels of design, termed level 1, level 2, and level 3. Different design levels are chosen based on the amount of traffic the pavement will be subjected to. The amount of traffic is estimated in terms of the number of 80 kN (approx. 18,000 lbs.) equivalent single axle loads (ESALs) the pavement will be subjected to in its designated service life. The level of design is chosen in accordance to Table 2.1 (Cominsky, et al. 1994). Each design level has a more reliable degree of performance prediction than the previous level (The Asphalt Institute, 1996).

Table 2.2 Recommended Design Traffic for Level 1, 2, and 3 Mix Designs

Design Level	Design Traffic (80kN ESALs)
1 (low)	$\leq 10^6$
2 (medium)	$\leq 10^7$
3 (high)	$>10^7$

2.16.2.3.1 <u>Level 1 (Low Traffic) Mix Design</u>

The key feature of the Superpave Level 1 mix design is the use of the Superpave Gyratory Compactor (SGC). While its main purpose is to compact test specimens, the SGC can also provided information about the compactability of an HMA. The SGC can identify mixtures that may exhibit tender behavior or compact to very low air voids under traffic and behave plastically (The Asphalt Institute, 1996). The characteristics of the SGC as well as the Superpave SGC compaction criteria are summarized in the next section.

2.16.2.3.1.1 The Superpave Gyratory Compactor

During the development of the Superpave mix design procedure SHRP developers had several goals in developing a laboratory compaction method. These goals included developing a compaction device that could compact specimens to densities that occur under actual temperature and loading conditions, be capable of handling large aggregates, be able to measure compactability, and be portable enough for quality control operations (The Asphalt Institute, 1996). The Superpave Gyratory Compactor (SGC) was developed to meet these goals.

The SGC is shown in Figure 2.6. It is a modification of the Texas Gyratory Compactor. The modifications included setting the angle of gyration to 1.25^{o}, gyration rate to 30 rpm, and the vertical pressure at a constant 600 kPa (87 psi). These new parameters are shown in Figure 2.7.

Figure 2.6 Superpave Gyratory Compactor

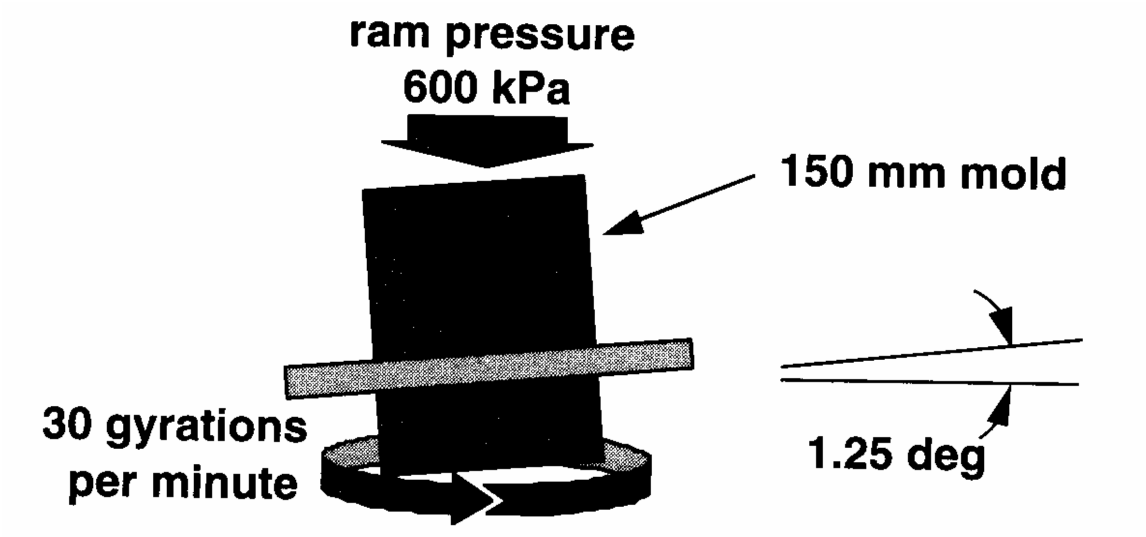


Figure 2.7 Superpave Gyratory Compactor Mold Configuration

Specimen compaction is done in accordance with SHRP Standard Method of Test M-002. The loose mix being compacted must be aged using a short-term aging procedure (SHRP Method of Test M-007). This simulates the aging of HMA paving mixes during field plant mixing operations and permits asphalt absorption to proceed to completion (Cominsky, et a., 1994).

One feature of the SGC is its ability to calculate specimen density (as a percent of its theoretical maximum specific gravity) throughout the compaction process. An additional feature of the SGC is it tends to orient the aggregate particles similar to that observed in the field (Roberts, et al., 1996).

In order to compact specimens to the same densities that will be obtained in the field, the amount of gyrations used to compact the specimen is adjusted. This is shown in Table 2.3. The amount of compaction, expressed as number of gyrations (N), is dependent on the expected high pavement temperatures and the traffic volume. As either of these variables increase, the compaction of the pavement in the field increases.

Table 2.3 Superpave Gyratory Compactor Compaction Characteristics (2)

Design ESALs (millions)	<39C			39-40C			41-42C			43-44C		
	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}
<0.3	7	68	104	7	74	114	7	78	121	7	82	127
0.3-1	7	76	117	7	83	129	7	88	138	8	93	146
1-3	7	86	134	8	95	150	8	100	158	8	105	167
3-10	8	96	152	8	106	169	8	113	181	9	119	192
10-30	8	109	174	9	121	195	9	128	208	9	135	220
30-100	9	126	204	9	139	228	9	146	240	10	153	253
>100	9	143	235	10	158	262	10	165	275	10	172	288

N_i, N_d, and N_m are three densities that need to be met during compaction. N_i (N-initial) is a measure of a mixture's compactability. Mixtures that compact too quickly are believed to be tender during construction and may be unstable under traffic (Roberts, et al., 1996). The bulk specific gravity of the specimen (G_{mb}) at N_i has to be less than 89% of the theoretical maximum density (G_{mm}). N_d (N-design) is the number of gyrations required to produce a density in the mix that is equivalent to the expected density in the field after the indicated amount of traffic (Roberts, et al., 1996). In the mix design process, and asphalt content is selected that will provide 4% air voids when the mix is compacted to N-design gyrations. N_m (N-maximum) is the number of gyrations required to produce a density in the laboratory that should absolutely never be exceeded in the field (Roberts, et al., 1996). The specimen must have 2% air voids (or have a G_{mb} less than 98% of G_{mm}) at N-maximum because mixtures with air voids lower than 2% the mix may behave plastically and be susceptible to rutting (Roberts, et al., 1996).

Level 1 mix design employs a performance-based asphalt binder specification with, empirical, performance-related aggregate specifications, and principles of volumetric mix design.

It is not possible to estimate the pavement performance of level 1-mix designs. However, the level 1 mix design provides a reasonable guarantee of adequate performance if all specified criteria are met (Cominsky, et al., 1994).

The main procedures of the level 1 mix design include choosing the correct binder grade, an adequate aggregate structure, and the optimum asphalt content using the Superpave Gyratory Compactor.

2.16.2.3.1.2 Superpave binder selection

Prior to the Superpave performance graded asphalt binder specifications, asphalt binders were primarily graded using the viscosity grading system, penetration grading system, or the AR (aged residue) viscosity grading system. These physical tests and specifications have many limitations (Roberts, et al., 1996):

- The tests are empirical and are not directly related to pavement performance.
- Tests are conducted at one standard temperature in spite of different climatic conditions at project sites. As a result binders with the same grading may perform differently over a range of temperatures.
- These grading systems do not consider long-term aging of the binder.
- These grading systems are not applicable to modified asphalt binders.

Recognizing the preceding deficiencies, SHRP developed the Superpave performance graded asphalt binder tests and specifications. This grading system addresses the limitations of the previous binder grading systems as follows (The Asphalt Institute, 1997 and Roberts, et al., 1996):

- The Superpave tests measure physical properties that can be related directly to field performance using engineering principles.

- The specified criteria remains constant, however, the temperatures at which these properties must be reached vary depending on the climate in which the binder will be used.
- Superpave binder specification utilizes the following test devices to simulate different stages of a binder's service life:
- The Rotational Viscometer simulates the original binder,
- The Rolling Thin Film Oven simulates the binder after construction, and
- The Pressure Aging Vessel simulates the binder after 5-10 years of in-service use.
- Tests and specifications are intended for both modified and unmodified binders.

A more detailed discussion of the Superpave binder test equipment and specifications are outlined by the Asphalt Institute (1997). The selection of the proper asphalt binder in the Superpave binder system involves the determination of the high and low temperatures the pavement will be subjected to, the traffic speed, and traffic loading. Methods of determining the asphalt binder grade are summarized by the Asphalt Institute (1997).

The Superpave mix design system has methods, which includes a selection procedure for an appropriate asphalt binder, for testing asphalt binder to see if it is susceptible to fatigue cracking. HMA is particularly susceptible to fatigue cracking at lower temperatures due to an increase in stiffness of the asphalt binder. The binder is aged in both the Rolling Thin Film Oven (RTFO-AASHTO T240 or ASTM D 2872) to simulate aging during manufacture and construction and the Pressure Aging Vessel (PAV-AASHTO PP1) to simulate aging in the field. The binder is then tested in the Dynamic Shear Rheometer (DSR-AASHTO TP5). Of particular interest is the value of $G^*\sin\delta$. This value is derived from the complex shear modulus and the phase angle that the binder undergoes throughout the DSR test. The maximum acceptable limit for G*sinδ is 5,000kPa. Lower values of these parameters are desirable to resist fatigue cracking in asphalt pavements.

2.16.2.3.1.3 Superpave Aggregate Selection

Consensus properties are properties that SHRP researchers believed were critical in designing a high performance HMA. These properties must be met at various levels depending on both the aggregates position within the pavement structure as well as the anticipated traffic volume. The consensus properties include coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and clay content (The Asphalt Institute, 1996). The test methods and specifications pertaining to consensus aggregate properties are detailed by the Asphalt Institute (1996).

Source properties are those that local governing agencies often use to qualify local aggregates. Superpave does not include specifications for source properties since they can vary from region to region or even locally. Superpave does, however, recommend that some source specific properties be taken into account during the mix design. These properties are toughness, soundness, and deleterious materials. Tests used to estimate these properties are also summarized by the Asphalt Instituted (1996).

To specify aggregate gradation, Superpave uses the 0.45 power gradation chart with gradation control limits and a restricted zone (Figure 2.8) to develop a design aggregate structure.

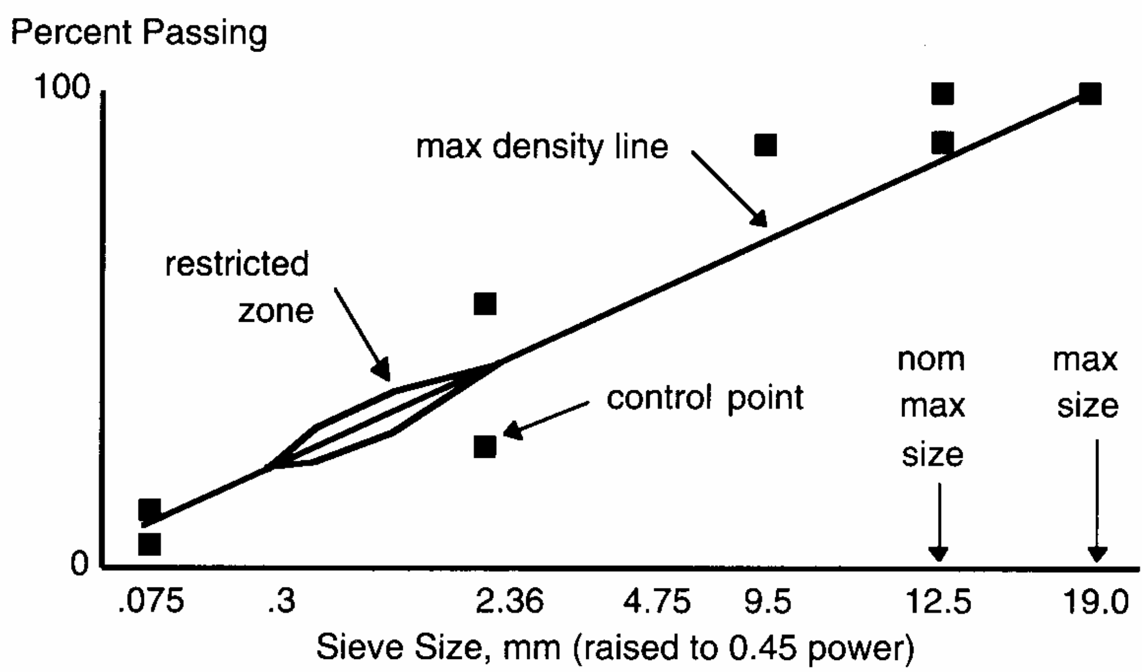


Figure 2.8 Superpave Gradation Limits

The Superpave design aggregate structure ensures that the aggregate will develop a strong, stone skeleton. The maximum density line represents a gradation in its densest possible form. The control limits act as a specification to ensure a dense aggregate structure. The restricted zone is specified in Superpave and is used to discourage oversanded mixes and /or mixtures that possess too much natural sand in relation to total sand. Gradations passing through the restricted zone often result in compaction problems and/or reduced resistance to permanent deformation. In addition to these factors, the restricted zone ensures that the fine aggregate does not follow the maximum gradation line, which may result in too little voids in the mineral aggregate (VMA). This may result in durability and stability problems (The Asphalt Institute, 1996).

2.16.2.3.1.4 Superpave Volumetric Mix Design

The Superpave level 1 mix design method includes three principal procedures (Cominsky, et al., 1994):

- Selection of materials,
- Selection of a design aggregate structure (expressed as an aggregate gradation),
- Selection of a design asphalt (binder) content.

The first step, the selection of materials, was summarized in the previous two sections. The second step, the selection of a design aggregate structure, is used to evaluate the effect of the aggregate structure on the mixture volumetric properties, predominately VMA. The objective is to select a design aggregate structure that meets the Superpave requirements. The requirements of the design aggregate structure are (Cominsky, et al., 1994):

- It must provide adequate VMA at the design number of gyrations and 4% air voids.
- It must meet density requirements at N_i (N-initial) gyrations.
- It must meet density requirements at N_m (N-maximum) gyrations.

The selection of a design aggregate structure first involves the selection of three or more aggregate blends that satisfy the requirements put forth in the previous section. A trial asphalt content is estimated for each aggregate blend and the asphalt-aggregate mixture is compacted in the Superpave Gyratory Compactor using the number of gyrations specified in Table 2.2. The compaction characteristics of each trial aggregate gradation are evaluated. First, the volumetric properties of the mixture at N_d gyrations are determined. Secondly, the densities at N_i and N_m are evaluated to determine the acceptability of the trial aggregate gradation as defined by the Superpave criteria. The aggregate blend that satisfies the Superpave criteria is chosen as the design aggregate structure. This procedure is detailed and the lab procedure and associated mathematical equations are summarized by the Federal Highway Administration (FHWA) (Cominsky, et al., 1994).

After the design aggregate structure has been established, the design asphalt content is determined. The design asphalt content is defined the asphalt content that provides 4% air voids at the N_d gyrations (Cominsky, et al., 1994). This is accomplished through the compaction of mixtures at, below, and above the estimated asphalt content and using the densification data (N_i, N_d, and N_m) generated by the SGC. This procedure is also summarized by the FHWA (Cominsky, et al., 1994).

2.16.2.3.1.5 The Determination of the Superpave Design Mixture's Susceptibility to Moisture Induced Damage

The final step in the Superpave Level 1 mixture design is to evaluate the moisture sensitivity of the design mixture. The test used to access the mixtures moisture sensitivity is AASHTO T 283, "Resistance of Compacted Bituminous Mixture to Moisture Induced Damage." The test involves the determination of the Indirect Tensile Strength of cylindrical specimens compacted to 7% air voids both before and after a laboratory freeze-thaw cycle. The end result is the tensile strength ratio (TSR): TSR=(Design Mixture IDT strength / Conditioned Design Mixture IDT strength) Superpave mixtures must have a design mixture TSR of 80%, minimum (The Asphalt Institute, 1996).

2.16.2.3.2 <u>Level 2 Mix Design (Intermediate Traffic Levels)</u>

The Superpave level 2-mix design system is used when traffic levels exceed 10^6 ESALs but do not exceed 10^7 ESALs. The intention of the analysis procedure is to ensure that Superpave mixtures designed in the level 1 volumetric mix design exhibit acceptable amounts of the distress types considered by SHRP researchers. The distress types considered in the level 2 mix design are permanent deformation, fatigue cracking, and low temperature cracking. In the design of a new pavement all three of the distress types are taken into consideration while only permanent deformation is predicted for asphalt overlays (The Asphalt Institute, 1996). Although the level 2 and 3 mix design procedures have not been implemented, a review of these procedures provides a foundation for implementation of reasonable procedures. It is noted that the primary concern in implementing both the level 2 and 3 procedures is that the procedures take roughly 2 ½ weeks to complete, not a practical time frame to perform a mixture design.

The level 2 analysis is performed using two performance test devices, the Superpave Shear Tester (SST) and the Indirect Tension Test (IDT). A table summarizing the performance tests performed as well as the temperatures at which the tests are performed is provided in Table 2.4. The results from the performance tests are used in the Superpave performance models. These are algorithms that predict pavement performance from the test results (The Asphalt Institute, 1996). The SST, IDT, and performance models are explained briefly in the following three sections.

Table 2.4 Superpave Level 2 Proposed Performance Tests (15)

Permanent Deformation Tests	Fatigue Cracking Tests	Low-Temperature Cracking Tests
Repeated shear at constant stress ratio (tertiary creep)	Simple shear at constant height at T_{eff}	Indirect tensile creep at 0C, -10C, and –20C
Simple shear at constant height at T_{eff}	Frequency Sweep at T_{eff}	Indirect tensile strength at –10C
Frequency sweep at T_{eff}	Indirect tensile strength at T_{eff}	Creep stiffness (S) and slope (m) of binder from bending beam test

2.16.2.3.2.1 The Superpave Shear Tester

The Superpave Shear Tester (SST) is a laboratory device used to estimate a paving mix's resistance to permanent deformation and fatigue cracking. Pictures of the SST are shown in

Figure 2.9. The SST device is designed to apply both vertical and horizontal (shearing) loads to a specimen that has been compacted in the SGC. A photo of a SST specimen is shown in Figure 2.10. These loads can be applied to the specimen simultaneously to simulate the compression and shear forces applied at the edge of a vehicle tire. The test device captures critical aspects of the asphalt mixture behavior including (Cominsky, et al., 1994):

- Dilatancy in shear,
- Stiffening with increased confining stress,
- Temperature and rate dependence, and
- The accumulation of permanent strain in a specimen under repetitive shear stress.

The level 2-mix analysis includes the following tests that utilizing the SST (Cominsky, et al., 1994):

- The Simple Shear at Constant Height Test
- Frequency Sweep at Constant Height Test
- The Repeated Shear at Constant Stress Ratio Test

Figure 2.9 Superpave Shear Test Device

Figure 2.10 Superpave Shear Test Specimen

2.16.2.3.2.1.1 Simple Shear at Constant Height Test

The simple shear at constant height test is used for permanent deformation and fatigue cracking analysis (The Asphalt Institute, 1996). The specimen is maintained at a constant height while a shear load is applied at 70 kPa/s. The load is applied rapidly so only the elastic response is recorded(i.e., virtually no creep occurs) (Cominsky, et al., 1994). This test is completed at temperatures corresponding to T_{eff}(PD) and T_{eff}(FC). T_{eff}(PD) is as defined before and T_{eff}(FC) is the effective temperature for fatigue cracking. It is defined as the single temperature at which an equal amount of fatigue damage would occur to that measured by considering each season separately through out the year (The Asphalt Institute, 1996). The shear stresses for the test performed at T_{eff}(PD) and T_{eff}(FC) are performed at 35 and 105 kPa, respectively. During the test, axial and shear loads and deformations are measured and recorded (The Asphalt Institute, 1996). The test duration is approximately 40 seconds, and is deemed not destructive to the test specimens. The specimens are next tested in the frequency sweep at constant height test mode.

2.16.2.3.2.1.2 Frequency Sweep at Constant Height Test

The frequency sweep at constant height is used for permanent deformation and fatigue cracking analysis. During the test a repeated shearing load is applied to the specimen to achieve a controlled shearing strain of 0.05 percent. An axial load is applied to the specimen to maintain a constant specimen height. One hundred cycles of the shearing load are used at the following frequencies: 10, 5, 2, 1, and 0. 5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The test is performed at T_{eff}(PD) and T_{eff}(FC). During the test the axial and shear loads as well as the deformations are measured and recorded (The Asphalt Institute, 1996). Analysis of the test data provides the phase angle (δ) and the complex shear modulus (G^*). δ is an indicator of the relative amounts of recoverable and non-recoverable deformation. G^* is a measure of the total resistance of the asphalt mixture to deformation. Like the simple shear at constant height test mode, the frequency sweep test is not destructive to the test specimens. The Superpave Shear Tester's effectiveness in predicting distresses will be examined later in this literature review.

2.16.2.3.2.1.3 The Repeated Shear at Constant Stress Ratio Test

The repeated shear at constant stress ratio test is a screening test to identify an asphalt mixture that is subject to tertiary rutting. Tertiary rutting occurs when an asphalt mixture densifies to a very low air void content under traffic, normally less than about two or three percent air voids. In this condition, the mixture exhibits extreme plastic flow with very few load applications and this will result in permanent deformation (The Asphalt Institute, 1996). The test is typically performed at high asphalt contents corresponding to three- percent air voids, which is extreme condition for tertiary rutting, or a lower air void. The test is performed at the control temperature (T_c) for permanent deformation. T_c is a function of the predicted traffic volume and the effective temperature for permanent deformation (T_{eff}(PD)). T_{eff}(PD) is defined as a single test temperature at which an amount of permanent deformation would occur equivalent to that measured by considering each season separately throughout the year (Cominsky, et al, 1994). This test accumulates permanent strain and is destructive to the test specimens. In this test, repeated synchronized shear and axial load pulses are applied to the specimen. The test specimen is subjected to between 5000 to 120,000 load cycles depending on traffic and climate conditions, or until accumulated permanent strain reaches five percent. The ratio of the axial to shear stress is constant throughout the test and is normally 1.2 to 1.5 (e). The data acquired during the test is plotted as shown in Figure 2.11. If the mixture exceeds the linear phase and enters the tertiary flow region, the mixture is unsuitable (Cominsky, et al, 1994). If the mixture fails the screening test, it is necessary to make adjustments to the mixture or redesign the mixture completely (Cominsky, et al, 1994).

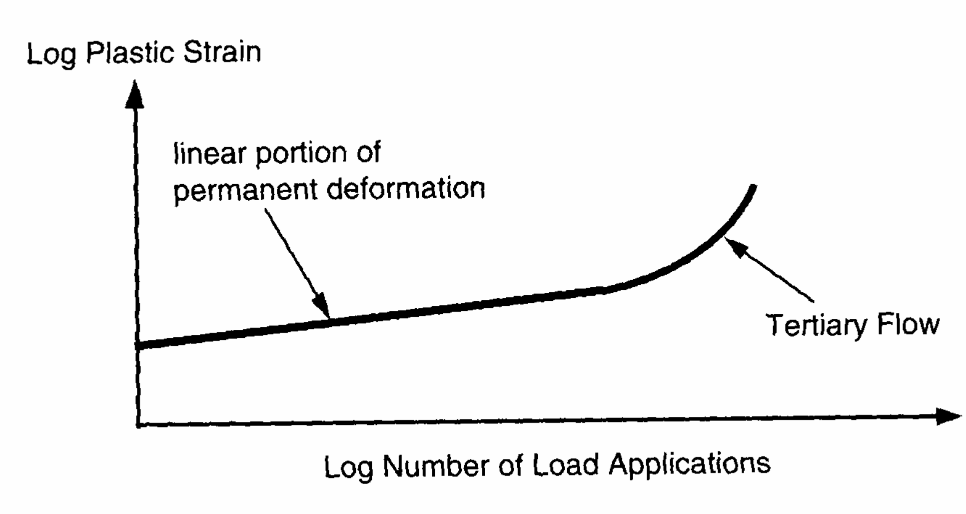
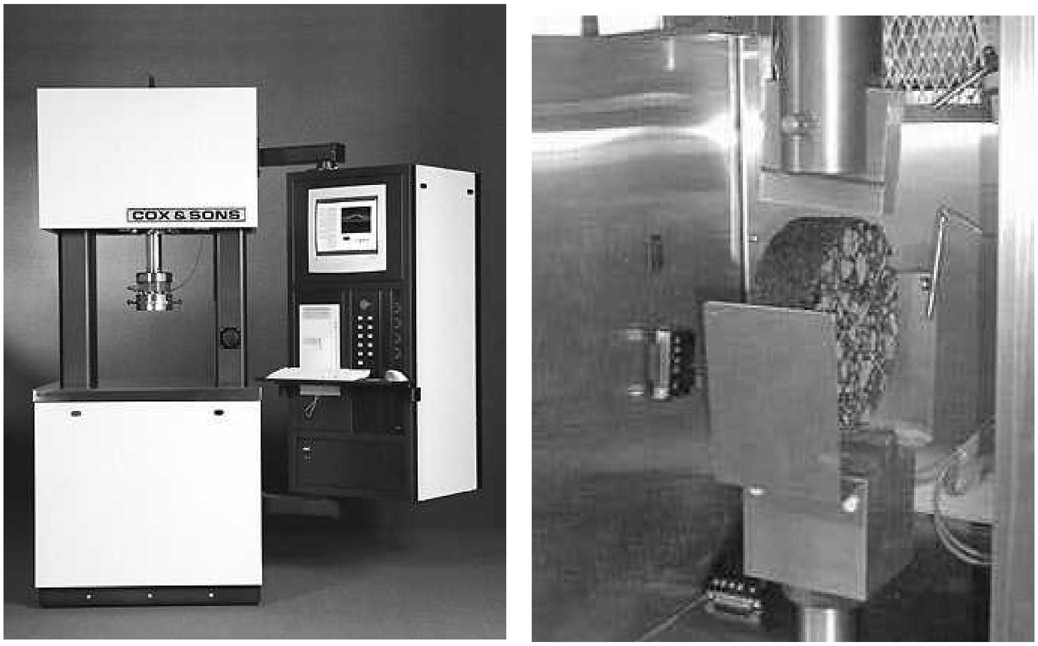


Figure 2.11 Tertiary Rutting

2.16.2.3.2.1.4 The Indirect Tensile Tester

The indirect tensile tester (IDT) is used to measure the creep compliance and strength of asphalt mixtures using indirect tensile loading at intermediate to low temperatures (>20^{o}C) (The Asphalt Institute, 1996). A photo of an IDT without an environmental chamber is shown in Figure 2.12a. Figure 2.12b shows a specimen in an environmental chamber.



(a) (b)

Figure 2.12 Indirect Tensile Test (a) and Specimen (b)

The specimen used for the IDT tests is the bisquette-like specimen compacted in the Superpave Gyratory Compactor. The IDT applies a compressive force along the cylindrical specimen’s diametral plane. This results in a near constant tensile stress within the specimen as shown in Figure 2.13. Analysis of the test results yield the stiffness master curve, the slope of the stiffness versus loading time relationship, and the tensile strength. The capability of a paving mix to resist the development of load and temperature-induced cracking can be estimated from these material properties (Cominsky, et al, 1994).

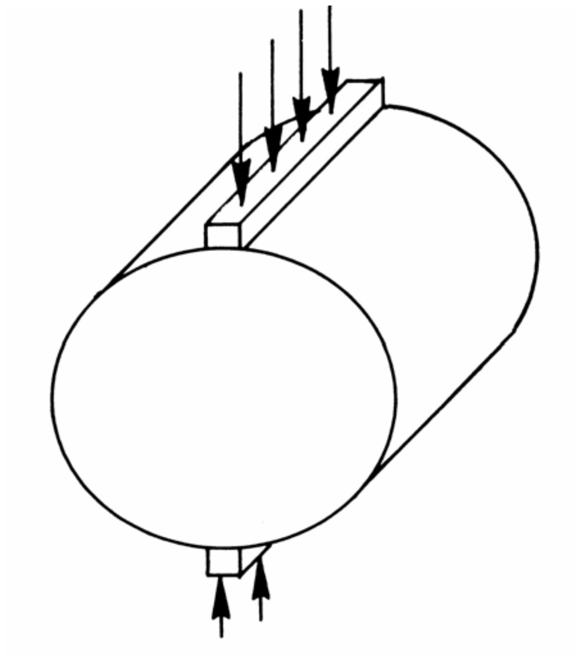


Figure 2.13 Indirect Tensile Test - Constant Tensile Stress

The two tests performed using the IDT during the level-2 analysis are:

- The Indirect Tensile Creep-Strength Test and
- The Indirect Tensile Strength Test.

The tensile strength test can be performed separately if only a fatigue cracking analysis is required. The Indirect Tensile Creep-Strength Test can be performed separately if only a low-temperature cracking analysis is required.

2.16.2.3.2.1.5 The Indirect Tensile Creep-Strength Test

This test is used to analyze mixtures for low temperature cracking. The creep portion of the test is performed at 0^o, -10^o, and -20^o C. The strength portion of the test is only required at -10 ^{o} C (The Asphalt Institute, 1996). Initially, a static creep load of a fixed magnitude is applied to the specimen. The load is selected to keep strains in the linear viscoelastic range (typically below 300 microstrain) and the load is sustained for 100s (Cominsky, et al., 1994). Following the creep loading, the specimen is loaded until failure by applying additional load at a rate of 12.5 mm per minute. This load is applied until failure. Failure is taken as the point where the load has decreased 10 percent less than the peak load (The Asphalt Institute, 1996).

The data collected from the creep portion of the test is the deformation vs. time data during the whole 100s test and the creep load. During the strength portion of the test, deformation vs. time data as well as the measured load vs. time data is recorded.

2.16.2.3.2.1.6 The Indirect Tensile Strength Test

The indirect tensile strength test can be utilized when only fatigue cracking needs to be predicted. The test is performed at T_{eff}(FC) when the level 2 analysis is being performed. In this test, the specimen is loaded at a constant deformation rate of 50 mm per minute until failure-peak load (The Asphalt Institute, 1996). The load vs. deformation data is recorded throughout the test (The Asphalt Institute, 1996). The indirect tension test (IDT) is used to test asphalt materials for the resilient modulus (M_R), which is an indicator of an asphalt mix's ability to resist cracking while being loaded. The complete description of this test can be seen in ASTM D4123, "Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures." The test involves compressively loading a cylindrical specimen (150 mm diameter and 50 mm thick) vertically in the diametrical direction with a haversine waveform. Five pulses are delivered to the specimen and the deflections that occur along the horizontal axis are measured. Based on the force applied and the deflections induced, the M_R is calculated. Typically, the M_R is higher at cooler temperatures and is lower at higher temperatures, due to the viscoelastic nature of asphalt.

2.16.2.3.2.2 Superpave Level 2 Performance Models

Although much attention was focused on the new test equipment developed by Superpave, the key component of Superpave performance testing is the performance models. These are algorithms that predict pavement performance using the data acquired during the SST and IDT testing (The Asphalt Institute, 1996). In addition to the test results, the performance models take into account the new asphalt mixture being designed and the characteristics of the in-place pavement. The performance models are all embedded into the Superpave software which is designed to guide the mix design process from beginning to end (Cominsky, et al., 1994). The end results of the level 2-mix analysis are the following graphs (Cominsky, et al., 1994):

- Total permanent deformation (mm) vs. asphalt content,
- Percent of pavement area fatigue cracked (%) vs. asphalt content, and
- Low-temperature crack spacing (m) vs. asphalt content.

Upon reviewing the distress vs. asphalt content graphs, the optimal design asphalt content is an asphalt content that satisfies the governing agency's criteria for all three distress types. The governing agency may specify only one or two of the distress types in the specifications. In this case, the asphalt content need only satisfy the specified distress type specifications.

If an acceptable asphalt content cannot be established that satisfies all of the distress factors being evaluated, the mix proportioning must be adjusted or a modifier or modified binder should be considered in the mix design (Cominsky, et al., 1994).

2.16.2.3.3 Level 3 (High Traffic) Mix Design

The level 3-mix design (or level 3-mix analysis) is to be used when the projected traffic volume is projected to exceed 10 million ESALs (The Asphalt Institute, 1996). The level 3 mix design is similar to the level 2 mix design procedure, except there is a more complete set of performance-based properties obtained through an increased number of performance tests for increased reliability. In addition to this, the performance models used to predict fatigue and permanent deformation are more comprehensive in level 3 than in level 2 (Cominsky, et al., 1994). Two additional SST performance tests are included: the uniaxial strain test and the hydrostatic state of stress (volumetric) test. The tests included in the level 3 analysis as well as the test temperatures are summarized in Table 2.5.

2.16.2.3.3.1 Performance Tests Utilizing the SST

The simple shear at constant height, repeated shear at constant stress ratio, and frequency sweep at constant height ratio are performed in the same manner as in the level 2 analysis except that they are conducted over a broader range of temperatures (Cominsky, et al., 1994). The two additional tests, the uniaxial and hydrostatic state of stress (volumetric) tests, are included to measure an additional material property, the nonlinear elastic behavior of the aggregate skeleton. This property is based upon the aggregate particles and is a measure of the increase of stiffness that results from the additional particle to particle contact occurring during specimen strain (Cominsky, et al., 1994). These additional tests are summarized in the next two sections.

2.16.2.3.3.2 The Uniaxial Strain Test

The Uniaxial Strain Test is used to generate data to predict both permanent deformation and fatigue cracking using the performance models. In this test the specimen is encased in rubber membrane and subjected to an axial load applied at a rate of 70 kPa/s. When the load is applied the specimen will react by bulging out and increasing its diameter. A confining load is applied to counteract this affect and keep the specimen circumference constant (The Asphalt Institute, 1996).

2.16.2.3.3.3 The Hydrostatic State of Stress (Volumetric) Test

The hydrostatic state of stress (volumetric) test is used to generate data to be used in the performance models to predict both permanent deformation and fatigue. In this test, a rubber membrane surrounds the specimen. The confining stress on all surfaces is increased at a rate of 70 kPa/s, and the change in the specimen's perimeter is recorded (The Asphalt Institute, 1996).

2.16.2.3.3.4 Performance Tests Utilizing the IDT

The performance tests performed utilizing the IDT during the level 3-mix analysis are conducted in the same way as in the level 2 analysis. These tests are required to be performed at a broader range of temperatures as shown in Table 2.5 (Cominsky, et al., 1994).

Table 2.5 Proposed Performance Tests for Superpave Level 3

Permanent Deformation	Fatigue Cracking	Low-Temperature Cracking
Repeated shear at constant stress ratio (T_{eff} (PD))	Frequency Sweep at constant height (4, 20, 40º C)	Indirect tensile creep (-10, 4, 20º)
Volumetric (4, 20, 40º C)	Indirect tensile strength (50mm/min) (-10, 4, 20º)	Indirect tensile strength (10, 4, 20º)
Uniaxial strain (4, 20, 40º C)		
Frequency sweep at constant height (4, 20, 40º C)		
Simple shear at constant height (4, 20, 40ºC)		

2.16.2.3.4 <u>Superpave Level 3 Performance Models</u>

The performance models written for the Level 3 analysis take into account the additional properties calculated in the two additional tests utilizing the SST. The performance model is more complex than the Level 2 performance model and is meant to predict pavement performance with an increased of reliability.

2.16.2.3.5 <u>Optional Proof Tests</u>

Proof testing provides an independent confirmation of routine Superpave test. Some situations where proof tests may be used to supplement the Superpave mix design system are as follows (Cominsky, et al., 1994):

- The pavement will need to meet severe service requirements,
- An exceptional degree of design reliability is required,

- A unusual or new materials is being used,
- Or when paving mixes have a top-size aggregate greater than two inches.

The following proof tests are recommended in the level 3 mix design to confirm the results of the Superpave analysis (Cominsky, et al., 1994):

- Wheel Tracking Devices
- Flexural Beam Fatigue Test
- The Thermal Stress Restrained Specimen Test

Much attention has been given to proof tests by State Highway Agencies (SHA) as a supplement to the Superpave Level 1 (Volumetric) mix design. The performance tests that are to be analyzed for use in the state of Michigan will be summarized later in this literature review.

2.16.2.4 Michigan's Mix Design Specifications

In the state of Michigan the asphalt the contractor performs the mixture design and it is developed utilizing Superpave mixture design criteria (MDOT, 1999). Aggregate consensus properties and gradation specifications are identical to the specifications written by the Asphalt Institute (The Asphalt Institute, 1996). Source aggregate property specifications include aggregate toughness and amount of soft (deleterious) materials (MDOT, 1999). The Superpave Mix Design Criteria for the state of Michigan is nearly identical to that developed by the Asphalt Institute. The Superpave criteria have been outlined by the state of Michigan and can be found in the "Special Provisions for Superpave Mixtures" (MDOT, 1999).

2.17 Quality Management of Hot Mix Asphalt Mix

Quality Management is the control of the HMA manufacturing and placement process to ensure high performance in HMA pavements (Decker, 1995). Most industries manufacturing raw materials, such as the HMA production industry, consider quality in the following three broad areas (Hughes, et al., 1997):

- Quality of Design – defines the stringency of the design requirements for manufacture of the product.
- Quality of Conformance to Design – defines how well the manufactured product conforms to the original design requirements.
- Quality of Performance – defines how well the product performs.

Control of the HMA manufacturing process ensures that the plant mixed HMA is representative of the HMA designed in the laboratory mix design. Control of the HMA placement process ensures that the construction of the HMA pavement provides a pavement that meets the owner's specification, the owners in most cases being the State Highway Agency (SHA).

There are several criteria to bear in mind when designing any asphalt mixture. The first criterion is resistance to permanent deformation (rutting). Rutting is a pavement distress in which the pavement exhibits shear flow and deforms laterally, which forms ruts in the pavement. HMA pavements are particularly susceptible to this distress in hot ambient temperatures, when the viscosity of the asphalt binder is lower than it would be in colder temperatures. The second criterion is fatigue resistance. Fatigue cracking occurs when a pavement is subjected to repeated loading over time. The third criterion is resistance to low temperature cracking. A proper selection of an asphalt binder that can withstand low temperatures will ensure a pavement that can perform well in a cold environment. The fourth criterion is durability. Durability is the ability of an asphalt mixture to perform over a period of time when subjected to various loads. Two important parameters known to be related to durability are the asphalt film thickness and the air void content (Roberts et al., 1996). The fifth criterion is resistance to moisture induced damage (stripping). By choosing an asphalt binder that is hydrophobic (one that is not attracted to water) the possibility for a loss of adhesion between the aggregate and the asphalt binder can be minimized. It is also known that moisture damage can be related to the choice of aggregate used in an asphalt mixture (Roberts et al., 1996). The sixth criterion is skid resistance. Traffic must be permitted to turn and decelerate normally without the possibility of skidding. Skidding can occur due to an improperly designed or constructed surface asphalt mixture. Aggregates should be resistant to polishing (a loss of friction). Also, an excessive amount of asphalt binder should not be used so that flushing (physically forcing the binder out of the mix upon compaction) does not occur. Both polishing and flushing contribute to a loss of friction between a vehicle's tires and the roadway. Finally, the seventh criterion is workability. Workability means that the asphalt mixture can be placed and compacted in the field with relative ease. Currently there are no laboratory diagnoses to address workability (Roberts et al., 1996).

2.18 Quality Management Specification Methods

A quality management specifications method is the framework of the system in which the control of the HMA production, placement, and the acceptance testing by the SHA is carried out. Three types of methods will be reviewed here: Quality control/Quality Assurance, Performance Related Specifications, and Performance Based Specifications.

2.18.1 Quality Control/Quality Assurance (QC/QA) Specifications

The most common approach to Quality Management involves Quality Control/Quality Assurance (QC/QA) specifications. QC/QA specifications define the contractor's responsibility for quality control or process control and the agency's responsibility for final acceptance (Scott, 1997). QC normally refers to those tests necessary to control a product and to determine the quality of the product being produced. QA refers to those tests necessary to make a decision on acceptance of a project and hence to ensure that the product being evaluated is indeed what the owner specified (Roberts, et al., 1996). Current QC/QA specifications are statistically based including random sampling, testing, and statistical analysis of selected material properties or workmanship to measure variability of process and product, define optimal quality levels, and develop price adjustments (Scott, 1997). In a survey of 40 states it was found that QC/QA specifications consistently found three fundamental measures for acceptance: mix properties, density, and smoothness. The survey also indicated that five measures are being used to determine specification compliance: average, quality level analysis, average absolute deviation, moving average, and range (Schmitt, et al., 1999). QC/QA methods utilizing the Hveem, Marshall, and Superpave mix design methods are summarized in the following two sections.

2.18.1.1 Quality Control/Quality Assurance based on Marshall and Hveem Mix Design Methods

An example of a procedure for the application of the Marshall and Hveem methods of mix design and QC/QA as developed by Harvey et al., as shown in Figure 2.14. Both QC and QA are based on the field mixed HMA meeting the properties established in the mix design (the Job Mix Formula (JMF)) within certain predescribed tolerances.

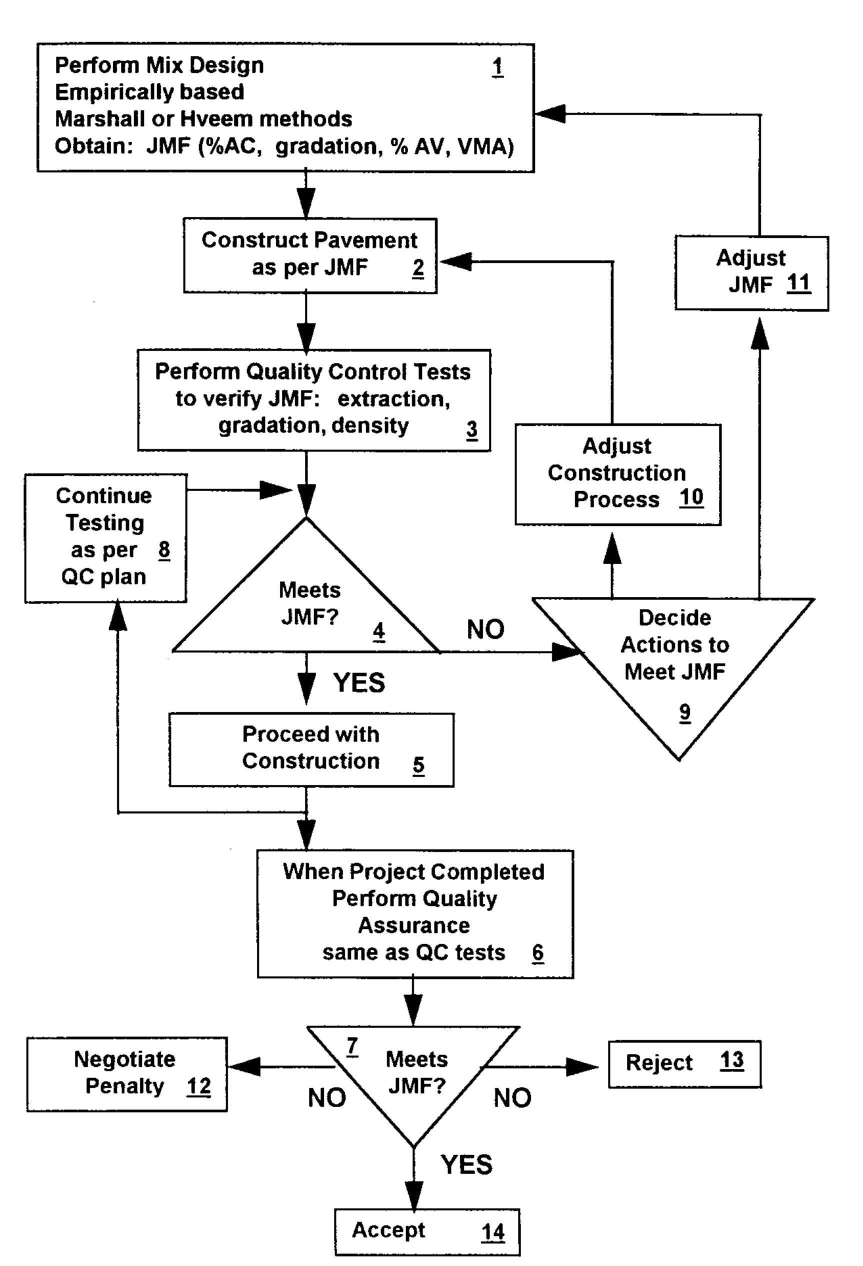


Figure 2.14 Existing QC/QA Systems

As described earlier in this literature review, properties in both Hveem and Marshall mix design methods are related empirically to HMA performance in the past. Because this QC/QA method is not based on unique in situ properties, but rather on past project performance, there are cases in which the JMF meets all criteria for construction quality and is with the tolerances of the JMF, yet it does not perform adequately. It is also difficult to predict what the effect of not meeting JMF tolerances has on future performance. This makes it difficult to access pay factors based on a loss of future performance. This QC/QA system, since it does not adequately predict performance, presents risk to the contractor in warranty situations, and results in higher bid prices (10).

2.18.1.1.1 <u>NCHRP Report 409, "Quality Control and Acceptance of Superpave-Designed Hot Mix Asphalt</u>

NCHRP Report 409, "Quality Control and Acceptance of Superpave-Designed Hot Mix Asphalt", presents a plan, in the form of a draft AASHTO standard practice, for quality control (QC) and quality acceptance (QA) of field production, placement, and compaction of HMA prepared in conformance with the Superpave Mix Design (7). The Contractor provides and maintains the QC system.

The QC of the asphalt binder is in accordance with AASHTO PP26-96, "Standard Practice for Certifying Suppliers of Performance-Graded Asphalt Binders. The Contractor must also develop a Laboratory Trial Mix Formula (LTMF) in accordance with Superpave mix design criteria and verify the field mixed HMA is with tolerances summarized in Table 2.2. In addition to this, the Contractor must demonstrate with the use of a control strip that the construction methods utilized will meet the density requirements required by the SHA. Since it is recognized that field properties differ from those calculated in the lab (D'Angelo, et al., 1991) the mix properties of the field verification of the LTMF and the density achieved in the compacted HMA become the target values for QC operations.

The unique characteristic of the QC operation is the use of Superpave Gyratory Compactor (SGC) properties. The properties used to control the HMA mixing process are the slope of the gyratory compaction curve (m) and the corrected, estimated mix bulk density (G_{mb}). Target values and standard deviations of these properties are calculated from the field verification and the first few days of production and if the properties fall out of control the asphalt content or aggregate gradation can be adjusted to provide mixture compliance. QA is performed by the SHA and consists of evaluating the percent of material or construction within a predetermined specification limits (PWL) established for the Superpave-mixed HMA.

2.18.2 Performance Related Specifications

A Performance Related Specification (PRS) system for pavement is "a method or model that allows pavement engineers to prepare practical construction specifications that focus heavily on the actual material properties and construction practices that have the most effect on the long-term performance of the pavement" (Seeds, et al., 1997). The PRS system provides a way to equitably reward or penalize the contractor for the as-constructed pavement delivered. These rewards and penalties are based on the monetary loss or gain of performance that will result from deviating from the "target property." The difference between a PRS and a QC/QA specification is that in the PRS the monetary loss or gain of performance as a result of missing the "target property" is known while in the QC/QA process it is not. The QC/QA fixes penalties/rewards arbitrarily while the PRS penalties/rewards are based on real costs of loss or gain in performance based on a life cycle cost analysis of the as-constructed pavement. The PRS is superior to QC/QA for that reason. A PRS can be based on either a volumetric property model, which considers only the HMA material properties, or a mechanistic-empirical model, which considers the entire pavement structure.

One of the Westrack (Seeds, et al., 1997) test track teams objectives is to develop a PRS for

HMA pavement construction by evaluating the impact on performance that deviations in materials and construction properties make. Based on data accumulated from the test track the Westrack team will:

- Generate a hot-mix asphalt specification, and
- After lot construction, calculate a bonus or penalty based on the as-constructed characteristics and the resultant life cycle cost.

When the research is through at Westrack, the specifications for HMA pavement will take the form of a comprehensive computer program, which may serve more as a research tool, and a series of equations and nomographs that will be more attractive to state highway agencies.

2.18.3 Performance Based Specifications

The philosophy of a performance-based specification is to design and construct and HMA pavement that will provided a required level of performance (Harvey, et al., 1997). This level of performance may include all or any combination of the following distresses: permanent deformation, fatigue, thermal cracking, or moisture damage. Rather than being based on material properties or construction practices the performance based specification is based primarily on whether or not the HMA pavement will perform in-service. The key to a performance-based specification is the performance test. . A performance test accesses the ability of a HMA to resist distress mechanisms, such as the environment or repeated loading, and predicts the future performance of an HMA pavement layer in terms or ESALs. An example of a performance test is the Asphalt Pavement Analyzer. The rut depth and the number of load cycles can be converted empirically to actual pavement rut depth and ESALs. In this case the rut depth and number of load cycles corresponding to maximum pavement rut depth and expected ESALs in the design life would be the design specifications. The Quality Management method is simply administering the performance test on a as-constructed HMA core and being sure the field mix meets the design specifications.

Performance based specifications are especially useful in warranty situations where the contractor agrees to warrant the HMA pavement against defect for a time period (typically 2 to 5 years (Scott, 1997)). In the warranty case there is no need for state quality assurance of quality since the contractor takes full responsibility for the pavement.

2.18.3.1 Performance Based Specifications Utilizing the SHRP A-003A Performance Tests

One performance-based specification that has gotten alot of attention in terms of research are the performance-based specifications based on the SHRP A-003A test methods and equipment (Solaimanian, et al., 1993, Harvey, et al., 1997, Tayebali, et al., 1993, Harvey, et al. 1996, Monismith, et al., 1993, and Sousa, 1994). The performance tests utilized in this specification are the Superpave Shear Tester (SST) and the Flexural Beam Fatigue Test (FBFT). These performance based mix designs follow the general steps outlined in SHRP, “Accelerated Performance-related Tests for Asphalt-Aggregate mixes and Their use in Mix Design and Analysis Systems” (Oregon State University, 1994).

Harvey et al. (1997), after performing the Superpave performance based mix design, reported that the ability of the SST and FBFT to meet the requirements as a performance prediction test was demonstrated and that the system appears to be ready to provide the builder with rapid feedback regarding the effects of changes in the construction processes on pavement performance.

2.18.3.2 NCHRP Report 409, "Quality Control and Acceptance of Superpave-Designed Hot Mix Asphalt"

At times, the guidelines given in NCHRP Report 409 may not detect that the HMA production has gone out of control. Therefore, field performance tests have been developed that the contractor may use in concert with the gyratory compactor to measure performance-based engineering properties for the purpose of QC (Cominsky, et al., 1993). In this case, the Superpave Shear Test (SST) can be run in accordance with the AASHTO TP7. In particular, the simple shear and frequency sweep at constant height tests are ran and compared to the results taken from the initial mix design. In addition to the SST, two new field grade performance tests have been developed to accompany the Superpave method of QC/QA: The Field Shear Device (a stripped down version of the SST) and the Rapid Triaxial Tester. The Field Shear Device can determine a HMA mixture propensity for rutting while the Field Shear Tester is used to ascertain material properties of the HMA for use in pavement models. Little research has been done on either of these tests as they have just been developed.

2.19 Pay Factors

Pay factors for a construction operation can be difficult to formulate. Hot mix asphalt can be a highly variable material (if the contractor does not have tight quality control over the material being produced), that can present a difficult problem for determining which performance characteristics should be used to derive the pay factor. Basically, the performance parameters that are believed to correlate with a certain type of distress are required to meet certain criteria or set with some bounds to define a pay factor. The contractor is then penalized for inferior performing materials or given a bonus for superior performing materials.

One of the most recent developments in the study of pay factors is the interaction between certain variables or performance parameters concerning a mix. It was typically the case that each performance characteristic was evaluated separately. For example, the air void content and the pavement stiffness could have an interaction effect on the overall performance of a specimen when it is being tested for fatigue. When a pavement layer has a high amount of air voids, the stiffness tends to be lower because the air voids create space and cannot transfer a load. Thus, a pavement layer with higher air voids may have a lower stiffness and may be more prone to fatigue cracking because of the inability of the air voids to transfer a load.

2.19.1 Methods Used for Determining Pay Factors

Many different methods can be used to derive a pay factor for any pay item. Usually these pay factors have some rational basis behind them. Statistical reliability can be used to ensure that a system will work with a certain probability of success or failure. Reliability is used to account for any variability of a certain distress (Lin et al., 2001).

Another method is to use composite weighting factors. This is when certain performance characteristics are combined together to formulate a pay factor. A single pay factor for a distress is calculated and then combined with weights to come up with an overall pay factor. The weights are subjectively assigned and therefore can be a drawback of this type of system. An example is shown below in equations 2.1 and 2.2.

$$PF = \frac{Fatigue_{as-designed}}{Fatigue_{as-built}} \qquad \text{Eqn. 2.1}$$

$$PF = w_{fatigue} PF_{fatigue} + w_{rutting} PF_{rutting} + \ldots\ldots + w_n PF_n \qquad \text{Eqn. 2.2}$$

Where:

$Fatigue_{as\text{-}designed}$ is the expected design fatigue life,

$Fatigue_{as\text{-}built}$ is the expected fatigue life of the constructed pavement,

PF is the pay factor corresponding to a certain HMA criteria, and

$W_{fatigue}$, $W_{rutting}$, W_n are the weights for given pay factors.

Another recommended method for pay factor calculation is a combination of Life Cycle Cost Analysis (LCCA) and Principal Component Analysis (PCA). LCCA is an analysis that compares different alternatives to a single problem using the total cost of a solution over the lifetime of the project (Lin et al., 2001). This method basically normalizes a distress parameter and then turns it into what are known as principal components (PC) using matrix algebra. The percentage of the total variance for each PC is calculated to weight a particular parameter in a specified pay factor equation. The pay factor is calculated as follows in equations 2.3 and 2.4:

$$PF = \min\left(1.05, \frac{B}{A}\right) \qquad \text{Eqn. 2.3}$$

$$Adjusted\ Payment = PF \times LBP \qquad \text{Eqn. 2.4}$$

Where:

A=a reliability level specified by the owner, (e.g., 95%),

B=reliability of the constructed pavement and

LBP is the lot bid price for the asphalt (Lin et al, 2001).

In summary, there are many different ways to calculate a pay factor. The most important point is to derive a pay factor that accurately reflects the significance of the pay item in question and its effect on HMA pavement life.

2.19.2 Different Highway Department Specifications and Pay Factors

In 1997 Schexnayder and Ohrn performed a case study that investigated different specifications and the pay factor criteria used for HMA by the Federal Highway Administration (FHWA), the Arizona Department of Transportation (ADOT) and the New Jersey Department of Transportation (NJDOT).

It was noted by the researchers that the agencies used different qualitative measures and characteristics for the purposes of justifying pay to the contractors. The FHWA used asphalt content, gradation and density to determine pay factors. This led the researchers to believe that the FHWA was more concerned with the appropriate amounts of materials being present in the HMA at the specified compactive effort. ADOT used density only as their critical payment criteria. The researchers concluded that this does not take into account other variable parameters (e.g. air voids and asphalt content). NJDOT used asphalt content, gradation, air voids, Marshall stability and pavement layer thickness. The Marshall stability is a parameter that refers to the pavement's ability to carry a load and the resulting deformation that accompanies the maximum load.

It was also noted by the researchers that the limits of acceptance within the specifications varied among the three agencies. Of primary concern was the difference for density requirements between the FHWA and ADOT. The FHWA had an acceptable allowance of 10% below the maximum density of the HMA, whereas ADOT allowed only 5%.

The agencies also had different methods of calculating pay factors. Both the FHWA and ADOT used statistical reliability, whereas the NJDOT used a straight cumulative percentage adjustment of the unit price for the HMA.

The final point of interest in the case study was the possibility of the contractor receiving a payment bonus. The FHWA was the only agency that awarded a payment bonus for superior quality materials and construction. ADOT only allowed a payment bonus in the event that it could be used to offset previous work on the same project that had been deemed inferior. The NJDOT only had negative pay adjustments with regards to changing the contractor's pay.

2.19.3 Discrepancies Encountered While Deriving a Pay Factor

As previously stated, pay factors should take into consideration the performance characteristics and the test methods to accurately measure them with regard to a certain pavement distress. This is important because contractors frequently raise questions with regard to the accuracy of a pay factor or the limits of a specified tolerance for a certain parameter (Schexnayder and Ohrn, 1997). This is especially true in the case where a material is to be removed and replaced.

Another common complaint is that many statistical quality assurance (SQA) plans do not recognize or give compensation for superior construction. Nichols Consulting Engineers, Chtd in 1998 included this topic in their study for the California DOT (Caltrans). The following is a summary of what was found.

From an owner agency's perspective, it should be many times harder for a contractor to get a bonus for superior construction. This should be reflected in the number of bonuses given. Ideally, every contractor should get a pay factor of 1, provided that the work is within specification and expected to perform according to the design. It was suggested that, if there are bonuses given, the odds of a contractor getting a higher bonus should decrease dramatically as the amount of the bonus increased. Another observation is that when contractors are required to perform Quality Assurance/Quality Control (QC/QA) work, the bid price often is more than expected. This is because the contractor is trying to ensure that he will be compensated for the extra work being performed during the construction project. The lot size and number of samples within a lot are also important when calculating pay factors. A lot should contain a single uniform independent sample of HMA. In other words the variance between material properties should be small within a given lot. Many statistical pay factors are dependant upon this assumption. If it is noted that a lot has a large number of samples, then the lot size should be reduced.

2.20 Performance Tests

The need has arisen in Michigan to identify a performance test to supplement the Superpave Level 1 (Volumetric) mixture design. A performance test can be used to access the ability of a HMA to resist distress mechanisms, such as the environment or repeated loading, and predict the future performance of an HMA pavement layer in terms or ESALs until failure. Performance tests evaluate material properties of HMA that can be related to particular distress types, including permanent deformation (rutting), fatigue, thermal cracking. The adverse effects of moisture sensitivity on these distresses should also be considered (Roberts, et al., 1996). The type of performance test used to determine these mixture characteristics depends on the following general criteria (Roberts, et al., 1996):

- Material variability and project size,
- The ability to estimate fundamental properties,
- Ease of testing,
- and the reproducibility of test results.

This literature review will cover research pertaining to the performance tests to be evaluated in this project: the Asphalt Pavement Analyzer, the Superpave Shear Tester, the beam fatigue test, and the indirect tensile test.

2.20.1 The Asphalt Pavement Analyzer

Pavement Technologies, Inc. first developed the Asphalt Pavement Analyzer (APA) in 1996 based upon the Georgia Loaded Wheel Tester. The Georgia Loaded Wheel Tester (GLWT) as developed in the mid 1980’s through a collaborative effort of the Georgia Department of Transportation and the Georgia Institute of Technology (Lai, 1986). The basis of its development was to perform efficient, effective, and routine laboratory rut proof testing and field production quality control of HMA (Lai, 1989). A photo of the Asphalt Pavement Analyzer is shown in Figure 2.15 (Cooley, et al., 2000).

Figure 2.15 Asphalt Pavement Analyzer

The APA applies a load to an aluminum wheel that lies upon a pressurized rubber hose (in Figure 2.15 the specimen tray is pulled out and the aluminum wheels and pressurized rubber hoses can be seen behind the specimen tray). The loaded aluminum wheel moves back in forth over the pressurized hose. The hose lies directly upon the HMA samples. The loaded aluminum wheel “imitates” a real wheel and leaves a rut. The depth of this rut is measured after a predetermined number of cycles (a cycle consists of two wheel loads) and can be used to access the HMA’s susceptibility to rutting. The APA is primarily used to predict rutting, but can also be used to predict moisture susceptibility and fatigue.

2.20.1.1 Compaction of HMA Specimens being Evaluated in the APA

The specimens tested in the APA can be either beam or cylindrical samples prepared in the lab or field cores and beams taken from the field. Lab and field mix beam and cylindrical specimens are typically compacted to 7% air voids (Messersmith, 2000). The two predominant sample types are the lab produced beam and cylindrical specimens. It has been shown that the two sample types provide different rut depths and stripping inflection points; however, both types generally rank mixes similarly (Miller, et al., 1995, Izzo, et al., 1999, Choubane, et al., 1998).

2.20.1.2 Preconditioning of HMA Specimens being Evaluated in the APA

It has been shown that when using the APA for rut testing it is adequate to condition the sample for 6 hrs. prior to testing at the test temperature (West, 1999). In a proposed ASTM test method for determining the rutting susceptibility using the APA, it is recommended that samples are preconditioned at test temperature for at least 6 but not more than 24 hours (Messersmith, 2000). The effect of AASHTO T 283 sample conditioning did not yield significant differences in rut depth in a study by Cross et al. (Cross, et al., 2000). For moisture testing, samples are generally preconditioned under water (Kandhal, et al., 1999) although no specific length of time for preconditioning has been determined.

2.20.1.3 Machine Settings Typically Used for APA Performance Testing

For rut testing using the APA, a wheel load of 445 N (100 lb.) and a hose pressure of 690 kPa(100 psi) are typically used (Cooley, et al., 2000, Messersmith, 2000), although 533 N (120 lb.) and 830 kPa (120 psi) have been used with success. The most recent standards state that the upper temperature of the standard Superpave binder performance grade should be used as the preconditioning and test temperature (Messersmith, 2000) although 60^{o}C has been used and resulted in a good correlation between laboratory APA rut depth and field rut depth (Williams, et al., 1999). 8000 loading cycles are usually used during the APA rutting test with measurements of the rut depth typically taken at 500, 2000, 4000, and 8000 cycles (Messersmith, 2000).

2.20.1.4 Research Associated with the APA's Ability to Predict Pavement Distress

Some of the advantages and drawbacks of the a Loaded Wheel Tester (LWT) were stated by West, et al. in 1991. The LWT is advantageous because:

- The principles of the test are straight forward (i.e. it is unnecessary to be familiar with engineering properties),
- The LWT realistically models a moving wheel load,
- The LWT is easy to operate,
- The LWT appears to correlate well with actual field performance, and
- The LWT is versatile (i.e., it can test under various temperature, loading, support, confinement conditions, and wet or dry).

The disadvantages as stated by West, et al. are that the relationship between field and LWT results is empirical and in 1989, the total time to mix, compact, cure and test a specimen took approximately 10 days. Currently, the using the APA predict pavement performance takes considerably less time.

Numerous studies have been conducted to compare the rut depths measured in the APA to actual field rut depths. In most of these studies the APA rut depths correlated well with rut depths measured in the field. In a study conducted by Williams et al. (1999) the APA was shown to correlate well with field rut depths. It was concluded that mixture design specifications for a performance based specification could be established for the APA and that using temperatures that reflect the in-service temperature of the pavement improves the correlation between lab

APA rut depths and field rut depths.

In studies performed in Georgia and Florida, the GLWT was able to rank mixtures similarly to their actual field performance (Lai, 1986 and West, et al., 1991). In another study in Florida, the APA ranked three pavements similarly to their known field performance and the author concluded that the APA had the capability to rank mixes according to their rutting potential (Choubane, et al., 1998). Miller, et al. (1995) reported increased correlation between lab rut depths and field rut depths with an increase in testing temperature from 40.6^{o}C to 46.1^{o}C. It was also discovered that surface treated HMA was sensitive to rutting and it may be necessary to remove the treated portion of the HMA core before testing in the GWLT. Lai (1993) indicated that GLWT rut depths are very sensitive to beam density and as a result variability of measured rut depths between labs was quite high.

The objective of NCAT Report No. 99-4, "Evaluation of Asphalt Pavement Analyzer for HMA Mix Design" (Kandhal, et al., 1999), was to demonstrate the APA's sensitivity to gradation and binder type and to determine a pass/fail rut depth criterion. The following conclusions were made as a result of this study:

- The APA is sensitive to aggregate gradation.
- The APA is sensitive to performance grade of binder.
- The APA had a fair correlation with the Superpave Repeated Shear at Constant Height test – both performance tests characterized the HMA specimens in the same way.
- It appears that the APA has the potential to predict relative rutting potential of HMA mixtures.
- A tentative pass/fail rut criteria was determined to be between 4.5-5 mm at 8000 cycles.

Cross et al. (2000) studied the use of the APA to predict moisture susceptibility of HMA utilizing different methods of specimen preconditioning. After preconditioning the HMA in a 40^{o}C water bath for 2 hours and running the APA with the specimen submerged in 40^{o}C water it was determined that an APA rut depth of 5.00 mm differentiated mixtures that passed and failed the AASHTO T 283 moisture sensitivity test.

2.20.2 Superpave Shear Tester (SST)

The scope of Strategic Highway Research Program (SHRP)Project A-003A, "Performance-Related Testing and Measuring of Asphalt Aggregate Interaction and Mixtures" included the development of laboratory performance tests for HMA. Both the Superpave Shear Tester (SST) and the Flexural Beam Fatigue Test (FBFT) were developed at the University of California-Berkley as part of the SHRP program. SHRP concluded that the most promising test method to evaluate shear susceptibility, and consequently permanent deformation, was the SST (Anderson, et al., 1999). The SST was originally intended to measure material properties of HMA for use in the performance prediction models used in Levels 2 and 3 of the Superpave mix design and analysis system (Cominsky, et al., 1994 andMcGennis, et al., 1995). However, work done by the University of Maryland's Model Evaluation Contract (FHWA, 1995) as well as experimental work done by other researchers (Pellinen, et al., 1996 ,Zhang, 1997, and May, et al., 1995) has shown that the existing Superpave models do not always provide reasonable performance predictions when compared with expected trends, accelerated road tests, or controlled pavement sections. Although the performance models do not predict pavement performance, the output of the Superpave performance tests can be used to calculate mechanical properties that permit relative determinations of an asphalt mixture's stiffness and estimations of its ability to withstand permanent deformation. Although initially it was intended for the SST to characterize HMA with several tests, only three of these tests have been used regularly to ascertain HMA's resistance to permanent deformation: the Simple Shear at Constant Height Test, the Repetitive shear at Constant Height Test, and the Frequency Sweep at Constant Height Test. A key assumption made while testing with the SST at constant height is that permanent deformation is primarily a plastic shear flow phenomenon at constant volume occurring near the pavement surface and caused by shear stresses below the edge of truck tires (Anderson, et al., 1999). The axial stress applied to the HMA specimen to keep the specimen height constant keeps the specimen from dilating. Dilation is the result of aggregate trying to roll over one another while being sheared. The inability of the specimen to dilate results in increased shear stiffness. It is thought that shear strain in an actual pavement is confined to a constant volume (i.e. a HMA mixture that is resistant to permanent deformation will provide a confining pressure that will provide additional shear stiffness). This is the basis of why the SST holds specimens at constant volume while administering a shear stress to the specimen.

2.20.2.1 Compaction of HMA Specimens being Evaluated in the SST

For test results to be meaningful, specimens prepared in the laboratory must resemble as closely as possible the in-service HMA. The compaction method has been found to significantly affect the permanent deformation properties of HMA evaluated using SHRP A-003A test methods and equipment (Sousa, et al., 1991 ,Harvey, et al., 1994). The Rolling Wheel Compactor (RWC) is recommended for use by the SHRP A-003A project team for the following reasons (Sousa, et al., 1993):

- It generally produces specimens whose permanent deformation characteristics lie in between those of specimens produced by gyratory and kneading.

- It produces a homogenous aggregate and air void structure.
- It has all cut surfaces.
- The air voids can be reasonably controlled.
- It is a comparatively easy procedure to accomplish.
- It enables the rapid fabrication of a large number of specimens in a wide array of shapes.
- The procedure is intuitively similar to field compaction.

The RWC has been found to produce specimens with permanent deformation performance similar to field cores in the Repeated Shear at Constant Test (Sousa, et al., 1996). Although the RWC may be superior for A-003A testing methods, the Superpave Level 1 (volumetric) mix design procedure uses the Superpave Gyratory Compactor (SGC) and as a result research has been carried to show its applicability to testing with the SST. Both the Texas Gyratory Compactor (TGC), from which the SGC was developed, and the kneading compactor produce specimens with different aggregate orientation and air voids near the mold surfaces. This is undesirable so it may be beneficial to saw away the faces near the mold. SST specimen sawed from a TGC specimen has a relatively homogenous aggregate orientation and air void structure (Harvey, et al., 1994). Specimens compacted in different brands of SGC (Pine or Troxler) when tested in the SST have relatively the same mechanical properties (Anderson, et al., 1999). The kneading compactor produces specimens that are more resistant to shear deformation and have a greater dilantancy when tested in the SST then the SGC or RWC (Harvey, et al., 1994). Harvey et al (Harvey, et al., 1993) found that testing specimens with air voids similar to those that will be found in the field after traffic induced compaction is appropriate. He estimated this could be achieved by compacting the specimen to 3% air voids. RWC specimens compacted in 7.5 cm lifts have little air void gradient and no mold effects (Harvey, et al., 1993).

Compaction Methods cannot be used interchangeably when testing using the SST because they produce specimens significantly different with respect to shear deformation properties (Sousa, et a., 1991 and Harvey, et al., 1994).

2.20.2.2 Preconditioning of HMA Specimens being Evaluated in the SST

According to the Standard Practice for Measurement of the Permanent Deformation and Fatigue Cracking Characteristics of Modified and Unmodified Hot Mix Asphalt (Harrigan, et a., 1994), specimens to be tested in the SST should be conditioned at all test temperatures for at least 2 hours. This is appropriate except for testing over 40^{o}C. In this case the specimen should be conditioned for at least 2 hrs. , but not more than 4 hrs.

The sample type, whether it be an original mix or a reheated mix, does not appear to effect properties obtained from the Frequency Sweep or Simple Shear Tests performed at Constant Height (Anderson, et al., 1999).

Romero and Mogawer (1997) determined that because of the inherent variability in mixture testing in the SST, it is recommended that the statistical sample sizes (i.e. the number of specimens) be examined, and the mean HMA property values of more than three replicate samples be tested to improve mixture performance rankings.

2.20.2.3 The Simple Shear at Constant Height (SSCH) Test

A detailed procedure of SSCH Test is described in AASHTO TP7-94, "Standard test Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt Using the Simple Shear Testing Device." The SSCH test consists of applying a shear stress to the sample at a rate of 70 kPa/s up to 35 kPa for 40^{o}C and up to 15 kPa for 58^{o}C. This stress level is maintained for 10 s, after which the stress reduced to 10 kPa at a rate of 25 kPa. The SST has the ability to apply an axial load to the specimen to keep the specimens height constant throughout the test. The test is considered non destructive. The HMA properties analyzed in this test are (Romero, et al., 1997):

- Shear Modulus Parameter – it is believed that this value can provide a measurement of the stiffness of the mixture. Mixtures with a high Shear Modulus Parameter are expected to be rut resistant.
- Percent Recovered Strain – Provides an estimate of the resiliency of the mixtures. Mixtures with high recovered strains are expected to show less rutting.
- Maximum Axial Stress – is believed to be the result of the aggregates trying to roll past each other and is a measure of the ability of the mixture to develop confining stresses when it is subjected to shear strains (Williams, et al., 1998). It is not known how this parameter relates to pavements.
- Maximum Shear Strain (γ_{max})

Romero and Mogawer (1997) in an effort to evaluate the ability of the SSCH test to differentiate mixtures with different nominal maximum aggregate size found that the test was unable to differentiate between mixtures used no matter what binder type was used.

Based on data accumulated from a variety of projects, Anderson et al. (1999) found that with an increase in asphalt content $\gamma_{\mu\alpha\xi}$ increased. Utilizing the SGC, it was found that γ_{max} was greater for lab prepared mixes then for those prepared in the field.

2.20.2.4 The Repetitive Shear at Constant Height (RSCH) Test

The RSCH test consists of applying a repeated haversine shear load while holding the specimen height constant. In literature reviewed the load cylce is normally a 0.1 s loading and

0.6 s rest period. This process is repeated up to 5,000 cycles or until the permanent shear strain reaches a predescribed limit, shown in the literature to be anywhere from 2 to 5% strain. Test temperatures were shown in the literature to coincide with the upper PG grade temperature or the mean maximum 7-day pavement temperature. Sousa et al. suggests formulas developed by Solaimanian and Kennedy (1993) to obtain the maximum 7-day pavement temperature for any depth less than 8 in into the upper pavement layer. This test is considered destructive to the specimen. The two parameters normally recorded during the RSCH test are the cumulative permanent shear strain and the resilient shear modulus.

The RSCH is the only test performed utilizing the SST that seems to have gained favor in the HMA industry. There has been much research pertaining to the use of the RSCH in a performance based mix design.

Sousa et al. (1993) suggested that it appears to be desirable to use a single test with the requirement that it be sensitive to the most important aspects of the mix behavior, rather than a battery of tests as suggested in Superpave. The RSCH seems to meet these requirements on cylindrical specimens because:

- Specimen geometry - a specimen 6 in diameter and 2 in. high can be cored from the as-built pavement or it can be prepared in the laboratory utilizing the SGC or the RWC.
- Rotation of principal axes – It is a simple test that permits controlled rotations of the principal axes of strain and stress, which is important in predicting permanent deformation.
- Repetitively applied loads – Studies have suggested that repetitive loading rather than creep loading is required to define the propensity of a HMA mixture to rut.
- Dilation – Since it is believed that HMA in a properly designed mix is confined against

dilantancy, the confining pressure applied axially to the specimen may simulate an actual HMA pavement.

Partl et al. (1996) found a good relationship between RSCH results and observed rutting in the field. Surface courses less susceptible to rutting generally exhibited higher RSST-CH resilient shear modulii. It was also documented that RSCH results were very susceptible to test temperature.

Harvey and Monismith (1994) determined that the results from RSST testing were generally most sensitive to binder type, aggregate type, fines content, air voids, and compaction.

Tayebali et al. (1999), found that the RSCH test was able to identify well performing vs. poorly performing mixes based on a study of three pavements of known field performance in North Carolina. It was also found that testing must be done at the proper temperature, similar to the findings of Partl et al. The temperature suggested in this study was the maximum 7-day pavement temperature predicted in accordance with AASHTO TP-7 Procedure F. This temperature should be based on a reliability level of 50 percent at a depth of 20 mm.

2.20.2.5 The Frequency Sweep at Constant Height (FSCH) Test

The FSCH procedure can be found in AASHTO TP7, Procedure F. It is a strain-controlled test that is executed by applying a sinusoidal shear load to the specimen. The peak strain amplitude, as specified in AASHTO TP7, is 0.0005 mm/mm. The specimen is maintained at a constant height with an axial pressure. Ten loading frequencies (from 10 to 0.01 Hz) are used to complete the frequency sweep test. It is considered a non-destructive test. The output can be used to determining the following mixture properties at each different temperature and frequency (Romero, et al., 1997):

- The complex shear modulus (G*),
- The phase angle (δ),
- The "recoverable" shear modulus (G'),
- The "loss" shear modulus (G"),
- The slope of log G* vs. log frequency – the slope relates the change in G* to loading frequency. It has been suggested that higher slopes can indicate HMA mixes less resistant to rutting (Zhang, et al., 1996),
- The Complex Modulus at 10 Hz (G^*_{10Hz})

Romero and Mogawer (1997) indicated that G^*_{10Hz} had the greatest ability to differentiate between mixtures made with different binders and the same aggregate. Anderson et al. (1999), after analyzing data from a variety of projects, found as asphalt increases G* decreases and also that field mixed SGC samples have lower G* than lab mixed samples.

Williams et al. (1998) reported that G^*_{10Hz} caught the binders influence on mixture stiffness when compared to Dynamic Shear Rheometer data at same temperature and frequency. It was also found that FSCH data did not agree with pavement performance taken from Westrack, a pavement testing facility in Nevada.

2.20.3 Flexural Beam Fatigue Test (FBFT)

The scope of Strategic Highway Research Program (SHRP) Project A-003A, "Performance-Related Testing and Measuring of Asphalt Aggregate Interaction and Mixtures" included the development of laboratory performance tests for HMA. Both the Superpave Shear Tester (SST) and the Flexural Beam Fatigue Test (FBFT) were developed at the University of California-Berkley as part of the SHRP program. SHRP concluded that the most promising test method to evaluate fatigue life was the Flexural Beam Fatigue Test (FBFT) shown in Figure 2.16. Research has been conducted on asphalt binder itself and the interaction of variables in an HMA mix to develop tests to predict fatigue life. Performance tests have been developed and included in specifications to help ensure a high quality material. A performance-based specification (PBS) involves engineering properties that are believed to be predictors of performance (Lundy 2001). A mathematical model is used to calculate the performance predictor. Parameters such as cycles to failure or resilient modulus are examples of predictor variables.

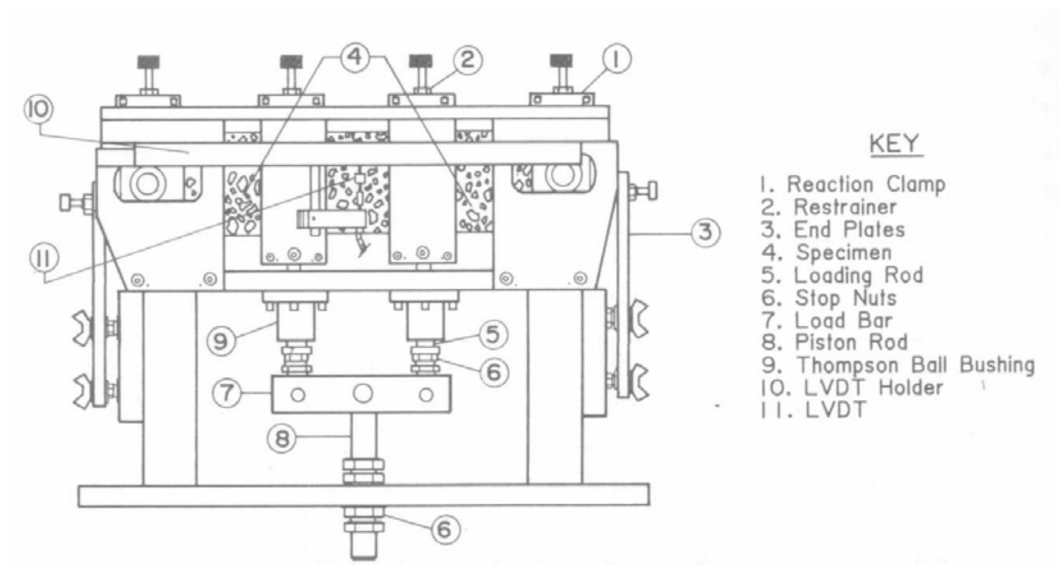


Figure 2.16 Flexural Beam Fatigue Test

The Flexural Beam Fatigue Test is based upon a 4 point loading system originally developed by Deacon (1965) and later modified by Epps (1969). The FBFT is equipped with a servohydraulic-controlled dynamic flexural fatigue test module that has increased the precision of the 4-point loading system, decreasing the coefficient of variation from 93% to 41% (54). Another action taken was increasing the size of the specimen to 2.5"W X 2"H X 15"L. As result of the improved loading system and the larger specimen, a smaller amount of specimens need to be tested to obtain the fatigue properties of an HMA pavement. As a result of this increased precision, a fatigue test procedure has been developed that can determine the fatigue relationship for a given mix and temperature in as few as 24 hrs (Tayebali, et al., 1994). In the past, the same task may have taken up to two weeks (Roberts, et al., 1996). The flexural bending beam test is the most common test method to determine the fatigue life of a pavement. It estimates the number of loading cycles the pavement can endure before fracture occurs. The method used here can be seen in its entirety in AASHTO TP8, Standard Test Method for Determining the Fatigue Life of Compacted Hot Mix Asphalt Subjected to Flexural Bending. This test system is similar to a system developed by Deacon for his doctoral research in 1964. The beam fatigue apparatus subjects asphalt beams that are 50 mm high (+/-5) x 63 mm wide (+/-5) x 400 mm long to a constant rate of microstrain. The microstrain refers to the magnitude that the beam is deflecting (e.g. 600 microstrain means 0.0006 in/in or mm/mm). The load the sample is subject to decreases as the test progresses with time and increasing cycles. Both the stiffness and the number of cycles are recorded. The failure point is defined as when the beam reaches 50 % of its initial stiffness. A plot of log cycles versus microstrain is then made. From this information the maximum flexural stress a pavement can endure is calculated. By knowing the relationship between the stresses or strains for a pavement and the repeated cycles to failure, the number of traffic loads to failure can be estimated. The test itself is typically used to compare different mixtures to give an indication of relative performance (Roberts et al. 1996).

2.20.3.1 Compaction of the HMA Specimens being Evaluated in the FBFT

Similar to the SST, the compaction method of choice for the FBFT is the Rolling Wheel

Compactor (Sousa, et al., 1993, Sousa, et al., 1996 and Tayebali, et al., 1997). Tayebali et al. (1997) found that utilizing the RWC resulted in a 33% decrease in coefficient of variation over the kneading compacted specimens. This means that twice as many specimens would have to be compacted with the kneading compaction machine to get the same precision as the RWC. RWC specimens compacted in 7.5 cm lifts have little air void gradient and no mold effects (Harvey, et al., 1993).

2.20.3.2 Preconditioning of HMA Specimens being Evaluated in the FBFT

Preconditioning of the HMA specimens to be tested in the FBFT consists of Superpave short term aging (SHRP M-007), compaction using RWC, and conditioning at test temperature for at least 2 hrs. before the test (Tayebali, et al., 1994).

2.20.3.3 Machine Setting's Typically Used for FBFT Performance Testing

According to SHRP test M-009 (Harrigan, et al., 1994) the test temperature is 20^{o}C. According to M-009, the load is to be a repeated sinusoidal load between 5-10 Hz, although most research encountered in this literature review used a frequency of 10 Hz. The test is a controlled-strain test whose deflection is chosen corresponding to a deflection that will permit at least 10,000 cycles until failure. Failure is taken when the beam stiffness is 50% of its initial stiffness. The initial stiffness is measure on the 50th load cycle.

Much discussion has been directed to the relative performance of mixtures in controlled-stress versus the controlled-strain mode of loading, with controlled stress loading conditions applicable to comparatively thick and stiff asphalt- bound layers and the controlled-strain mode of loading applicable to thinner and more flexible asphalt-bound layers. Tayebali et al. (1993) suggest that the relative ranking of the mixtures in both modes of loading is the same when comparisons are made of the performance of various mixes in representative pavement structures.

2.20.3.4 Research Associated with the FBFT's Ability to Predict Pavement Distress

Perhaps the greatest attribute of the FBFT is that the test measures a fundamental property. This enables the test results to be to be used in prediction models in the mixture evaluation and design process (Tangella, et al., 1990). As shown earlier in this literature review, the number of cycles until specimen failure can be correlated empirically with the number of ESALs until fatigue cracking initiates in the in-service pavement. This relationship can be used in a performance-based specification.

Some inherent disadvantages have been as shown by Tangella et al. (1990): Relating the lab results to field performance can be difficult, the state of stress being tested is uniaxial and isn't representative of the real pavement stresses, and the elastic theory is assumed to predict stresses and strains in the specimens when the material is actual viscoelastic.

In the SHRP program entitled "Performance Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures," much was discovered about the advantages and disadvantages of using this test (Tayebali, et al. 1994). The results must be interpreted using a mechanistic analysis to take into account effects such as loading, pavement structure, environment, and the mechanical properties of the mix (i.e the Modulus of Elasticity). Although the beam specimens in the study were sensitive to most mix variables the test didn't reasonably demonstrate the effect that the asphalt content has on fatigue life. The fatigue life estimated using SHRP procedures and the FBFT properties was found to correlate with observed pavement performance in a number of full-scale testing facilities when using conventional asphalt cement. When modified binders were used, however, the FBFT was unable to rank fatigue performance properly. In a report by Harvey and Tsai (1997) compaction was found to effect fatigue properties. With an increase of compaction, accompanied by a lower air voids, fatigue performance is improved. An increase of fatigue life of 10-20% was estimated for each increase 0.5 for the same pavement structure, also.

Ideally, an HMA mixture will perform well for a given environment, traffic loading and service life. An HMA mixture should consist of an optimum asphalt content, air void content and gradation that will not experience excessive distresses. However, this is not always the case. The following paragraphs provide a few examples of preventative measures and repair for fatigue cracking.

Fatigue cracking can be prevented by constructing a high quality pavement. Thicker asphalt layers have been shown to resist fatigue cracking as opposed to thinner layers (Roberts et al. 1996). This is especially true of overlays. If the asphalt layer is not thick enough in an overlay pavement, the material (especially if it is an older, unstable pavement) may develop fatigue cracks that reflect up to the surface layer.

Adequate drainage is another way to resist fatigue cracking (Roberts et al. 1996). It is necessary to have a strong underlying aggregate base system that will not be eroded away by the infiltration of moisture. If these aggregate layers are severely damaged, the asphalt will not be supported and the pavement system could fail in fatigue.

Finally, a higher quality asphalt mix that is designed to perform well under heavy, repeated traffic loading may also perform well in terms of fatigue. Even if a very thick pavement is not an option for whatever reason, a pavement can usually be designed to resist fatigue cracking when the proper performance characteristics are considered. The right gradation, asphalt content and air voids are all important considerations when designing an HMA pavement to resist fatigue (Roberts et al., 1996).

Fatigue cracking can be repaired in many different ways. One of the major disadvantages that fatigue cracks present is that the pavement layer can be cracked all the way through to the aggregate base. This could mean that the entire pavement layer must be removed and replaced, as opposed to permanent deformation that can sometimes be fixed simply by milling off the top layers and filling them in with a new hot mix.

Another method that is used to repair the damage done by fatigue cracking is an asphalt-based crack sealant; even though this method is not the preferred solution as is removal and replacement (Roberts et al., 1996). There are many different sealants available that can be used effectively. One problem, however, is that the overuse of a crack sealant may present a friction problem with traffic when trying to decelerate on wet pavement.

2.21 Current Research Findings Concerning Fatigue Cracking

Recently, much research has been done with regard to the cause and prevention of fatigue cracking in asphalt pavements. Current research has focused on several things. Environmental conditions are very important to pavement performance (Roberts et al., 1996). Performance parameters that can be used as predictors for fatigue life have been modeled by numerous people. The quality of the construction operation can have a profound effect on the lifetime of a pavement (Deacon et al., 1997). The actual amount of traffic as opposed to the design level of traffic and loading can also be very influential on the life of a pavement.

As was previously stated, the environment that a pavement is exposed to must be considered when that pavement is being designed. Moisture damage to the HMA and to layers (the aggregate base, subbase and the existing subgrade) can be detrimental to an asphalt pavement.

In terms of a mix design most of the findings seem to conclude the laboratory fatigue life of a mix can be increased by decreasing the air voids and increasing the asphalt content to a certain point without producing a mix that is prone to rutting (Roberts et al., 1996). Nichols Consulting Engineering, Chtd wrote in 1998 that, when considering long-term pavement performance, air voids can be used to predict pavement life. Air voids are used as opposed to relative density due to the former having less variability. The air void content along with the aggregate gradation and asphalt content have been linked to permanent deformation and fatigue cracking in asphalt pavements. The best suggestion is to have a field performance database that contains information about past QC/QA projects to aid in the calculation of pay factors.

Deacon et al. originally performed a study for Caltrans in 1997 to develop a pay factor system for newly constructed hot mix asphalt. In 2002 the work was further developed with the pavement distresses analyzed being permanent deformation and fatigue cracking. The study was initially based on work done previously with Caltrans' Heavy Vehicle Simulator (HVS) in 1997 by J.T. Harvey et al. for fatigue and the Federal Highway Administration's (FHWA) WesTrack for rutting (Deacon et al. 2002). Asphalt content, air voids, microstrain and pavement thickness were considered for a fatigue life prediction model. The models developed in the study were for pavement quality associated with expected life and a cost model, which, according to the authors, should provide a realistic expectation concerning a pavement's rehabilitation. The performance model provided an estimate of off- and on-target year equivalent single axle loads (ESALs). The cost model then uses the estimated ESALs and to calculate a ratio called the relative performance (RP).

The premise behind the study was that the quality of construction highly affects the performance and life of a pavement. To improve these operations, QC/QA procedures involving both payment penalties and bonuses were developed to motivate contractors (Deacon et al., 1997). It should be noted that the pay factor system in this paper was developed to reflect the owner or government agency's best interests. That is, a bonus within a maximum specified limit is given for a pavement with superior performance characteristics, which will in turn delay the future cost of any repairs to the pavement. The reverse is true for a pavement that results in less than desirable performance characteristics. In this case a pay penalty is issued in an amount equal to the cost to the agency for repairs that will be needed in the future. The cost models do not consider user costs, which can include: driver delay during construction and maintenance activities, tire wear, gasoline consumption or vehicle depreciation.

The first step was to develop a performance model for HMA with regard to fatigue life. This model was created based upon previous research done by SHRP in 1994 (Deacon et al.) and finally extended to the Caltrans HVS (Harvey et al., 1997). The fatigue model included asphalt content, air voids and pavement thickness. A regression analysis was performed resulting in equation 2.5.

$$N_f = \exp\left(-22.0012 - 0.164566 V_{air} + 0.575199 P_{Wasp} - 3.71763 \ln \varepsilon_t\right) \qquad \text{Eqn. 2.5}$$

where:

N_f is fatigue life,

V_{air} is air voids,

P_{Wasp} is asphalt content, and

ε_t is the tensile strain at the bottom of the HMA layer.

Next, Monte Carlo simulations were performed to evaluate the model and give predictions of the pavement life with regard to fatigue life. ELSYM5 (a linear elastic analysis computer program) was used to simulate pavement loading. After this, an estimate of the pavement loading in ESALs was produced using equation 2.6.

$$ESALs = \frac{N_f \times SF}{TCF} \qquad \text{Eqn. 2.6}$$

where:

N_f=fatigue life,

SF=shift factor used from Caltrans data and calibrated for this model, and

TCF=temperature conversion factor.

After the performance model was created, a cost model was developed. This model used a "relative performance" indicator. The RP ratio is defined as the off-target year ESALs over the on-target year ESALs. Basically, if the materials placed do not meet the agencies' expectations in terms of how long it should last and thus force the pavement to be replaced/rehabilitated sooner than expected, the RP is lower. An equation (equation 2.7) for the relative performance for fatigue was then developed. It was based upon the asphalt content (P_{Wasp}), air voids (V_{air}) and the pavement layer thickness (P_{tAC}).

$$Combined\ RP = RP_{P_{Wasp}} \cdot RP_{P_{Vair}} \cdot RP_{P_{tAC}} \qquad \text{Eqn. 2.7}$$

It should be noted that there were actually two RP indicators developed in this study, one for rutting and one for fatigue. When both distress modes are considered to determine a pay factor, the one indicating the smallest RP value is used. The reason for this is that lower values of RP indicate a shorter pavement life in terms of that mode of failure. This would increase an owner agency's cost of repair in the future. Tables were then developed with relative performance indicators with respect to air void content, asphalt content and pavement thickness.

A graph was then used with a range of combined relative performances versus possible pay factors. An example of such a graph from the study can be seen in Figure 2.17 (Deacon et al., 2002).

Stiffness is thought to be a major contributor to asphalt fatigue life. In a study by Harvey et al. in 1996 it was found that by decreasing the air void content and decreasing the asphalt content, the mixture stiffness increased in the stiffness models. However, by increasing the asphalt content and decreasing the air void content in laboratory samples, fatigue life increased. It was also found in the study that the higher the microstrain level was set for the beam fatigue apparatus, the shorter the fatigue life. It is interesting to note that the researchers found that increased initial stiffness did not always lead to a decreased fatigue life, especially when the air voids were decreased.

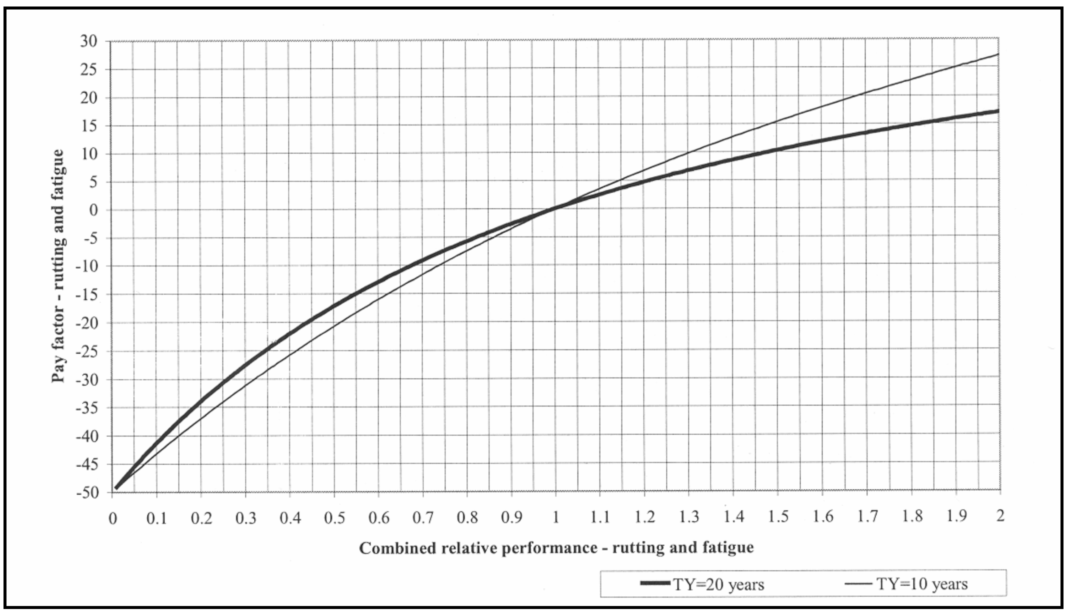


Figure 2.17: Pay Factors for Fatigue as a Function of Combined Relative Performance (Deacon, et al., 2002)

Recent developments at the University of Florida as reported in 2002 by Roque et al. analyzed fracture mechanics in HMA mixtures. The researchers developed a mechanical crack growth law that focuses on the concept of HMA being a viscoelastic material. The equipment and mix design system used were the indirect tension test and Superpave, respectively. The researchers contended that crack growth should not be simply modeled through the use of simple fatigue laws, such as those developed by Monismith in 1985. Most research prior to Roque's developments was based on the idea that many materials used in HMA mixtures initially had physical flaws in them before they were even used for a road building material. Roque contended that this assumption was not a viable one in that it could not properly access the mechanics of crack growth from its beginning. Another main concern that Roque had was the assumption that crack growth was a continuous process. In light of these previously held convictions, Roque hypothesized that cracks in HMA pavements are a function of two different types of responses to loading. The first response he called fully-healable micro-damage. This implies that the HMA material has undergone damage, but no cracking has begun. The second response he called un-healable macro-damage. According to Roque, this damage occurred when the beginning of a crack or actual crack growth had taken place in the HMA pavement. Therefore, he defined a threshold point concerning cracking damage in HMA pavements. This threshold was the point in time where the pavement began exhibiting macro-damage. Ultimately, Roque concluded that crack growth was not a continuous process; it was a stepwise process. His theory focused on zones of stress developed in the HMA material, the energy needed to cause creep failure, the m-value (the parameter that governs creep rate in HMA binders) and a healing rate parameter.

In a paper submitted for the 2004 Transportation Research Board (TRB) meeting, Hajj et al. discussed research that evaluated the performance of Superpave and Hveem mix designs using the flexural bending beam test. There were two types of binder used: polymer modified and neat. A polymer modified binder is a liquid binder that has polymer fibers blended into it. It was thought that the addition of these fibers to an asphalt binder may improve the performance characteristics of the mix with respect to fatigue and low-temperature cracking. A neat binder is simply a binder without any additive materials in it. The research showed that thermal aging (aging with intense heat) of modified binders did not have a negative impact on the fatigue life of the pavement, whereas some, but not all, of the neat binders tested generally did have a reduction in fatigue life. Given these differences, the researchers recommended to test mixes with respect to fatigue with binders that had been aged. Also, the researchers developed a fatigue life prediction model. They highly recommended that volumetric terms such as stiffness, air voids and asphalt content be used in the model, as opposed to using only the elastic modulus and the microstrain. The addition of the volumetric terms in the model led to significant model improvement.

CHAPTER 3 VERIFICATION OF MIX DESIGNS

3.1 Introduction

Verification of the hot mix asphalt mixture designs was undertaken for future use in performance testing of the laboratory designs as well as controlled variations in asphalt binder content and percent air voids. The Michigan Superpave mix design criteria were used for verification.

3.2 Verification of Aggregate Stockpile Gradations and Establishment of Job Mix Formula Gradations for Mix Design Verifications

Verification of the aggregate stockpiles were done in accordance with AASHTO T 27: Sieve Analysis of Fine and Coarse Aggregate and AASHTO T11: Materials Finer than 75μm (No. 200) Sieve in Mineral Aggregates. The MTU gradations for each stockpile and comparisons to the submitted JMF gradations are contained in Appendix A. The research team took considerable effort in producing a combined gradation that was as close as possible to the submitted JMF as slight changes in gradation typically is more critical than slight changes in particle shape in the demand for asphalt binder in the Superpave mix design system. Considerable changes in stockpile percentages used to replicate the JMF gradation would likely affect particle shape and absorption characteristics of the blend and thus result in significant changes in selection of the optimum asphalt binder content.

Four JMF gradations could not be achieved without changing at least one bin percentage or the percentage of at least one stockpile by 10% or more. The projects were:

M-35, Escanaba: ½” Dense (Pit #55-188) of 23% for the JMF vs. 14% for MTU and Screen Sand (Pit #55-183) of 55% for the JMF vs. 68% for MTU.

US-31, Elk Rapids:	Gallagher (Pit #45-41) of 25% for the JMF vs. 42.5% for MTU.
I-75, Flint:	Lime Sand (Pit #58-11) of 15% for the JMF vs. 6% for MTU.
M-28, Brimley:	½" Minus (Pit #17-41) of 40% for the JMF vs. 55% for MTU and 3/8" Sand (Pit #17-41) of 64% for the JMF vs. 36% for MTU.

3.3 Mix Design Verifications

The Superpave mix design process used to verify the submitted JMF's are summarized in Tables 3.1 through 3.3. In conducting the Superpave mix designs, the following AASHTO test specifications were used: Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor: AASHTO T 312, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens (Method A): AASHTO T 166-00, and Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures: AASHTO T 209-99.

The volumetric calculations of voids in the mineral aggregate (VMA) and effective binder contents (P_{be}) used the aggregate specific gravity design values submitted in the JMF to MDOT. The assumed aggregate specific gravity values also are then translated into the calculation of voids filled with asphalt (VFA) as this calculation is dependent upon VMA.

3.4 Mix Design Verifications

The mix designs are summarized in Tables 3.4 through 3.10. Figures 3.1 through 3.20 graphically summarize the mix designs. Using the MDOT mix design verification tolerance of +/-0.3 within the submitted JMF, 16 of the 20 mix designs were verified with respect to the optimum binder content. The four projects that were outside the binder content tolerance were:

M-52, St. Charles: 5.5% JMF vs. 6.0% MTU,
M-50, Brooklyn: 6.8% JMF vs. 7.2% MTU,

I-75, Grayling: 6.3% JMF vs. 5.8% MTU, and

M-43, Lansing: 6.0% JMF vs. 6.5% MTU.

One important note to mention is that the mix design oven temperatures for the M-43 project were adjusted to reflect that a polymer modified asphalt binder was used based on the rotational viscosity test results. This adjustment resulted in the compaction temperature being 162°C rather than the stated 157°C on the MDOT 1911 form for this mixture.

Several mix designs were attempted for the US-31 (Elk Rapids) project and in all cases the aggregate structure was relatively insensitive to changes in asphalt binder content. An optimum binder content for this aggregate structure could not be obtained as 4% air voids was not obtained even when 0.5% more binder was added than the JMF optimum. The mix design results for this project are shown in Figure 3.8.

Examining the voids in the mineral aggregate (VMA) criteria, there are four mixes that do not meet the minimum VMA criteria. The following projects fail to meet the minimum mix design VMA criteria:

8-Mile Road, Warren: 15.3% JMF (14.0% Design Minimum) vs. 13.9% MTU,

I-94 (4E30), Ann Arbor: 14.7% JMF (14.0% Design Minimum) vs. 13.7% MTU, and

I-94 (3E30), Ann Arbor: 13.7% JMF (13.0% Design Minimum) vs. 12.9% MTU.

The 0.2% drop in the MTU binder content from the JMF for the 3E30 Ann Arbor mix did place the VMA criteria out of specification.

The voids filled with asphalt (VFA) criteria have a range of 65-78% and 65-75% for E3 mixes and E10 and above mixes, respectively. Only the gap-graded Superpave mix on I-94 in Ann Arbor (78.1%) and the 5E30 mix on I-75 in Flint (78.2%) were out of specification. It is important to note that the gap-graded Superpave mix design was accepted with a VFA of 76.8%.

The remaining mixes were all within the design specification ranges.

3.5 Chapter Summary

An extensive amount of time was expended in the verification of the project mix designs. Of the 20 projects, the research team was unable to obtain a mixture design from only the US-31, Elk Rapids project. This mixture's aggregate characteristics (gradation) coupled with its insensitivity to changes in binder content indicates that a target air void content of 4% can not be achieved with a reasonable amount of asphalt binder. Ensuing performance tests of this mix design coupled with field performance will determine the quality of this mix.

Four other mixes had optimum asphalt binder contents outside of the +/-0.3% tolerance for MDOT's mix design verification, but in all instances were within +/-0.5%. Three mixes had VMA values below their respective MDOT minimum design thresholds. However, two of the three mixes did have JMF values relatively close to the minimum, indicating the mix designers may be designing too close to the design criteria considering VMA typically drops during field production. Only one mix was outside of the MDOT mix design VFA criteria.

Table 3.1 Superpave Mix Design Criteria

Design Parameter	Mixture Number			
	5	4	3	2
Percent of Maximum Specific Gravity (%G_{mm}) at the design number of gyrations	96.0%			
(%G_{mm}) at the initial number of gyrations, (N_i)	See Table 3.2			
(%G_{mm}) at the maximum number of gyrations, (N_m)	≤ 98.0%			
VMA min % at N_d [based on aggregate bulk specific gravity, G_{sb}]	15.0	4.0	3.0	2.0
VFA at N_d	See Table 3.3			
Fines to effective asphalt binder ratio ($P_{0.075}/P_{be}$)	0.6 - 1.4			
Tensile Strength Ratio (TSR)	80.0% Minimum			

Table 3.2 VFA Minimum and Maximum

Estimated Traffic (million ESAL)	Mix Type	Top Coarse	Leveling and Base Courses
< 0.3	E03	70-80	70-80
< 1.0	E1	65-78	65-78
< 3.0	E3	65-78	65-78
< 10	E10	65-78	65-75(1)
< 30	E30	65-78	65-75(1)
< 100	E100	65-78	65-75(1)
(1) For Mixture Number 5, the specified VFA range shall be 73% - 76%.			

Table 3.3 Superpave Gyratory Compactor (SGC) Compaction Criteria

Estimated Traffic (million ESAL)	Mix Type	%G_{mm} at (N_i)	Number of Gyrations		
			N_i	N_d	N_m
< 0.3	E03	≤ 91.5%	7	68	104
< 1.0	E1	≤ 90.5%	7	76	117
< 3.0	E3	≤ 90.5%	7	86	134
< 10	E10	≤ 89%	8	96	152
< 30	E30	≤ 89%	8	109	174
< 100	E100	≤ 89%	9	126	204
Note: Compact all mixture specimens fabricated in the SGC to N_m. Use height data provided by the SGC to calculate volumetric properties at N_i and N_d.					

Table 3.4 MTU Trial Gradations 3E3

	M-32, Lachine/ Alpena	**M-45 Grand Rapids**	**Old M-14 Plymouth**
Sieve Size, mm	Gradation, Percent Passing		
25.0	100.0	100.0	100.0
19.0	100.0	99.4	99.6
12.5	92.5	86.3	87.4
9.5	82.0	85.1	75.0
4.75	45.7	75.6	36.4
2.36	31.5	51.7	24.8
1.18	20.5	37.1	18.4
0.600	13.4	29.2	14.2
0.300	8.3	17.7	9.7
0.150	5.2	6.8	5.5
0.075	4.0	4.0	3.9
Mixture Characteristics			
Binder Content, %	5.4	5.1	5.1
%Gmm at N_{design}	96.0	96.0	96.0
VMA, %	14	13.2	13.6
VFA, %	71.4	69.6	70.7
F/P_{be}	0.88	0.98	0.98

Table 3.5 MTU Trial Gradations 4E3

	M-35 Escanaba	**M-52 St. Charles**	**M-90 Lexington**
Sieve Size, mm	Gradation, Percent Passing		
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	96.3	93.9	92.0
9.5	90.2	91.5	83.3
4.75	60.0	72.8	75.2
2.36	53.5	55.2	55.2
1.18	46.0	40.1	39.5
0.600	35.5	28.7	28.2
0.300	18.8	17.0	17.7
0.150	8.3	8.2	9.3
0.075	5.5	5.2	5.6
Mixture Characteristics			
Binder Content, %	5.9	6.0	6.2
%Gmm at N_{design}	96.0	96.0	96.0
VMA, %	14.9	16.4	15.9
VFA, %	73.1	75.7	74.8
F/P_{be}	1.12	1.05	1.04

Table 3.6 MTU Trial Gradations 5E3

	M-50 Brooklyn	**US-31 Elk Rapids**	**US-24 Monroe**
Sieve Size, mm	Gradation, Percent Passing		
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	99.6
9.5	99.5	98.4	94.2
4.75	83.4	66.1	61.1
2.36	62.0	34.7	34.5
1.18	43.2	20.3	20.6
0.600	31.2	15.8	12.9
0.300	18.8	9.6	8.0
0.150	10.1	7.1	5.7
0.075	5.9	4.1	5.0
Mixture Characteristics			
Binder Content, %	7.2		6.2
%Gmm at N_{design}	96.0		96.0
VMA, %	17.2		15.2
VFA, %	76.8		73.7
F/P_{be}	1.01		1.24

Table 3.7 MTU Trial Gradations 5E10

	I-75 Indian River	**I-75 Grayling**	**M-43 Saginaw Street**
Sieve Size, mm	Gradation, Percent Passing		
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	100.0
9.5	95.8	97.4	99.9
4.75	63.8	66.6	79.3
2.36	37.3	45.5	61.9
1.18	22.6	29.6	46.0
0.600	14.0	20.1	33.0
0.300	8.3	11.3	19.2
0.150	5.0	6.5	8.8
0.075	3.5	5.1	5.2
Mixture Characteristics			
Binder Content, %	6.3	5.8	6.5
%Gmm at N_{design}	96.0	96.0	96.0
VMA, %	15.2	14.1	16.2
VFA, %	73.8	72.7	75.4
F/P_{be}	0.69	1.13	0.94

Table 3.8 MTU Trial Gradations 4E30

	8-Mile Road Warren	**I-94 Ann Arbor**	**I-94 Ann Arbor SMA**
Sieve Size, mm	Gradation, Percent Passing		
25.0	100.0	100.0	100.0
19.0	100.0	100.0	99.0
12.5	99.1	99.3	95.9
9.5	87.9	87.2	75.5
4.75	56.9	47.1	33.2
2.36	29.3	29.3	16.0
1.18	18.6	20.3	12.5
0.600	13.7	14.8	11.3
0.300	10.1	10.3	10.5
0.150	6.8	6.9	9.6
0.075	4.7	4.8	7.9
Mixture Characteristics			
Binder Content, %	5.1	5.0	6.9
%Gmm at N_{design}	96.0	96.0	96.0
VMA, %	13.9	13.7	17.2
VFA, %	71.2	70.8	76.8
F/P_{be}	1.08	1.15	1.28

Table 3.9 MTU Trial Gradations 5E30

	I-75 Auburn Hills	**I-75 Flint**	**I-75 Saginaw**
Sieve Size, mm	Gradation, Percent Passing		
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	99.4	98.8
9.5	98.5	96.4	95.3
4.75	68.5	70.6	86.9
2.36	37.2	43.4	57.8
1.18	22.1	27.0	40.4
0.600	14.8	17.3	30.5
0.300	10.0	10.5	18.6
0.150	6.9	6.7	8.5
0.075	5.8	5.3	5.9
Mixture Characteristics			
Binder Content, %	6.3	5.8	5.5
%Gmm at N_{design}	96.0	96.0	96.0
VMA, %	15.8	18.4	14.7
VFA, %	75.0	78.2	72.7
F/P_{be}	1.10	1.11	1.24

Table 3.10 MTU Trial Gradations 5E1 and 3E30

	M-28 Brimley	**I-94 Ann Arbor**
Sieve Size, mm	Gradation, Percent Passing	
25.0	100.0	100.0
19.0	100.0	99.6
12.5	99.9	87.7
9.5	95.3	76.2
4.75	69.3	40.1
2.36	53.0	23.4
1.18	41.4	15.7
0.600	30.3	11.8
0.300	15.6	8.5
0.150	8.3	5.9
0.075	5.2	4.7
Mixture Characteristics		
Binder Content, %	6.4	4.8
%Gmm at N_{design}	96.0	96.0
VMA, %	16.0	12.9
VFA, %	75.1	69.1
F/P_{be}	0.92	1.21

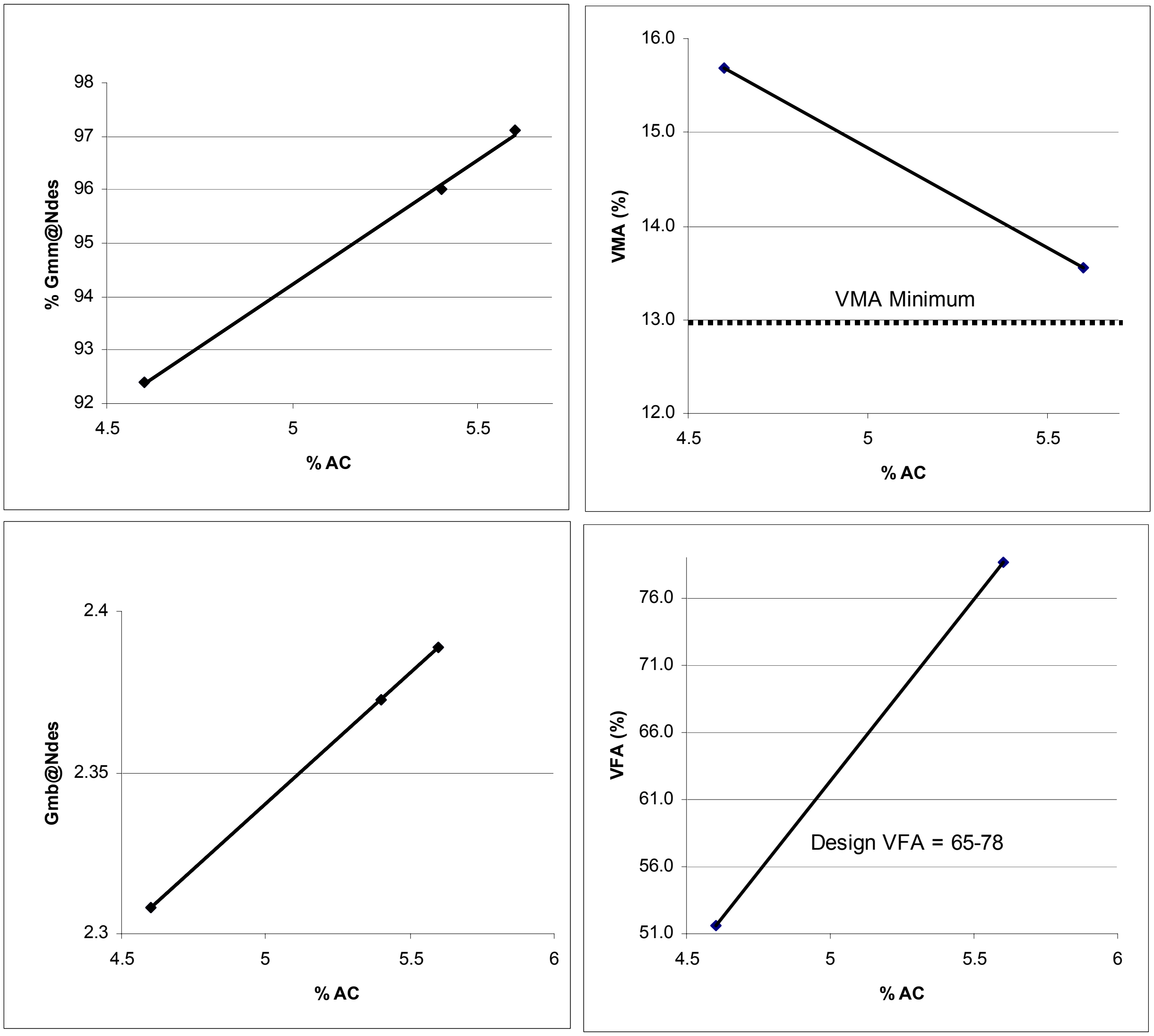


Figure 3.1 Mix Design Verification for M-32, Lachine/Alpena

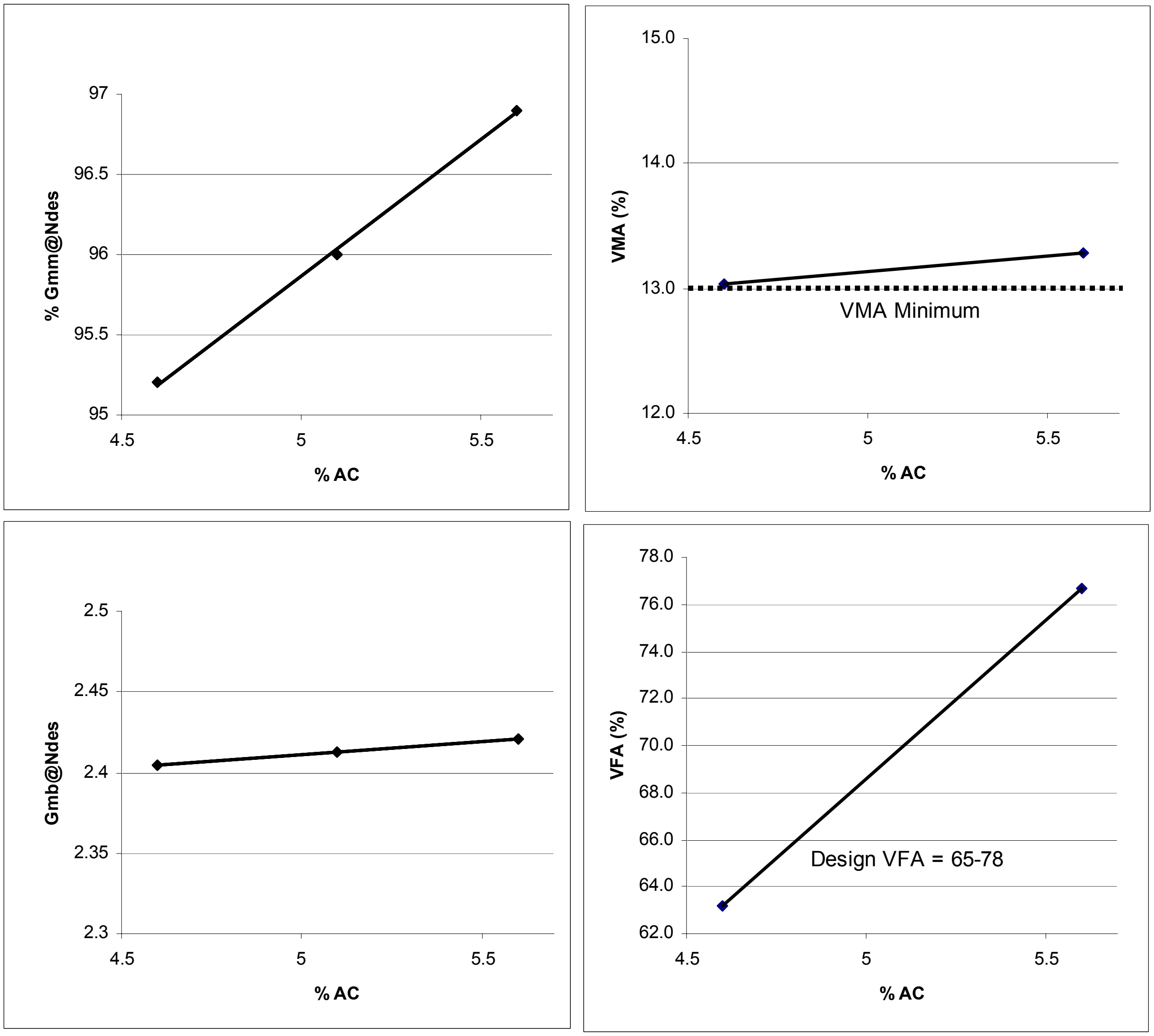


Figure 3.2 Mix Design Verification for M-45, Grand Rapids

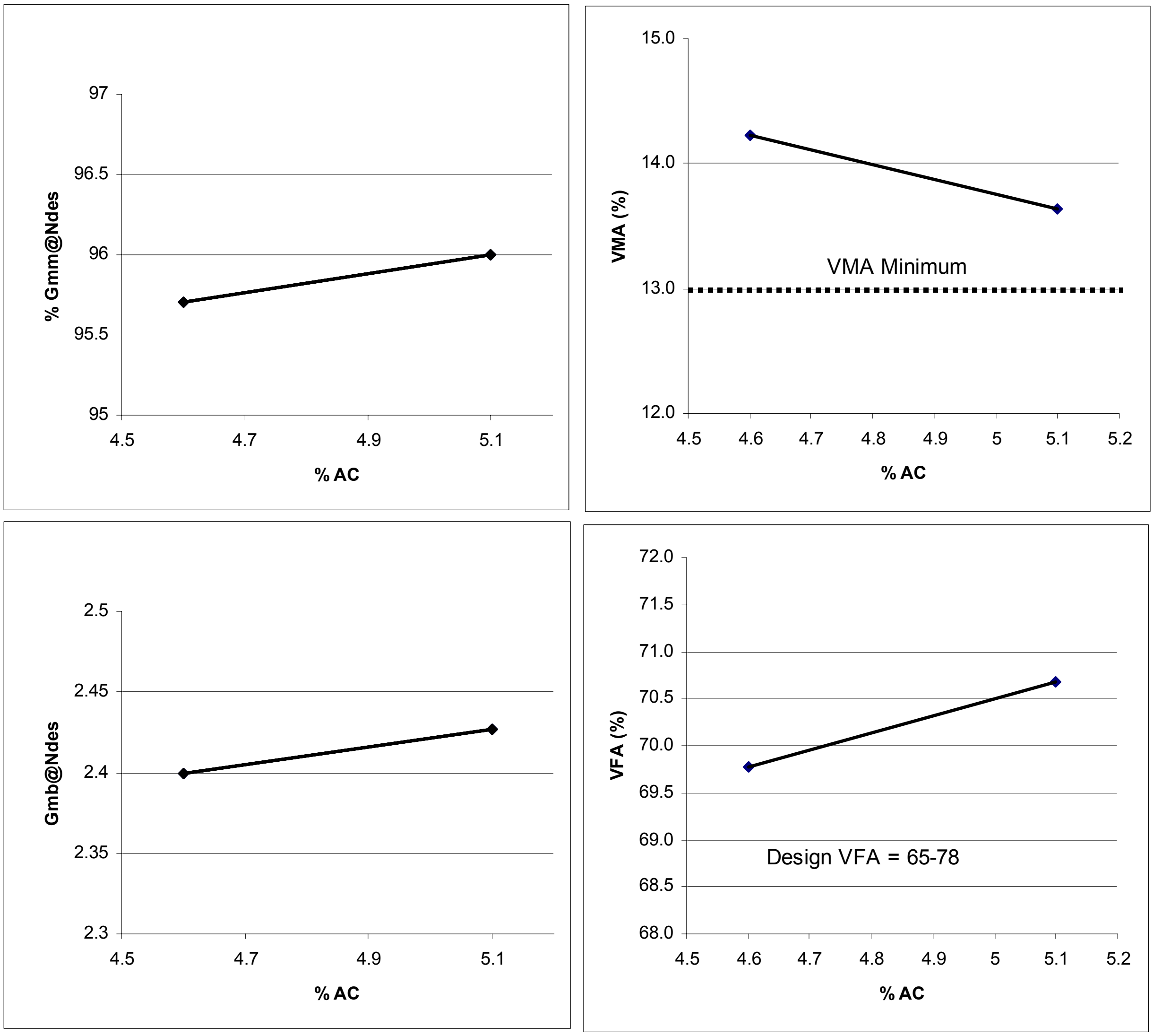


Figure 3.3 Mix Design Verification for Old M-14, Plymouth

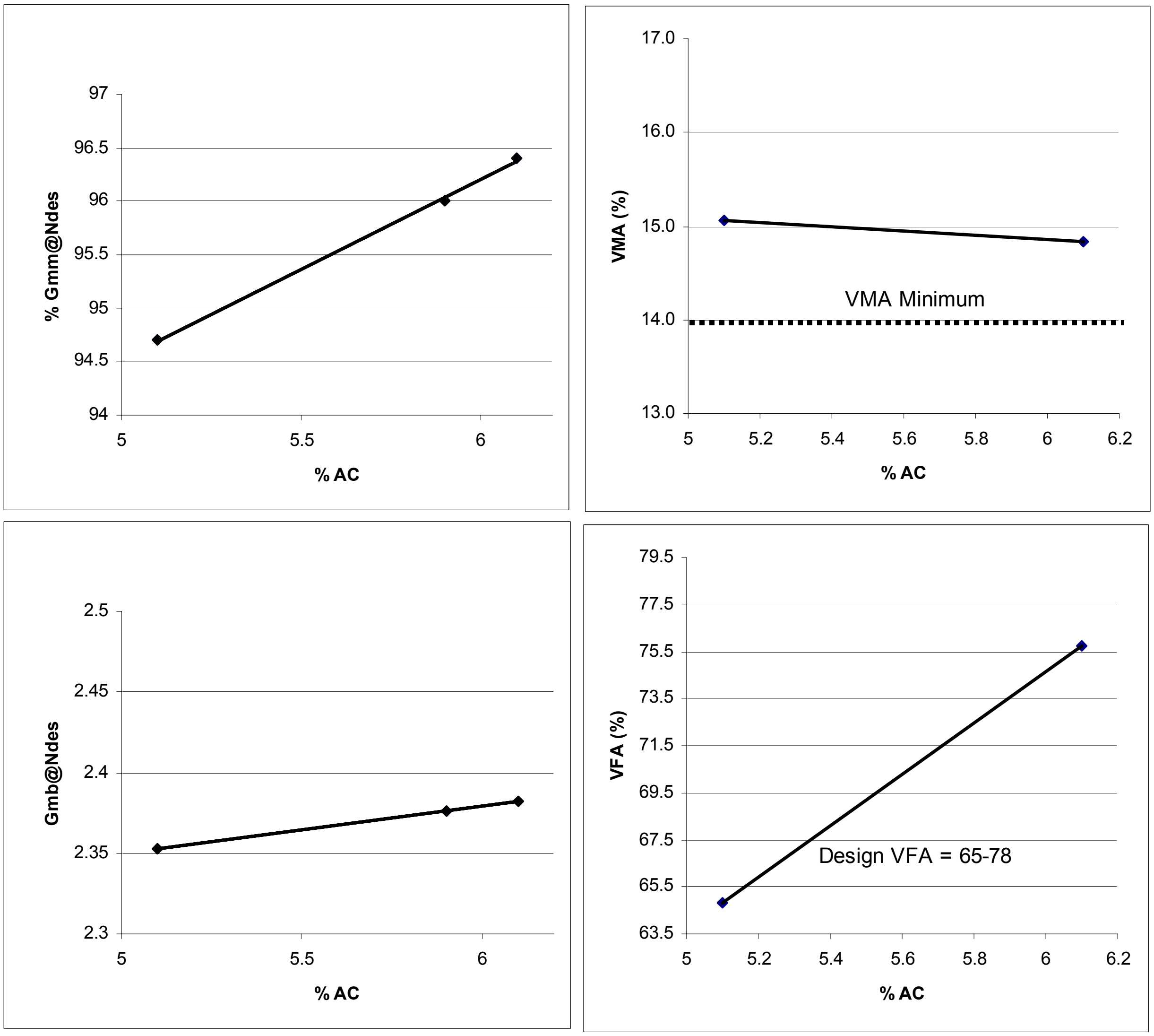


Figure 3.4 Mix Design Verification for M-35, Escanaba

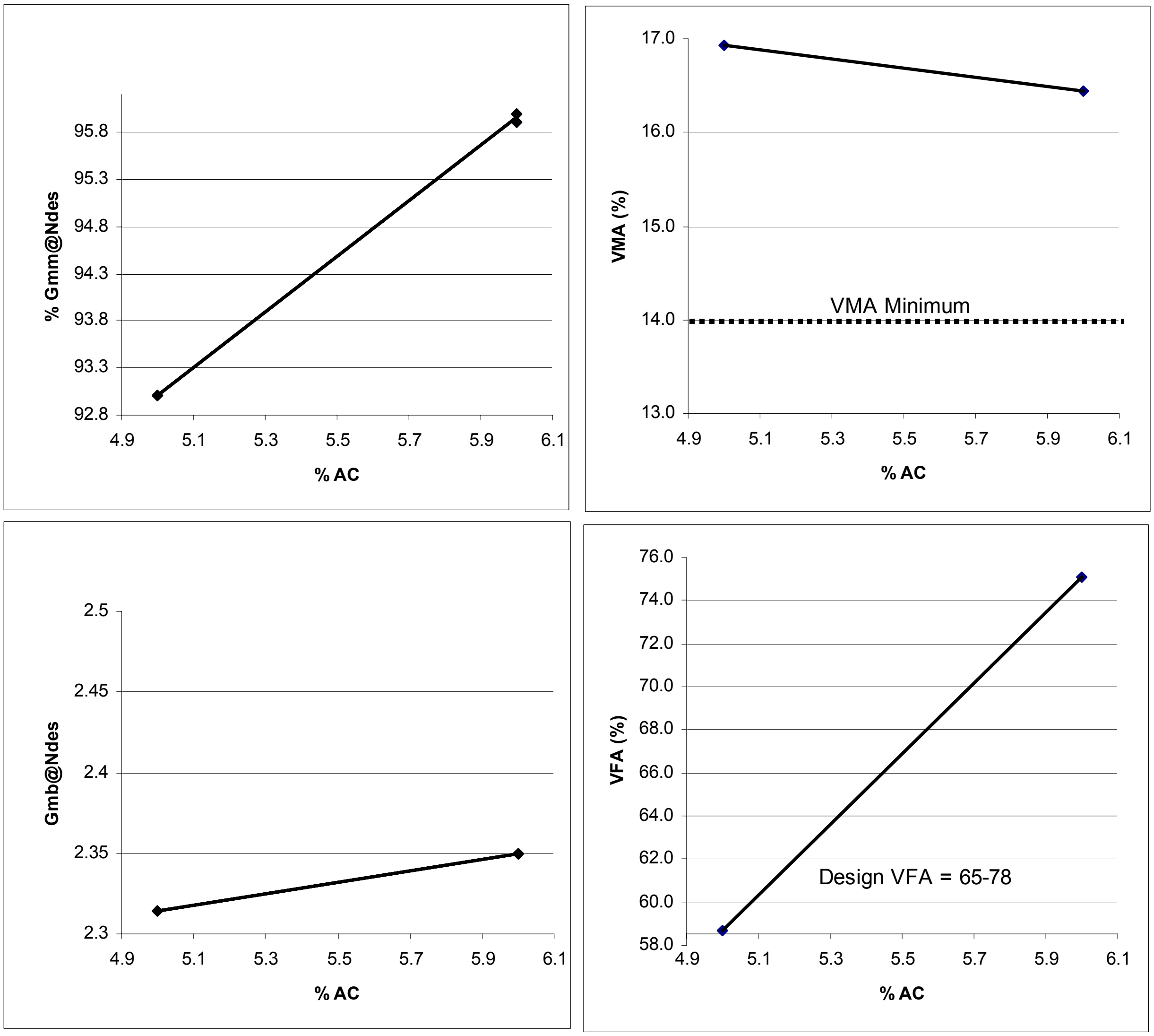


Figure 3.5 Mix Design Verification for M-52, St. Charles

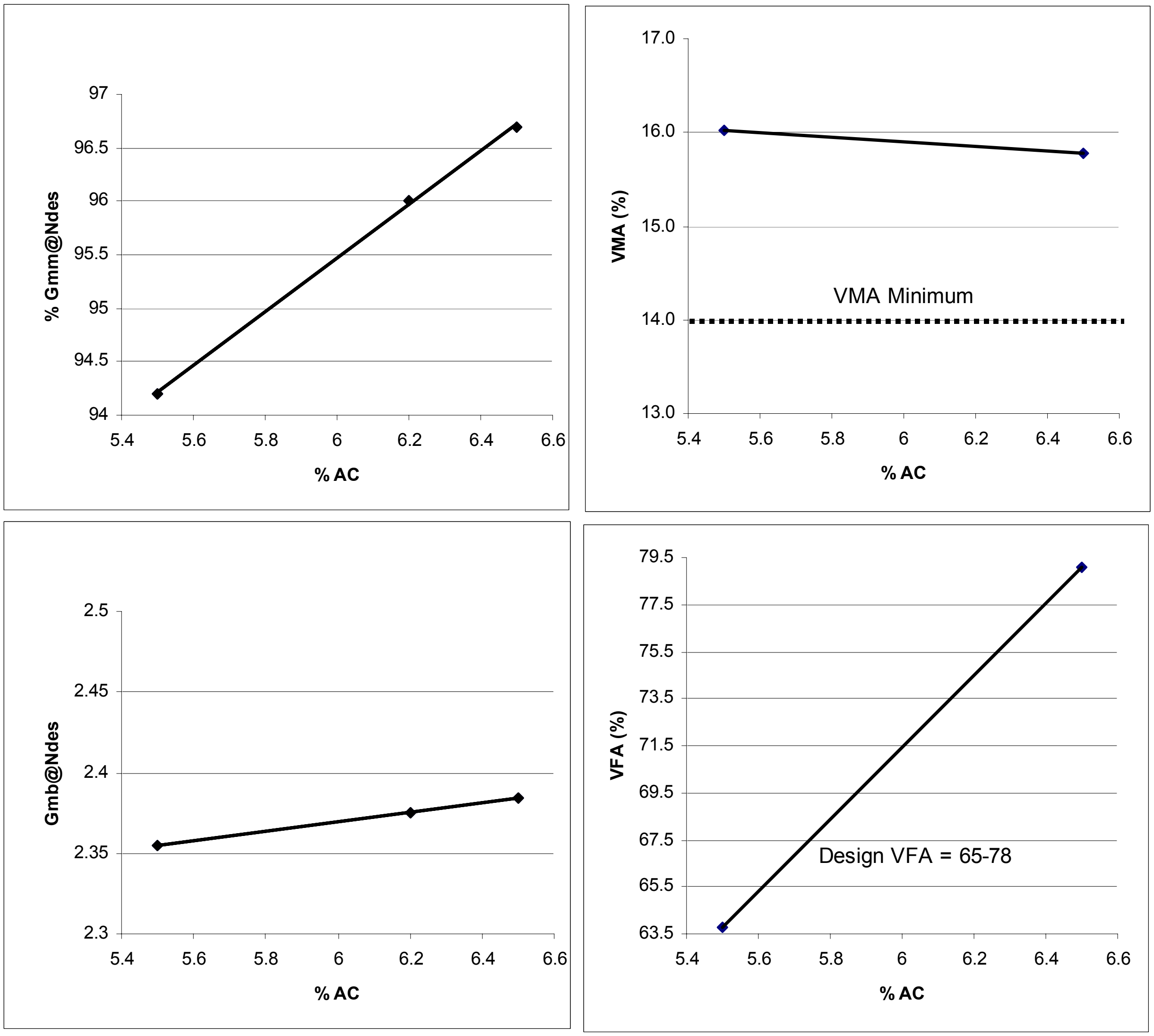


Figure 3.6 Mix Design Verification for M-90, Lexington

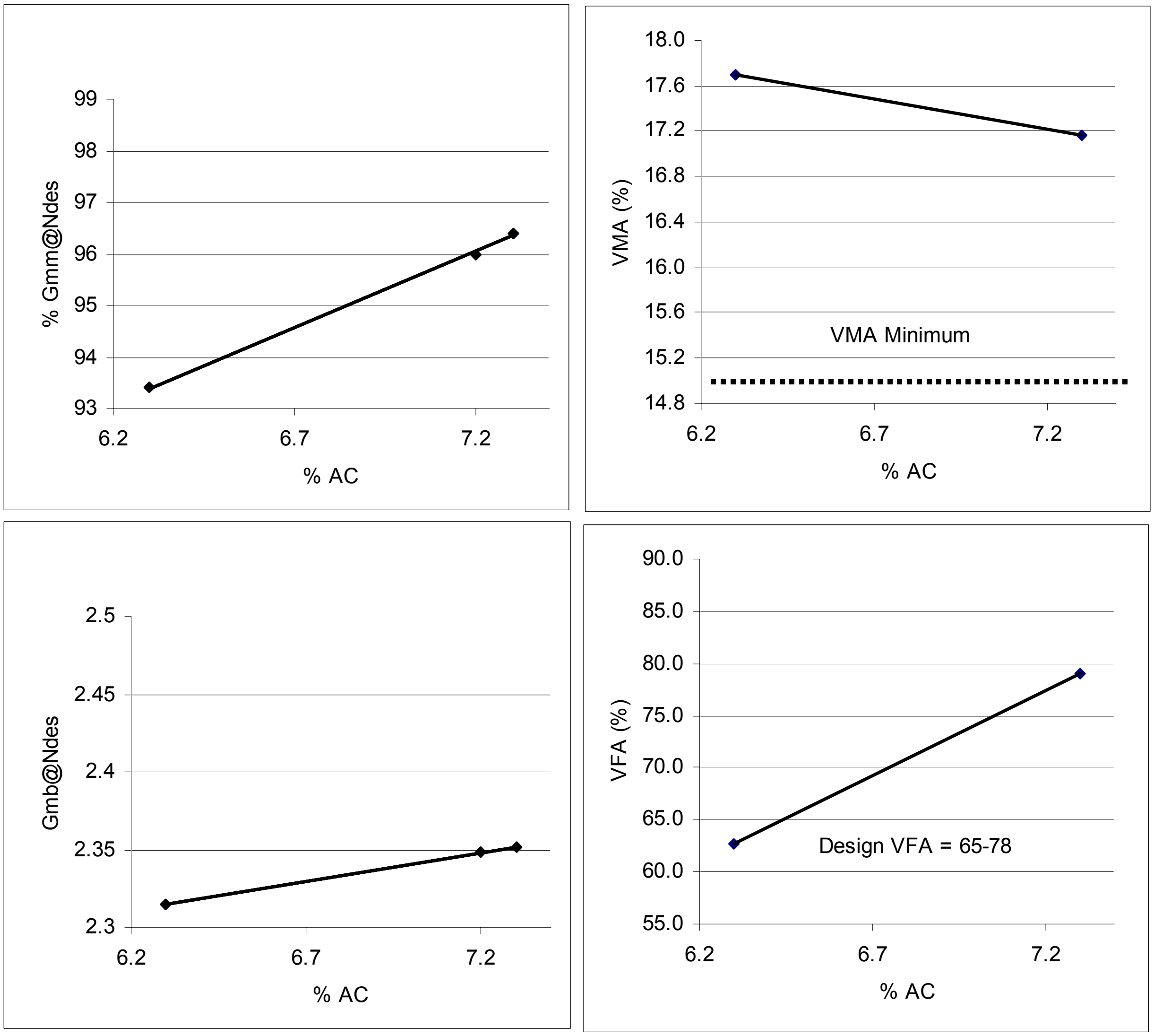


Figure 3.7 Mix Design Verification for M-50, Brooklyn

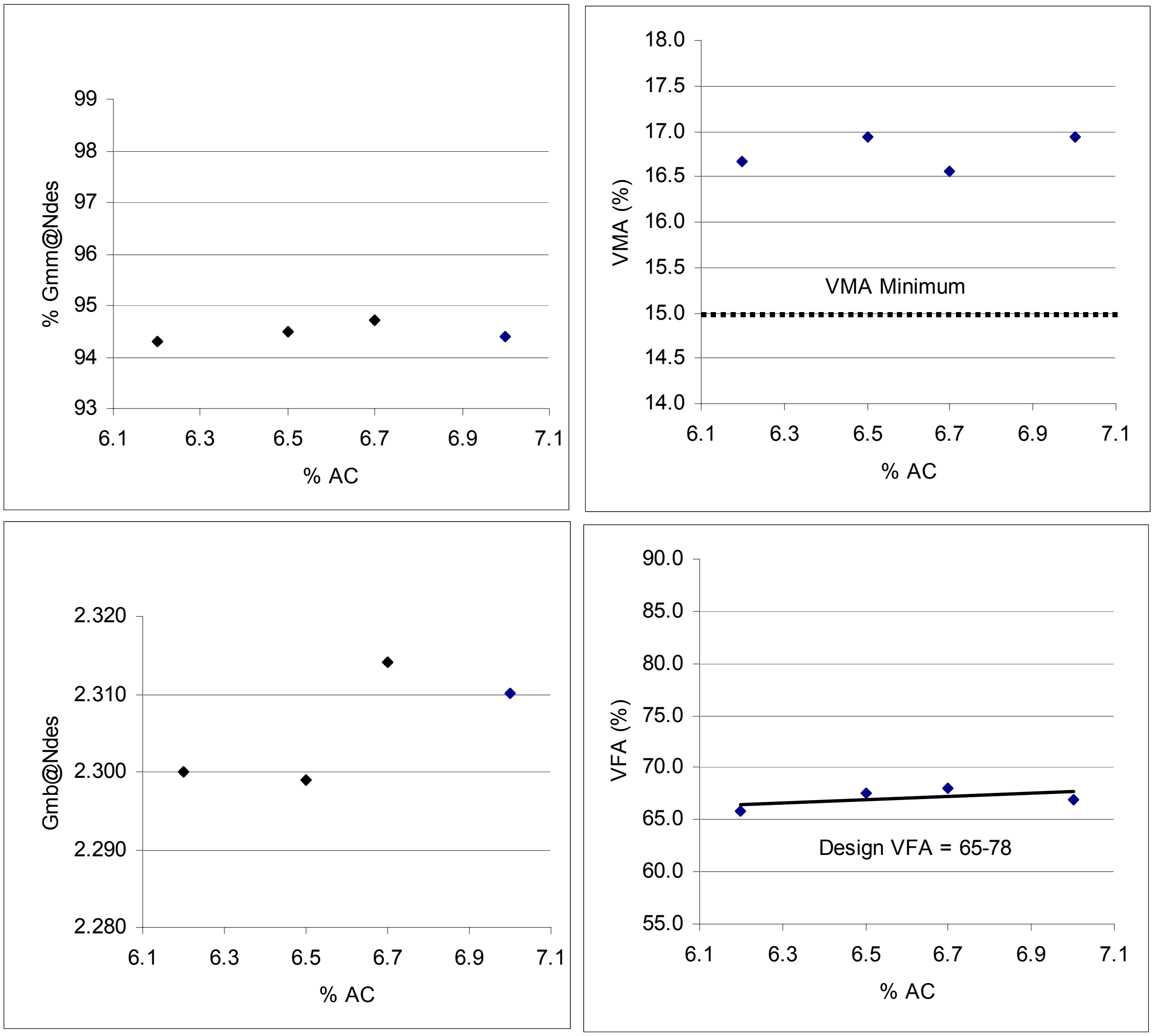


Figure 3.8 Mix Design Verification for US-31, Elk Rapids

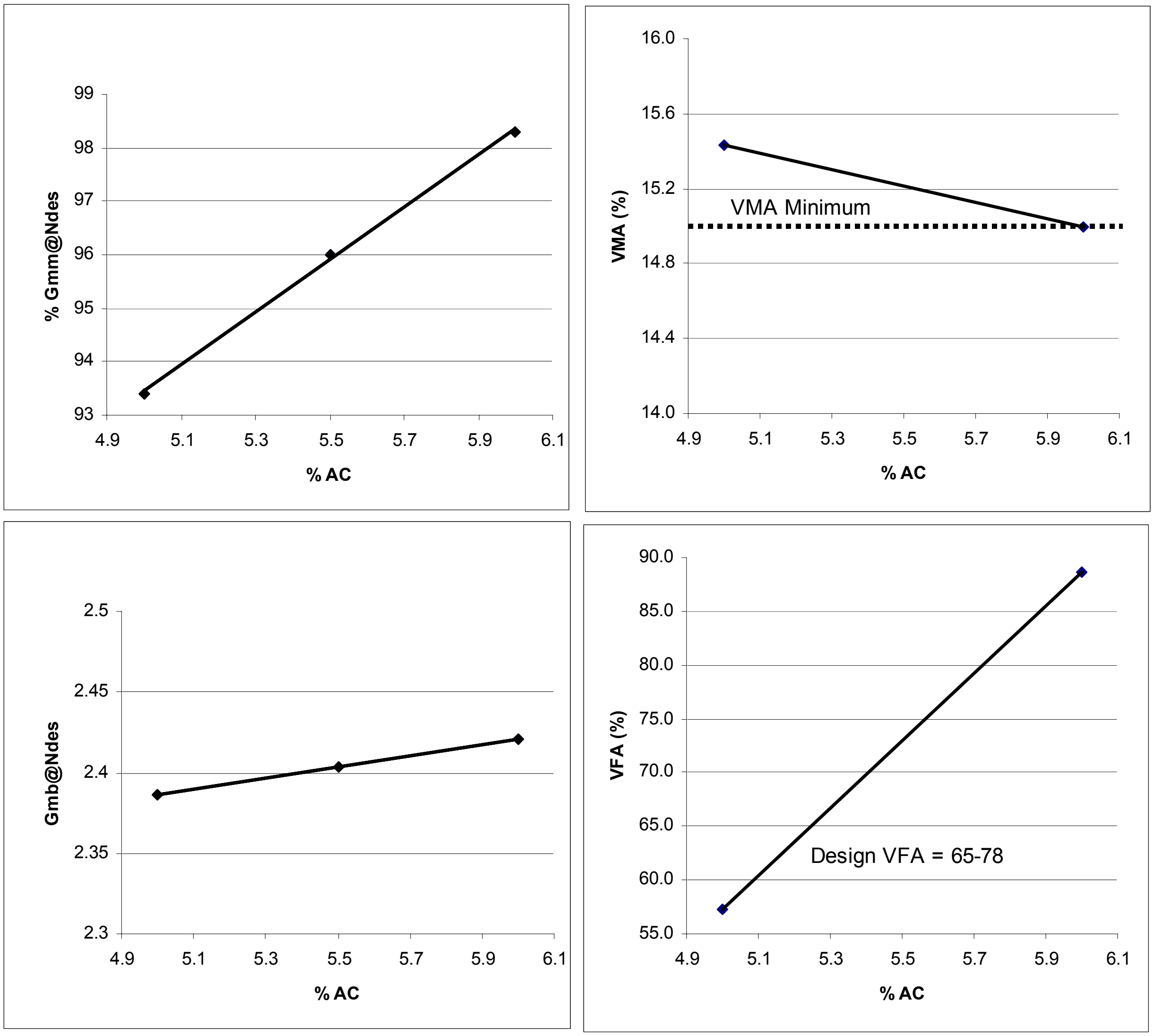


Figure 3.9 Mix Design Verification for US-24, Monroe

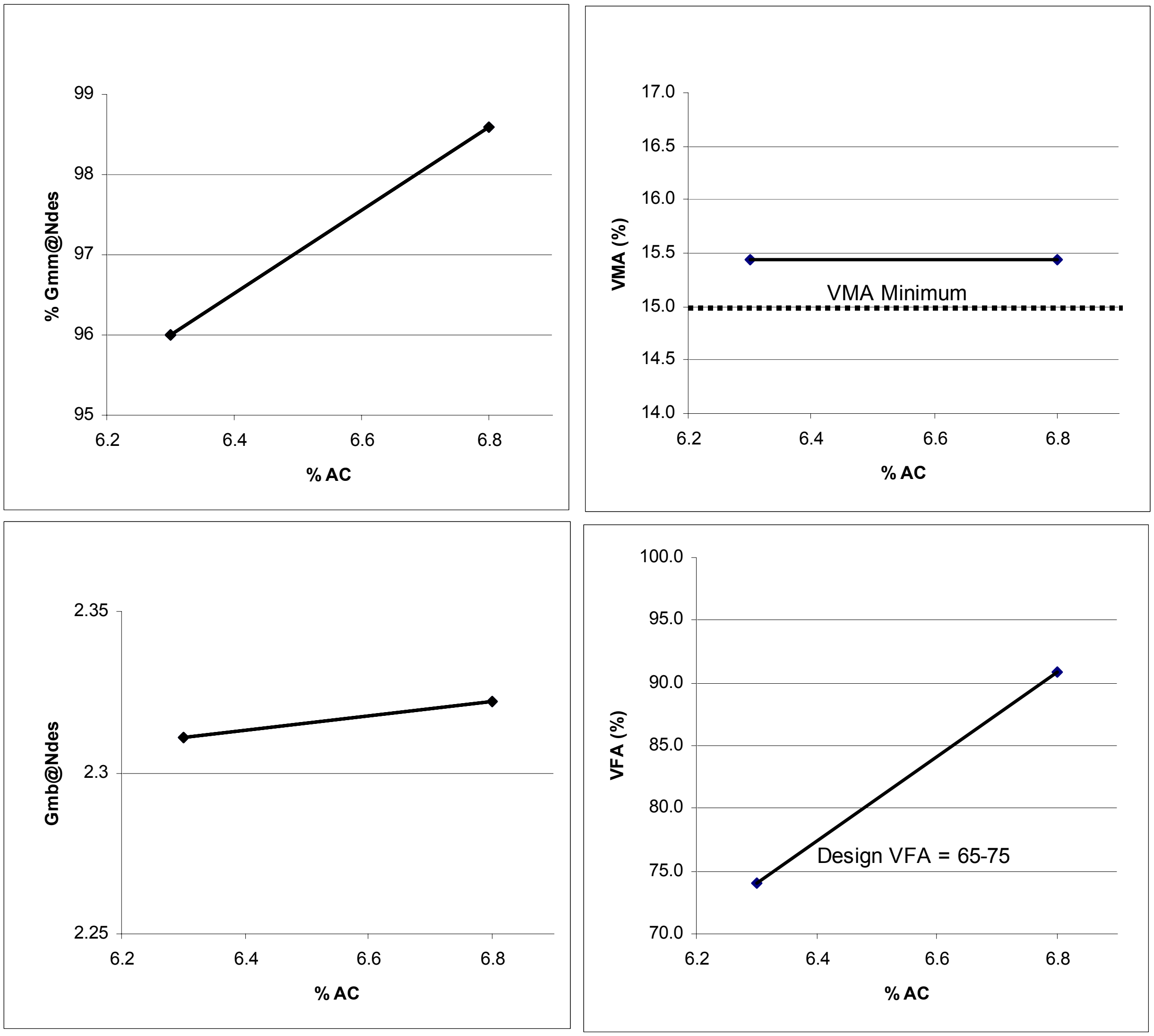


Figure 3.10 Mix Design Verification for I-75, Indian River

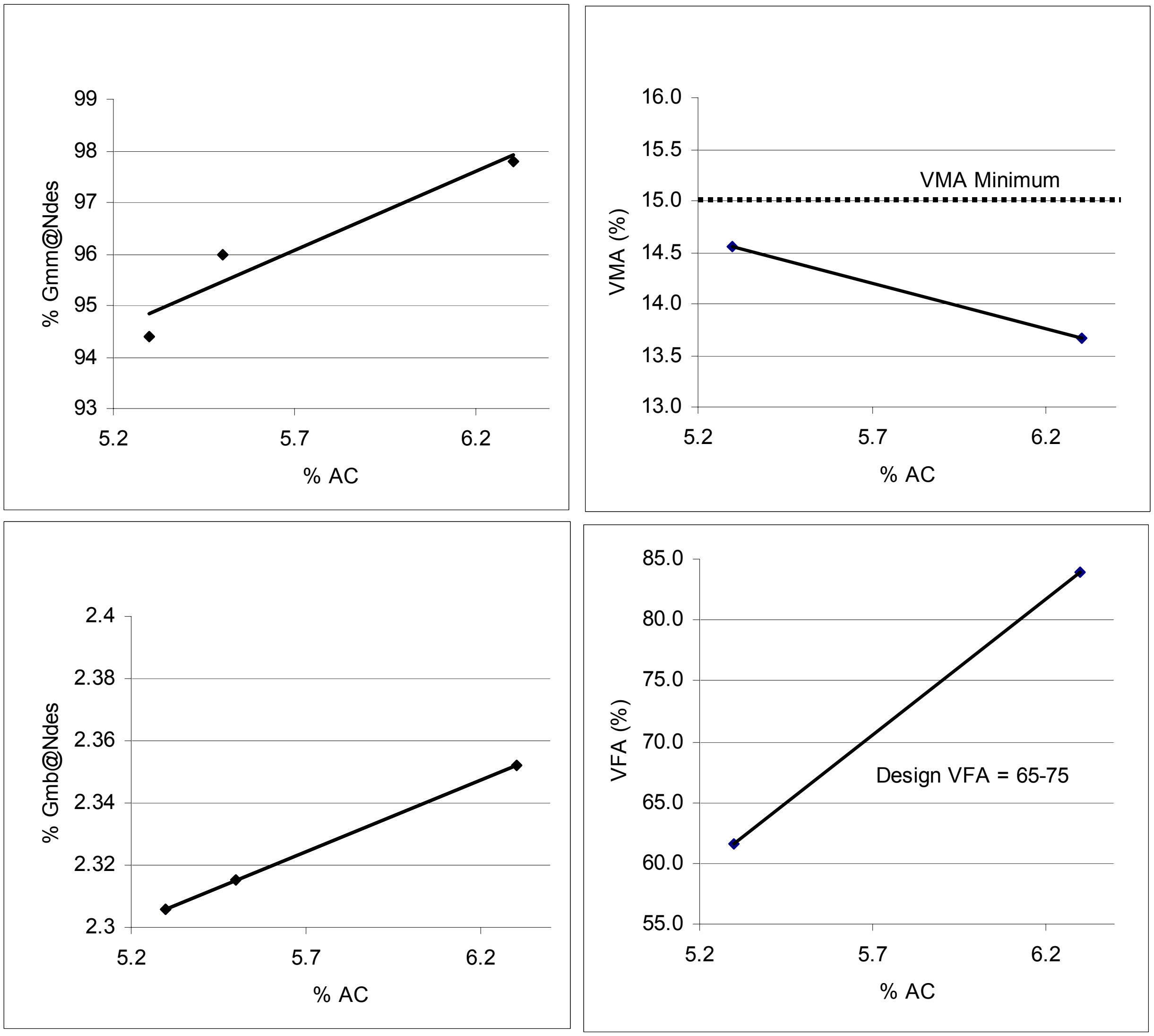


Figure 3.11 Mix Design Verification for I-75, Grayling

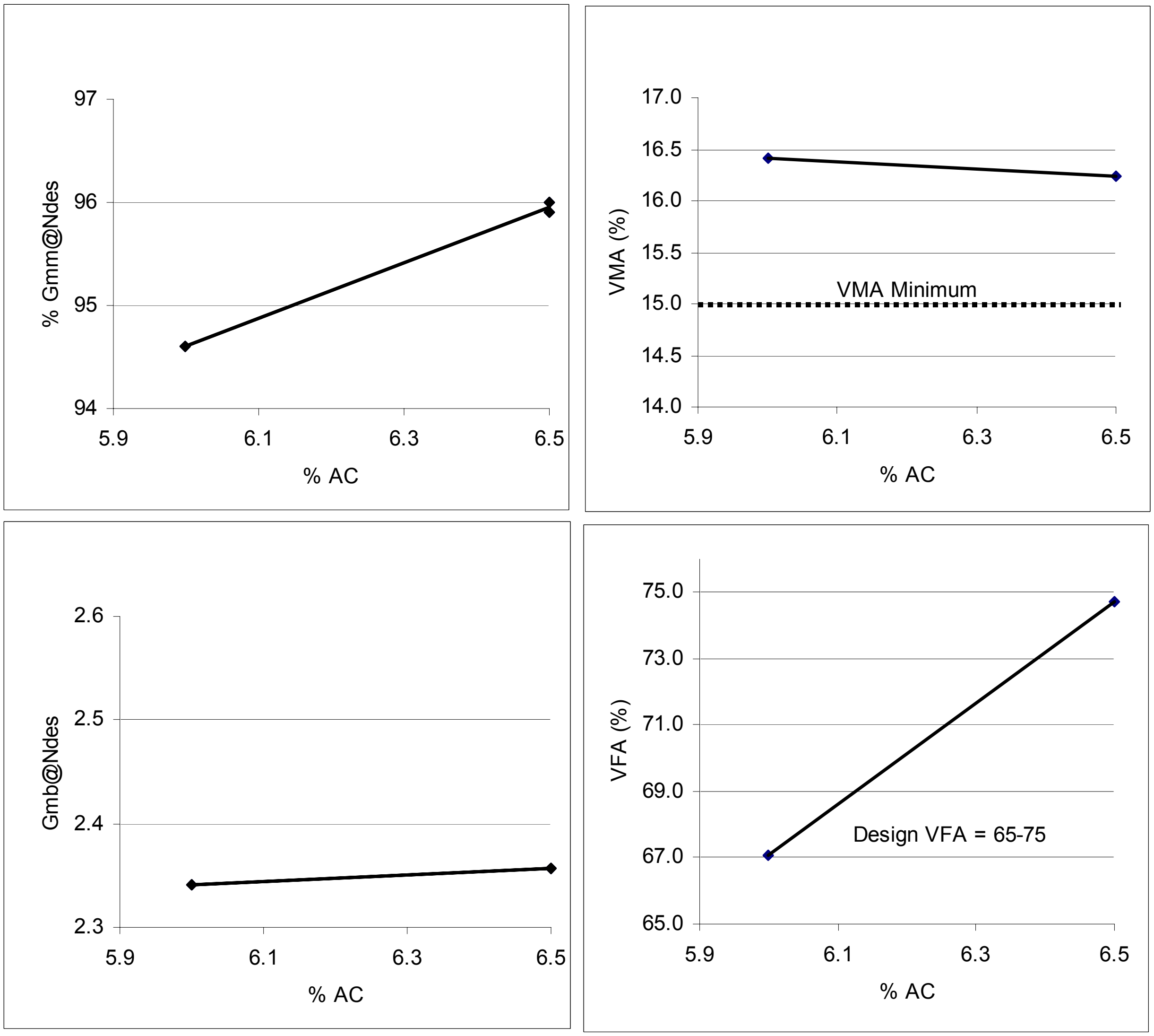


Figure 3.12 Mix Design Verification for M-43, Lansing

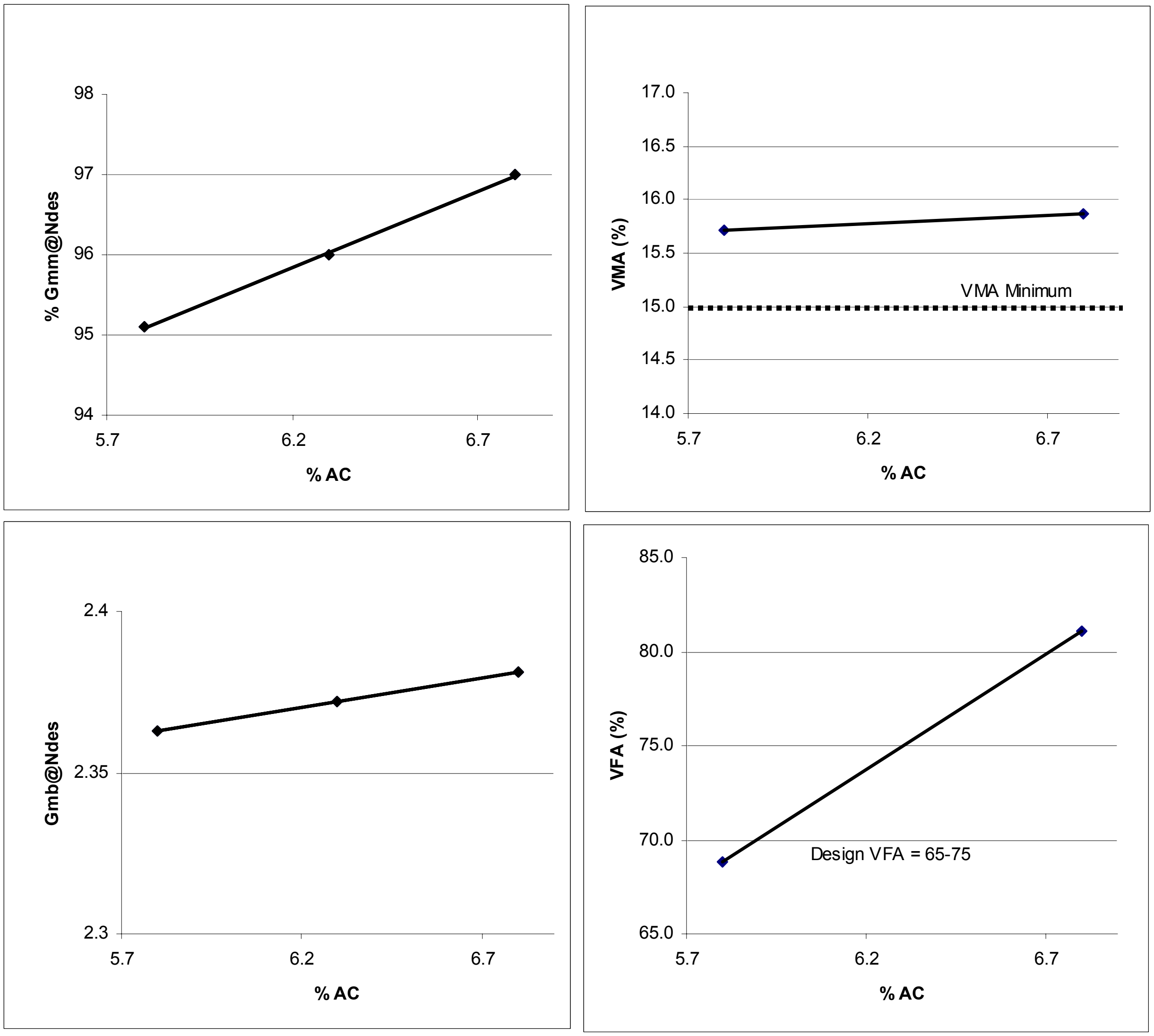


Figure 3.13 Mix Design Verification for I-75, Auburn Hills

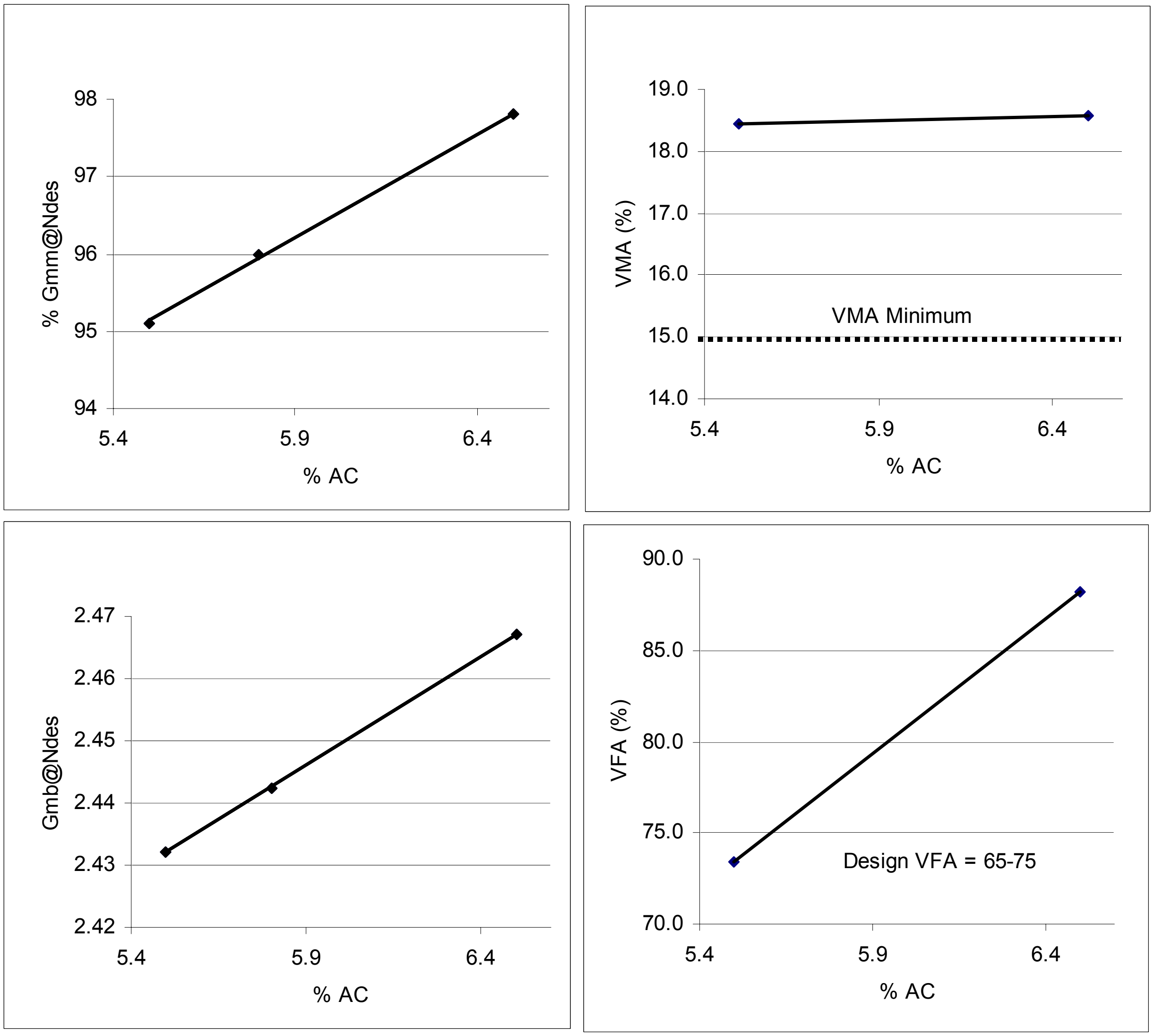


Figure 3.14 Mix Design Verification for I-75, Flint

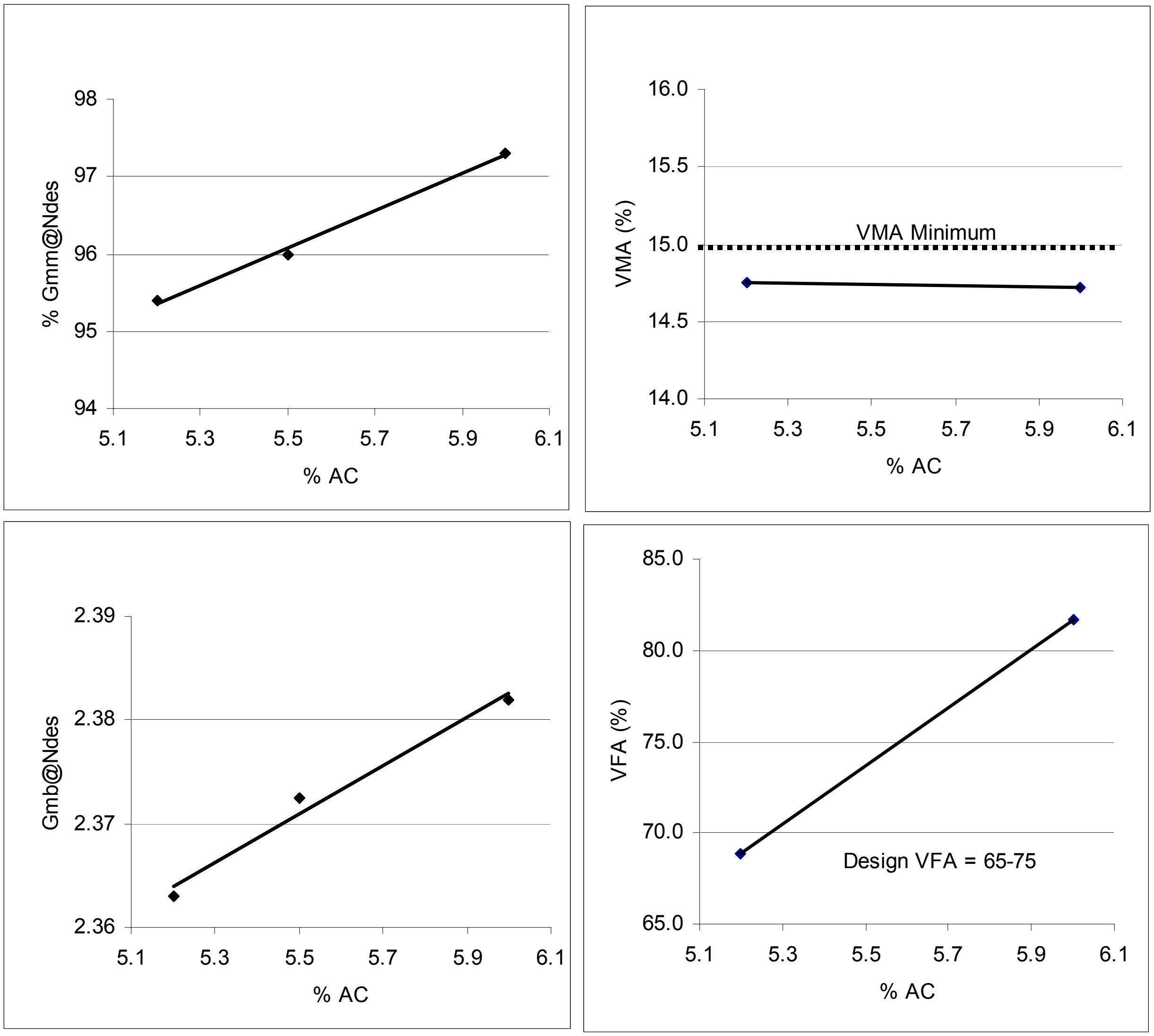


Figure 3.15 Mix Design Verification for I-75, Saginaw

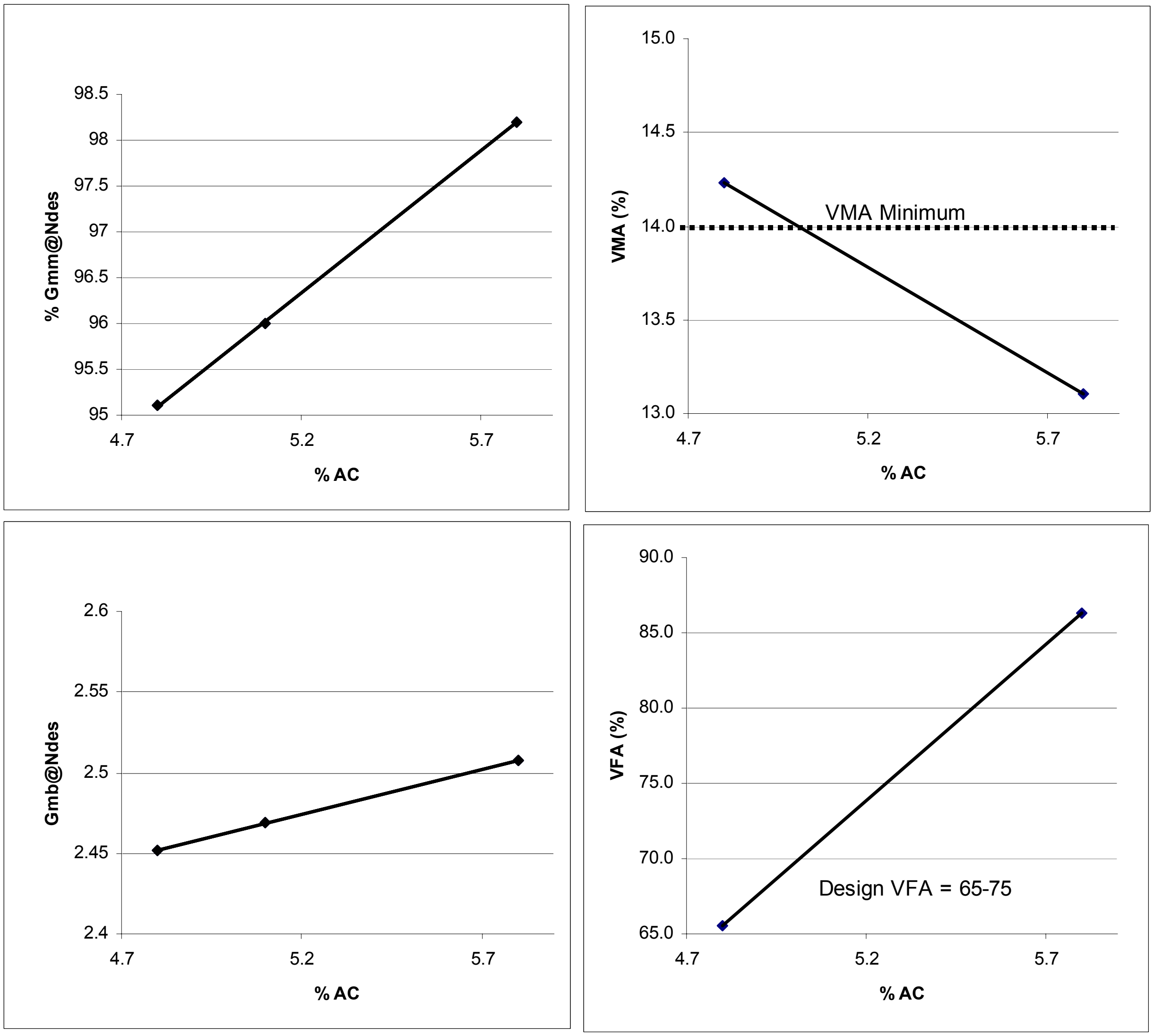


Figure 3.16 Mix Design Verification for 8-Mile Road, Warren

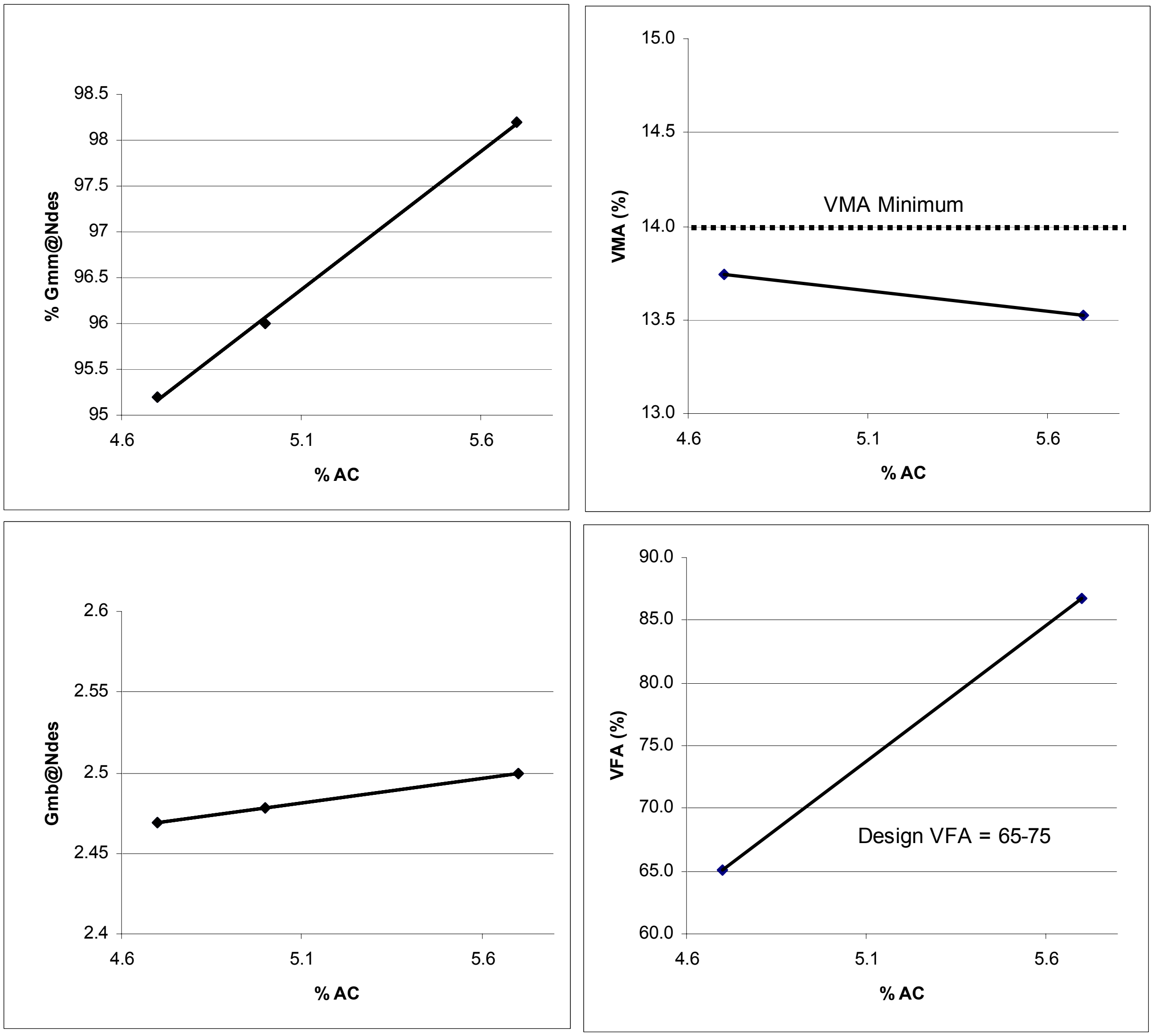


Figure 3.17 Mix Design Verification for I-94, Ann Arbor (4E30)

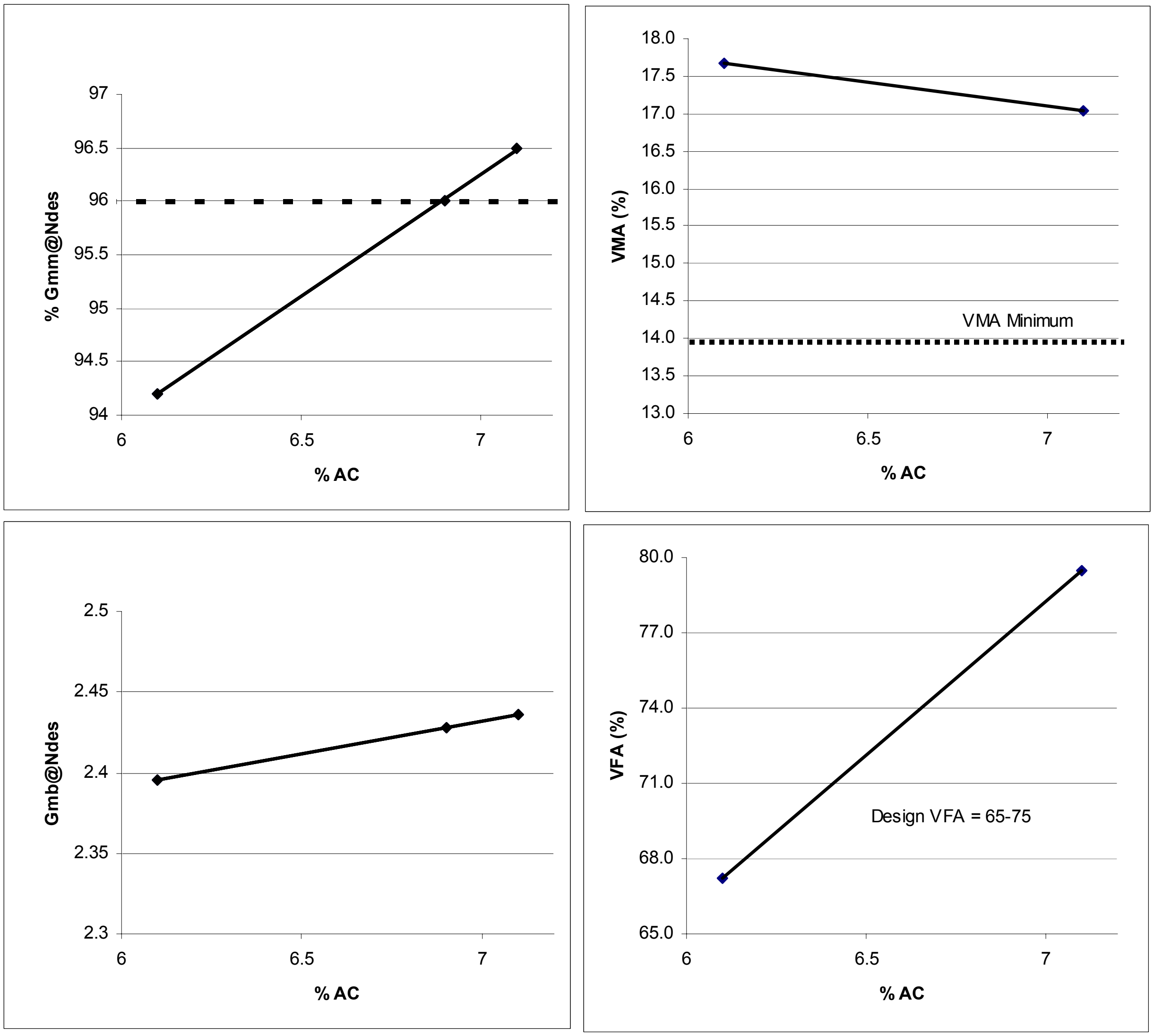


Figure 3.18 Mix Design Verification for I-94, Ann Arbor (SMA)

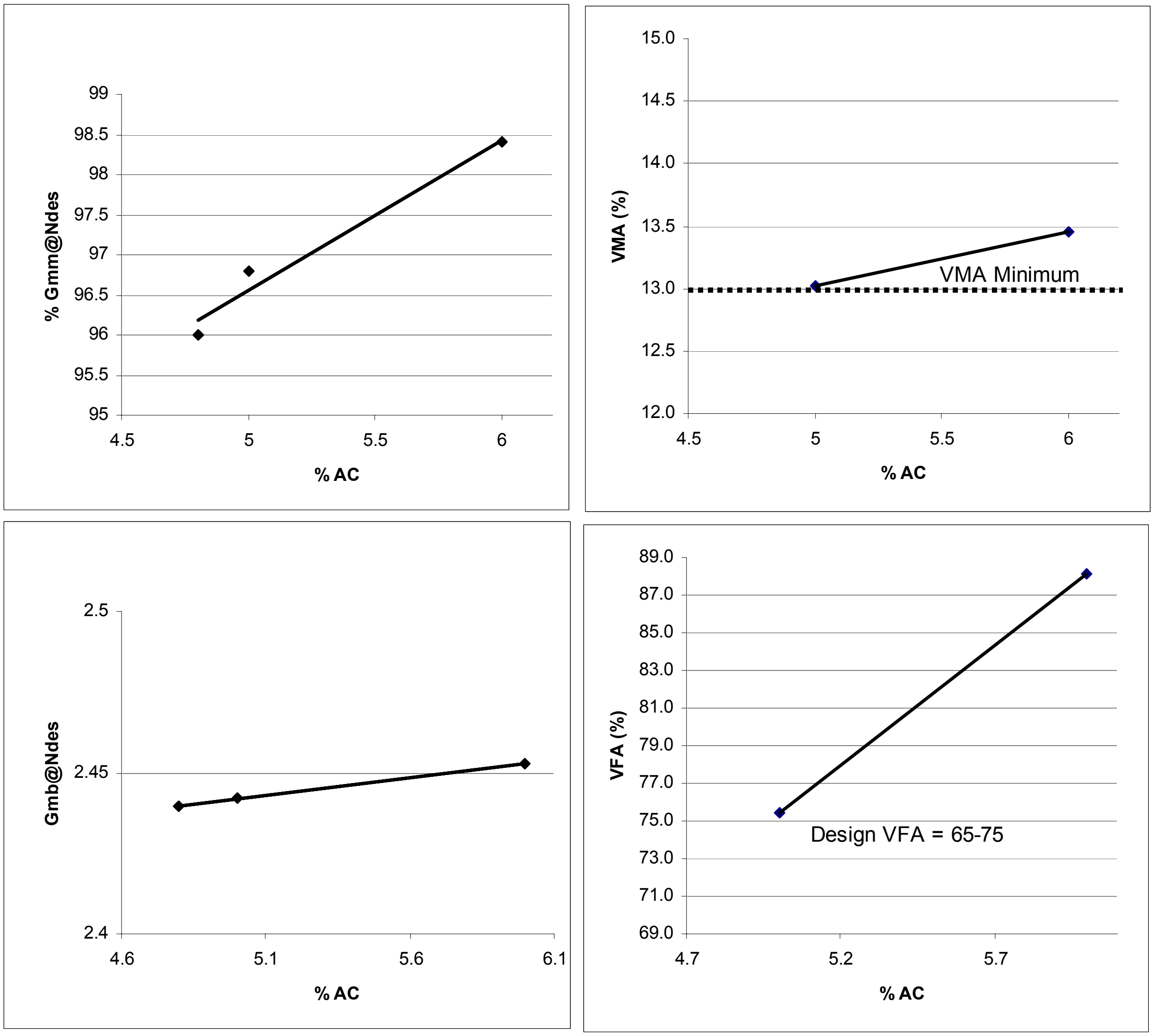


Figure 3.19 Mix Design Verification for I-94, Ann Arbor (3E30)

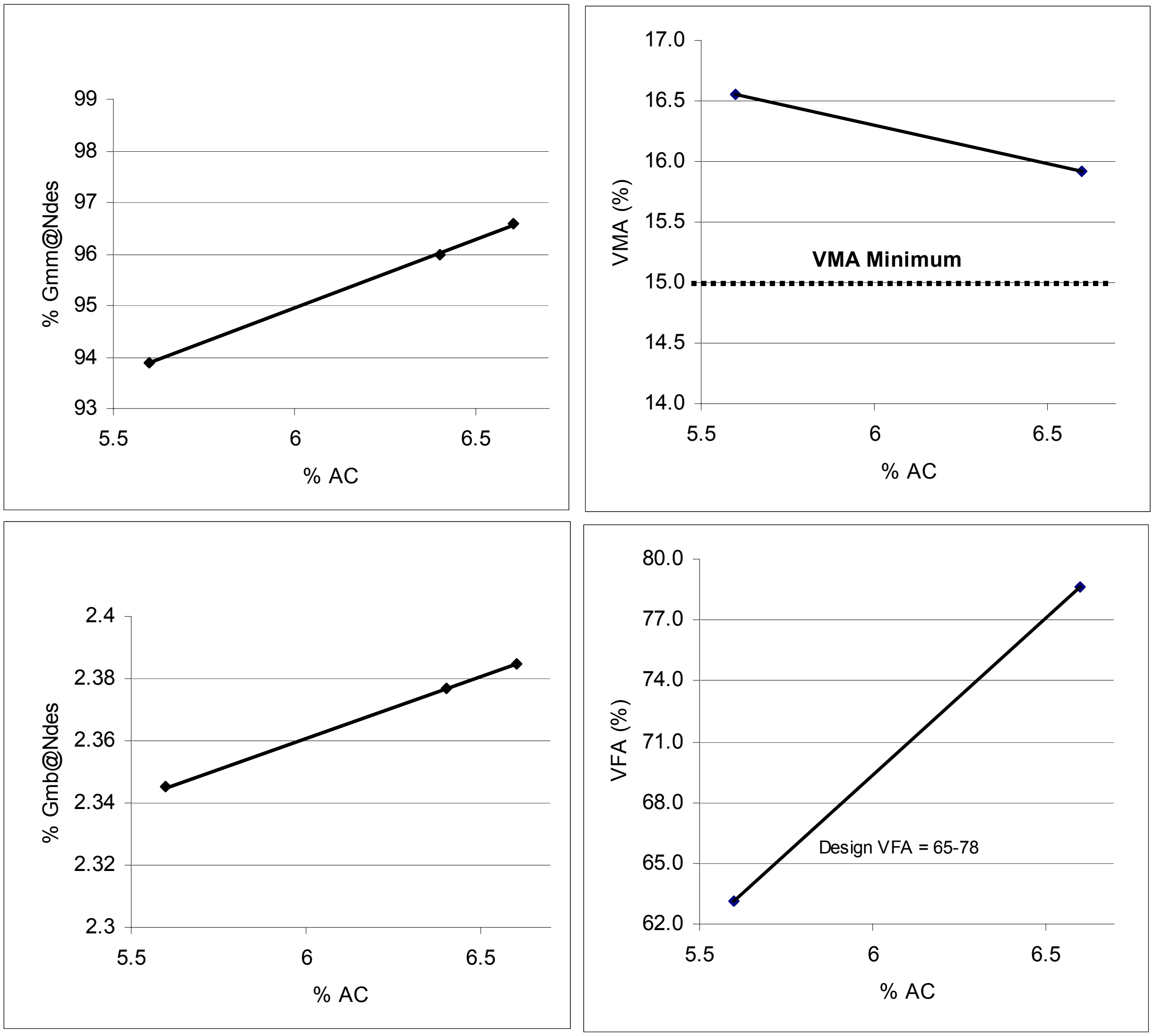


Figure 3.20 Mix Design Verification for M-28, Brimley

CHAPTER 4 CHARACTERIZATION OF ASPHALT BINDERS

4.1 Introduction

Asphalt binders were obtained from three sampling locations: the tank at the plants, recovered from truck HMA samples, and recovered from paver HMA samples for all projects.

Aging processes of tank binders, rolling thin film oven and pressure aging vessel, were done for comparisons with recovered truck and paver samples. The recovered paver samples were also aged using a pressure aging vessel for comparison to tanks samples at similar simulated ages. The methods for recovering and testing all samples were the same to minimize statistical error.

4.2 Superpave Characterization of Tank Asphalt Binders

Characterization of tank asphalt binders was done in accordance with AASHTO test specifications. The test specifications followed were:

-Rotational Viscosity: AASHTO TP48-97

-Dynamic Shear Rheometer: AASHTO TP5-98

-Rolling Thin Film Oven Test: AASHTO T240-97

-Pressure Aging Vessel: AASHTO PP1-98

-Bending Beam Rheometer: AASHTO TP1-98

-Direct Tension Test: AASHTO TP3-00

The test results are summarized in Table 4.1. Michigan Tech test results show that eight of the twenty tank binders did not meet their design grade. Upon further examination seven of the eight failing binders were not being met on the low temperature side and only one on the high temperature side.

4.3 Superpave Characterization of Truck Recovered Asphalt Binders

The characterization of recovered truck asphalt binders was done in accordance with AASHTO test specifications. The test specifications followed were:

-Quantitative Extraction of Bitumen from Bituminous Paving Mixtures: AASHTO T164

-Dynamic Shear Rheometer: AASHTO TP5-98

-Pressure Aging Vessel: AASHTO PP1-98

-Bending Beam Rheometer: AASHTO TP1-98

-Direct Tension Test: AASHTO TP3-00

-The test results are summarized in Table 4.2.

4.4 Superpave Characterization of Paver Recovered Asphalt Binders

The characterization of recovered paver asphalt binders was done in accordance with AASHTO test specifications. The test specifications followed were:

-Quantitative Extraction of Bitumen from Bituminous Paving Mixtures: AASHTO T164

-Dynamic Shear Rheometer: AASHTO TP5-98

-Pressure Aging Vessel: AASHTO PP1-98

-Bending Beam Rheometer: AASHTO TP1-98

-Direct Tension Test: AASHTO TP3-00

-The test results are summarized in Table 4.3.

4.5 Comparison of Superpave Test Results for Tank and Recovered Asphalt Binders

Tables 4.4 through 4.6 summarize the comparisons of binders from the three different field locations: tank, truck, and paver, respectively. Table 4.4 summarizes the comparisons between all of the binders, while Tables 4.5 and 4.6 compare the unmodified and modified binders, respectively. Overall, it was generally found that the recovered binder properties of paver samples was more representative of the corresponding aged tank binder than truck samples. The following sections describe the comparisons between the three sampling locations.

4.5.1 Comparison of Tank and Truck Recovered Binders

The comparisons of tank and truck samples are not very good for the DSR properties as evidence by the low R^2 values for both linear (0.12 and 0.35) and non-linear (0.23 and 0.36) models. The properties for the DSR demonstrate that the truck samples are stiffer than the tank RTFO aged samples at the same test temperature as evidenced by the ratio of the test properties being greater than 1.0 on average in Table 4.4. When the unmodified and modified binder samples are separated into two different classes, the correlations between the truck and tank samples tend to improve, but are still rather poor as they are all below 0.56 for both linear and non-linear relationships for the two different binder classifications (Table 4.5- unmodified and Table 4.6- modified binders).

The correlations for BBR properties is good for stiffness values for grouping all of the binders together, but the m-value is poor. The correlations for the stiffness values are just less than 0.80 for all of the binders grouped together. When the binders are separated into unmodified and modified binder groups, the correlation for the unmodified binder group improves slightly to about 0.80 while the modified binder group has a much poorer correlation of 0.53. None of the correlations between tank and truck samples for the m-values, whether grouped together or classified into unmodified and modified binder groups, were good (less than 0.48 correlation). It was found that the stiffness of the recovered binders were always greater than that of the aged tank binders. The increased stiffness values tend to indicate aging of tank binders is not aging binders enough to be representative of field recovered samples. One consideration for the greater aging effect of recovered samples could be due to the extraction and recovery process of the truck and paver samples. The difference in aging needs more study to determine the phenomenon that is impacting the difference in the recovered and tank aged binders.

The m-values of the recovered binders were found to always be less than the aged tank binders, whether the unmodified and modified binders are classified as one or two groups. Overall the correlations between recovered and aged truck samples and those of the aged tanks samples were poor, less than 0.48, as shown in Tables 4.4 through 4.6.

The correlations between the tank and truck samples for the direct tension test results were reasonably good as the correlations were 0.65 or better. Classifying the modified binders as a separate group, resulted in a correlation of 0.65, whereas the correlation between the unmodified binder group improved to greater than 0.82.

4.5.2 Comparison of Tank and Paver Recovered Binders

The recovered paver test properties in general correlated better with tank samples than recovered truck samples. This was particularly evident when the modified and unmodified binders were placed into two different classification groups as shown in Tables 4.4 through 4.6. It was also demonstrated that aging of the binders is still occurring during the transport and laydown of the HMA as the paver-tank binder property ratios increased over that of the truck-tank binder properties. The exception was that of the low temperature binder properties, the bending beam rheometer properties and the percent strain to fracture in the direct tension test, where these property ratios were nearly the same or decreased slightly.

4.5.3 Comparison of Truck and Paver Recovered Binders

In order to examine why consistently better correlations between recovered paver and tank samples existed than recovered truck and tank samples, a comparison between recovered truck and recovered paver samples was done. The comparison between the two recovered sampling locations would illustrate whether the error, or lack of correlation between recovered truck and tank sample, could otherwise be explained. The correlations between the recovered truck and recovered paver samples generally improved with the exception of the unmodified binder $G^*/\sin(\delta)$ grouping and the direct tension test results.

The comparison of the recovered truck and recovered paver samples, coupled with the comparisons of recovered truck and recovered paver samples to tank samples, indicates the following:

1. Additional aging occurring in the haul time and placement of HMA,
2. The recovery process leading to test property error beyond what is in AASHTO test specifications, and
3. An incomplete absorption of asphalt binder constituents into aggregate that may not be recovered by the abson method. This statement is more intuitive and thus more scientific investigation would need to be done to determine whether it is the recovery process (e.g. chemicals in the recovery process) or that certain fractions of asphalt binders that have been absorbed by aggregate are non-recoverable.

4.6 Penetration Test Results of Tank Asphalt Binders and Recovered Binders

Penetration of tank asphalt and recovered binders were done in accordance with AASHTO T49-97 “Penetration of Bituminous Materials.” The test results for the tank binder samples, recovered truck samples, and recovered paver samples are summarized in Tables 4.7, 4.8, and 4.9, respectively. The sections below provide an brief analysis of the data.

4.6.1 Comparison of Tank and Truck Recovered Binders

Figure 4.6 demonstrate a good correlation, R^2 of 0.69, between the Rolling Thin Film Oven aged tank binders and the recovered truck samples. The correlation is for all of the mixes that used unmodified and modified binders as well as recycled asphalt pavement(RAP). If the mixes containing RAP are removed from the data set, the correlation improves to 0.86. If the recovered binders without RAP are separated into modified and unmodified binder groups, the correlations are 0.79 and 0.04, respectively. The lack of any correlation (0.04) for the modified binders demonstrates the difficulty in using the penetration test as a method to grade modified asphalt binders.

Comparison of binders having been subjected to the pressure aging vessel to simulate field aging show a good correlation of 0.71 when all of the unmodified and modified binders as well as the mixes containing RAP compared against each other as a single group. The relationship is shown in Figure 4.7. When the mixes containing RAP are removed, the correlation improves significantly to 0.89. Separating the unmodified binders from the modified binders yields correlations of 0.78 and 0.22, respectively. The low correlation for the modified binders again demonstrates the difficulty in using the penetration test to grade modified binders.

4.6.2 Comparison of Tank and Paver Recovered Binders

Figure 4.8 shows there is a good correlation, 0.74, between the tank RTFO aged binder samples and the recovered paver "RTFO" aged binders. This correlation includes binders that are unmodified and modified as well as recovered binders from mixes containing RAP.

Removing the binders that have recovered RAP from the group significantly improves the correlation to 0.93. Separating the unmodified and modified binders into two groups, the correlations are 0.70 and 0.00, respectively. The decrease in the correlation of the unmodified binder group compared to the larger modified and unmodified binder group is in part due to the fact that the sample size is reduced. A smaller sample size is influenced more by a single data point that may be an outlier than a larger sample size would be. Consistent with the findings in the previous section, the lack of any correlation between the recovered penetration values of modified binders and those of the same tank RTFO aged binders, demonstrates the poor prediction capability of the penetration test for modified binders.

Comparison of the RTFO and PAV aged tank binders and the recovered and PAV aged paver samples is shown in Figure 4.9. The correlation for all the binders is 0.76. When the mixtures with RAP are removed from the data set, the correlation improves to 0.83. However, when the data set is split into the unmodified and modified binder groups, the correlations decrease considerably to 0.33 and 0.46, respectively. Again, this likely due to outliers affecting the smaller sample sizes as well as the limited range of the penetration values for the two groups, particularly the modified binders.

4.6.3 Comparison of Truck and Paver Recovered Binders

In this section we compare the truck and paver penetration values to determine whether additional aging is occurring in samples during the haul time and placement of the HMA. Figure 4.10 shows a very strong relationship between the recovered truck and recovered paver binder samples as the correlation is 0.96. However the slope of the best fit line equation has a value of 0.89 which is less than 1.0. This indicates that the binders are tending to get softer during the haul time and placement of the HMA according to penetration test values, which is counter intuitive as one would expect the binders to harden due to the additional oxidation occurring at elevated temperatures. Forcing the intercept to go through 0, still yields a high correlation of 0.95 and the slope increases from 0.89 to 0.92.

A similar comparison of the recovered and PAV aged truck and paver samples are shown in Figure 4.11. The correlation is 0.92 and is very good with the slope of the best fit line being 1.02. The slope being greater than 1.0 indicates that after PAV aging of the recovered samples, a limited amount of additional aging is occurring during the haul time and placement of the HMA. If the best fit line is forced through 0, the correlation is still 0.92 and the slope is 1.0, indicating no additional aging is occurring.

Table 4.1 Asphalt Binder Characterization Test Results for Tank Samples

Project	Binder Design Grade	MTU Binder Grade	Rotational Viscosity		Dynamic Shear Rheometer (Unaged Residue), KPa				RTFO Mass Loss, %	Dynamic Shear Rheometer (RTFO Residue), KPa			
			135°C	165°C	52°C	58°C	64°C	70°C		52°C	58°C	64°C	70°C
M-32, Lachine	58-28	58-28	0.290	0.105		1.29			0.037		3.05		
M-45, Grand Rapids	58-28	58-22	0.273	0.103		1.17			0.021		2.71		
Old M-14, Plymouth	64-22	58-28	0.378	0.132		2.21			0.019		4.47	2.51	
M-35, Escanaba	58-28	58-28	0.292	0.115		1.28			0.048		3.52		
M-52, St. Charles [R28]	58-28	58-22	0.338	0.128		1.73			0.042		4.50	2.09	
M-90, Lexington	64-28	64-22	0.448	0.153			1.37		0.047			3.42	
M-50, Brooklyn	58-28	58-22	0.278	0.113		1.23			0.018		3.24		
US-31, Elk Rapids	58-28	58-28	0.282	0.097		1.25			0.044		2.93		
US-24, Monroe [R15]	58-28	58-22	0.323	0.115		1.61			0.066		4.43	2.07	
I-75, Indian River [R26]	58-28	58-22	0.298	0.115		1.98			0.038		2.62		
US-27, Grayling [R22]	58-28	58-28	0.295	0.103		1.20			0.052		3.18		
M-43, Lansing	70-22	70-16	0.805	0.253				1.28	0.012				2.52
I-75, Auburn Hills [R10]	64-22	64-22	0.437	0.133			1.41		0.035			3.39	
I-75, Flint	70-22	70-22	0.675	0.195				1.20	0.018				2.26
I-75, Saginaw	64-22	64-22	0.493	0.153			1.71		0.043			4.38	2.05
8-Mile Road, Warren	70-22	70-22	0.507	0.120				1.23	0.037				2.71
I-94(4E30), Ann Arbor [R15]	70-22	70-22	0.658	0.177				1.40	0.007				2.42
I-94(SMA), Ann Arbor	70-22	52-22	0.175	0.068	1.27	0.65			0.039	3.35			
I-94(3E30), Ann Arbor [R10]	58-28	58-22	0.272	0.103		1.49			0.013		4.06	1.82	
M-28, Brimley	58-28	58-28	0.293	0.100		1.22			0.080		2.83		

R- Designates reclaimed asphalt pavement was an HMA constituent and its percentage by weight of the mix.

Table 4.1 Asphalt Binder Characterization Test Results for Tank Samples (continued)

Project	Dynamic Shear Rheometer (RTFO & PAV Aged), KPa					Bending Beam Rheometer								Direct Tension Test Failure Strain, %			
	16°C	19°C	22°C	25°C	28°C	-6°C		-12°C		-18°C		-24°C		-6°C	-12°C	-18°C	-24°C
						S, MPa	m-value	S, MPa	m-value	S, MPa	m-value	S, MPa	m-value				
M-32, Lachine		4724								245	0.304				2.93	0.77	
M-45, Grand Rapids		5443	3927					144	0.347	246	0.282				1.70		
Old M-14, Plymouth		8706	6370	4475				67	0.375	171	0.322	182	0.278			1.22	
M-35, Escanaba		4301								206	0.310				1.48	0.81	
M-52, St. Charles [R28]		4850	3898					110	0.394	253	0.296				2.43	0.69	
M-90, Lexington			4601					113	0.342	210	0.296					0.99	
M-50, Brooklyn		6034	4346					112	0.329	235	0.284				1.77		
US-31, Elk Rapids		4661	3935					94	0.369	252	0.314				2.27	0.75	
US-24, Monroe [R15]		5409	3789					118	0.327	265	0.271				2.40	0.54	
I-75, Indian River [R20]		4677	3545							232	0.297					1.07	
US-27, Grayling [R22]		4527								233	0.307				2.37	0.91	
M-43, Lansing				5007	3359	89	0.366	202	0.294					3.98			
I-75, Auburn Hills [R10]				3967				183	0.332						1.18		
I-75, Flint				5602	3769			220	0.301					1.96	0.83		
I-75, Saginaw				4743				185	0.319						1.20		
8-Mile Road, Warren				5652	3876			218	0.308						1.20		
I-94(4E30), Ann Arbor [R15]				5949	3860			203	0.310					3.63	0.71		
I-94(SMA), Ann Arbor	4900							79	0.365	200	0.295					1.10	
I-94(3E30), Ann Arbor [10]		5737	4260					115	0.302	216	0.270				1.97		
M-28, Brimley		4541	3365					101	0.382	222	0.303				1.97	0.74	

Table 4.2 Asphalt Binder Characterization Test Results for Truck Samples

Project	Binder Design Grade	MTU Binder Grade	Rotational Viscosity		Dynamic Shear Rheometer (Unaged Residue), KPa				RTFO Mass Loss, %	Dynamic Shear Rheometer (RTFO Residue), KPa			
			135°C	165°C	52°C	58°C	64°C	70°C		52°C	58°C	64°C	70°C
M-32, Lachine	58-28	58-28									2.92		
M-45, Grand Rapids	58-28	58-22									2.99		
Old M-14, Plymouth	64-22	58-16									2.77		
M-35, Escanaba	58-28	58-28									2.31		
M-52, St. Charles	58-28	58-22									3.79		
M-90, Lexington	64-28	70-22										6.15	2.76
M-50, Brooklyn	58-28	58-22									3.71		
US-31, Elk Rapids	58-28	58-28									3.00		
US-24, Monroe	58-28	64-22									4.79	2.2	
I-75, Indian River	58-28	58-22									4.12	1.86	
US-27, Grayling	58-28	58-22									4.46	2.05	
M-43, Lansing	70-22	70-16											4.23
I-75, Auburn Hills	64-22	64-22										3.87	
I-75, Flint	70-22	70-16											2.82
I-75, Saginaw	64-22	70-22											2.46
8-Mile Road, Warren	70-22	70-16											3.19
I-94(4E30), Ann Arbor	70-22	70-16											3.16
I-94(SMA), Ann Arbor	70-22	70-16											2.22
I-94(3E30), Ann Arbor	58-28	70-16											3.93
M-28, Brimley	58-28	64-22									3.42	2.54	

Table 4.2 Asphalt Binder Characterization Test Results for Truck Samples (continued)

Project	Dynamic Shear Rheometer (RTFO & PAV Aged), KPa					Bending Beam Rheometer								Direct Tension Test Failure Strain, %			
						-6°C		-12°C		-18°C		-24°C					
	16°C	19°C	22°C	25°C	28°C	S, MPa	m-value	S, MPa	m-value	S, MPa	m-value	S, MPa	m-value	-6°C	-12°C	-18°C	-24°C
M-32, Lachine	6760	4528								270	0.303					0.98	
M-45, Grand Rapids		5417	3899					128	0.316	246	0.275				1.16		
Old M-14, Plymouth				5720	3941	89	0.326	189	0.277					2.33			
M-35, Escanaba		4565	3162							226	0.300					1.08	
M-52, St. Charles		7566	5508	3860		53	0.384	133	0.320	283	0.269				2.16		
M-90, Lexington			5349	3905				138	0.319						2.08		
M-50, Brooklyn			4765					126	0.310						1.75		
US-31, Elk Rapids		5317	3511					106	0.364	259	0.314				1.75		
US-24, Monroe			4169					120	0.347	250	0.294				2.24		
I-75, Indian River		5929	3990					117	0.333	221	0.294				2.05		
US-27, Grayling			4252					123	0.337	243	0.288				1.89		
M-43, Lansing		2552	3741	5245		113	0.316	245	0.268					2.04			
I-75, Auburn Hills			5622	3850				177	0.322						1.33		
I-75, Flint					4288	114	0.335	229	0.275					1.82			
I-75, Saginaw				3820				176	0.327	324	0.245				1.30		
8-Mile Road, Warren					3820	95	0.366	211	0.295						1.14		
I-94(4E30), Ann Arbor					4166	119	0.337	226	0.284					2.74			
I-94(SMA), Ann Arbor					3827	111	0.329	214	0.291					1.53			
I-94(3E30), Ann Arbor					5149 (3664)[1]	115	0.317							1.91			
M-28, Brimley		4730						108	0.344	235	0.292				2.23		

1 The Dynamic Shear Rheometer test in parentheses ()result is at 31°C.

Table 4.3 Asphalt Binder Characterization Test Results for Paver Samples

Project	Binder Design Grade	MTU Binder Grade	Rotational Viscosity		Dynamic Shear Rheometer (Unaged Residue), KPa				RTFO Mass Loss, %	Dynamic Shear Rheometer (RTFO Residue), KPa			
			135°C	165°C	52°C	58°C	64°C	70°C		52°C	58°C	64°C	70°C
M-32, Lachine	58-28	58-22									3.46		
M-45, Grand Rapids	58-28	58-22									2.52		
Old M-14, Plymouth	64-22	N/T											
M-35, Escanaba	58-28	64-28									5.10	2.96	
M-52, St. Charles	58-28	64-22									9.28	3.88	
M-90, Lexington	64-28	70-22										5.05	2.34
M-50, Brooklyn	58-28	58-22									4.39		
US-31, Elk Rapids	58-28	58-22									3.85		
US-24, Monroe	58-28	64-22									6.12	2.72	
I-75, Indian River	58-28	64-28									4.81	2.44	
US-27, Grayling	58-28	64-28									4.65	2.32	
M-43, Lansing	70-22	70-16											3.61
I-75, Auburn Hills	64-22	64-22										4.00	1.96
I-75, Flint	70-22	70-16											2.96
I-75, Saginaw	64-22	70-22										6.44	2.96
8-Mile Road, Warren	70-22	70-16											3.91
I-94(4E30), Ann Arbor	70-22	70-16											3.98
I-94(SMA), Ann Arbor	70-22	70-16											2.67
I-94(3E30), Ann Arbor	58-28	70-16											3.83
M-28, Brimley	58-28	58-22									3.73		

Table 4.3 Asphalt Binder Characterization Test Results for Paver Samples (continued)

Project	Dynamic Shear Rheometer (RTFO & PAV Aged), KPa					Bending Beam Rheometer								Direct Tension Test Failure Strain, %			
	16°C	19°C	22°C	25°C	28°C	-6°C		-12°C		-18°C		-24°C		0°C	-6°C	-12°C	-18°C
						S, MPa	m-value	S, MPa	m-value	S, MPa	m-value	S, MPa	m-value				
M-32, Lachine	7420	4227						109	0.351	235	0.294					1.75	
M-45, Grand Rapids		5623	4060					119	0.312	250	0.264					2.23	
Old M-14, Plymouth																	
M-35, Escanaba			4080							117	0.338					2.11	0.66
M-52, St. Charles		6757	4799	3319				144	0.306	289	0.267					1.47	
M-90, Lexington			4813	2806				102	0.333	252	0.255					1.79	
M-50, Brooklyn		6694	4972					128	0.313							1.31	
US-31, Elk Rapids		5677	3955					111	0.354							2.00	
US-24, Monroe			4209					119	0.349	260	0.277					1.17	
I-75, Indian River			4178					123	0.332	242	0.284					2.02	0.72
US-27, Grayling		5472	3714					124	0.331	249	0.289					1.72	0.90
M-43, Lansing				7469	5407 (3680)[1]	107	0.328								2.40		
I-75, Auburn Hills				3849				181	0.322							1.65	
I-75, Flint				6412	4705	97	0.334	229	0.273						1.81		
I-75, Saginaw				4000	2704			179	0.318	355	0.269					1.20	
8-Mile Road, Warren				5309	3831	101	0.353	213	0.293						4.25	0.67	
I-94(4E30), Ann Arbor				6360	4818	117	0.330	212	0.285						2.90		
I-94(SMA), Ann Arbor				5557	3704	108	0.331	209	0.290						2.25		
I-94(3E30), Ann Arbor					5310 (3742)[2]	117	0.331							1.40	0.90		
M-28, Brimley		4510						101	0.320	228	0.293					1.28	

Table 4.4 Summary of Aging and/or Binder Recovery Effects on All Binder Properties

Binder Property	Statistic	Truck-Tank Binder Property Ratio	Paver-Tank Binder Property Ratio	Paver-Truck Binder Property Ratio
Dynamic Shear Rheometer (RTFO Aged), G*/sin(δ)	Average	1.13	1.38	1.11
	Std. Dev.	0.27	0.19	0.16
	High Value	1.68	1.84	1.31
	Low Value	0.62	0.93	0.82
	R^2, Linear	0.12	0.74	0.52
	R^2, Non-Linear	0.23	0.79	0.53
Dynamic Shear Rheometer (RTFO & PAV Aged), G*x[sin(δ)]	Average	1.06	1.13	1.03
	Std. Dev.	0.12	0.18	0.14
	High Value	1.28	1.61	1.42
	Low Value	0.81	0.84	0.72
	R^2, Linear	0.35	0.54	0.69
	R^2, Non-Linear	0.36	0.63	0.69
Bending Beam Rheometer (RTFO & PAV Aged), S	Average	1.10	1.06	0.98
	Std. Dev.	0.17	0.11	0.08
	High Value	1.79	1.31	1.12
	Low Value	0.87	0.83	0.74
	R^2, Linear	0.70	0.68	0.88
	R^2, Non-Linear	0.78	0.75	0.89
Bending Beam Rheometer (RTFO & PAV Aged), m-value	Average	0.95	0.93	0.99
	Std. Dev.	0.05	0.06	0.04
	High Value	1.08	1.07	1.10
	Low Value	0.81	0.79	0.90
	R^2, Linear	0.31	0.37	0.76
	R^2, Non-Linear	0.35	0.52	0.78
Direct Tension Test (RTFO & PAV Aged), Percent Strain	Average	0.93	0.92	1.01
	Std. Dev.	0.26	0.30	0.32
	High Value	1.40	1.65	1.92
	Low Value	0.38	0.41	0.47
	R^2, Linear	0.69	0.54	0.18
	R^2, Non-Linear	0.78	0.54	0.20

Table 4.5 Summary of Aging and/or Binder Recovery Effects on Unmodified Binder Properties

Binder Property	Statistic	Truck-Tank Binder Property Ratio	Paver-Tank Binder Property Ratio	Paver-Truck Binder Property Ratio
Dynamic Shear Rheometer (RTFO Aged), G*/sin(δ)	Average	0.94	1.29	1.08
	Std. Dev.	0.19	0.18	0.21
	High Value	1.15	1.48	1.28
	Low Value	0.62	0.93	0.82
	R^2, Linear	0.13	0.81	0.19
	R^2, Non-Linear	0.31	0.88	0.35
Dynamic Shear Rheometer (RTFO & PAV Aged), G*x[sin(δ)]	Average	1.07	1.11	1.00
	Std. Dev.	0.11	0.13	0.14
	High Value	1.28	1.39	1.29
	Low Value	0.89	0.89	0.72
	R^2, Linear	0.31	0.69	0.79
	R^2, Non-Linear	0.37	0.79	0.80
Bending Beam Rheometer (RTFO & PAV Aged), S	Average	1.09	1.07	0.97
	Std. Dev.	0.10	0.13	0.09
	High Value	1.22	1.23	1.12
	Low Value	0.89	0.83	0.74
	R^2, Linear	0.78	0.77	0.76
	R^2, Non-Linear	0.82	0.77	0.76
Bending Beam Rheometer (RTFO & PAV Aged), m-value	Average	0.96	0.94	0.98
	Std. Dev.	0.06	0.06	0.05
	High Value	1.08	1.07	1.05
	Low Value	0.81	0.81	0.90
	R^2, Linear	0.22	0.39	0.76
	R^2, Non-Linear	0.22	0.44	0.76
Direct Tension Test (RTFO & PAV Aged), Percent Strain	Average	0.99	0.95	0.98
	Std. Dev.	0.23	0.33	0.41
	High Value	1.33	1.65	1.92
	Low Value	0.64	0.49	0.52
	R^2, Linear	0.82	0.09	0.00
	R^2, Non-Linear	0.85	0.72	0.85

Table 4.6 Summary of Aging and/or Binder Recovery Effects on Modified Binder Properties

Binder Property	Statistic	Truck-Tank Binder Property Ratio	Paver-Tank Binder Property Ratio	Paver-Truck Binder Property Ratio
Dynamic Shear Rheometer (RTFO Aged), G*/sin(δ)	Average	1.29	1.41	1.10
	Std. Dev.	0.20	0.16	0.15
	High Value	1.68	1.64	1.26
	Low Value	1.14	1.18	0.85
	R^2, Linear	0.47	0.76	0.80
	R^2, Non-Linear	0.56	0.77	0.84
Dynamic Shear Rheometer (RTFO & PAV Aged), G*x[sin(δ)]	Average	1.00	1.08	1.12
	Std. Dev.	0.12	0.23	0.16
	High Value	1.14	1.49	1.42
	Low Value	0.81	0.84	1.00
	R^2, Linear	0.39	0.53	0.69
	R^2, Non-Linear	0.39	0.59	0.74
Bending Beam Rheometer (RTFO & PAV Aged), S	Average	1.06	1.03	0.98
	Std. Dev.	0.14	0.10	0.09
	High Value	1.27	1.20	1.10
	Low Value	0.87	0.88	0.78
	R^2, Linear	0.53	0.59	0.98
	R^2, Non-Linear	0.53	0.59	0.98
Bending Beam Rheometer (RTFO & PAV Aged), m-value	Average	0.94	0.95	1.00
	Std. Dev.	0.04	0.04	0.04
	High Value	1.03	1.02	1.10
	Low Value	0.86	0.90	0.95
	R^2, Linear	0.48	0.43	0.81
	R^2, Non-Linear	0.48	0.43	0.82
Direct Tension Test (RTFO & PAV Aged), Percent Strain	Average	0.97	0.96	1.00
	Std. Dev.	0.25	0.34	0.23
	High Value	1.21	1.40	1.24
	Low Value	0.51	0.56	0.59
	R^2, Linear	0.64	0.79	0.37
	R^2, Non-Linear	0.65	0.81	0.42

Table 4.7 Penetration Test Results of Tank Binder

Project	Original Penetration	RTFO Penetration	PAV Penetration
M-32, Lachine	129	68	34
M-45, Grand Rapids	102	68	32
Old M-14, Plymouth	74	44	23
M-35, Escanaba	123	68	33
M-52, St. Charles	104	61	29
M-90, Lexington	80	48	25
M-50, Brooklyn	103	56	31
US-31, Elk Rapids	123	65	31
US-24, Monroe	108	57	30
I-75, Indian River	125	66	34
US-27, Grayling	125	66	33
M-43, Lansing	51	36	19
I-75, Auburn Hills	66	41	18
I-75, Flint	56	37	17
I-75, Saginaw	64	36	21
8-Mile Road, Warren	50	36	14
I-94(4E30), Ann Arbor	51	36	14
I-94(SMA), Ann Arbor	180	88	38
I-94(3E30), Ann Arbor	106	58	27
M-28, Brimley	126	73	32

Table 4.8 Penetration Test Results of Recovered Truck Binders

Project	"RTFO" Penetration	PAV Penetration
M-32, Lachine	68	32
M-45, Grand Rapids	58	28
Old M-14, Plymouth	28	17
M-35, Escanaba	59	29
M-52, St. Charles	43	21
M-90, Lexington	36	22
M-50, Brooklyn	56	24
US-31, Elk Rapids	69	32
US-24, Monroe	56	27
I-75, Indian River	58	27
US-27, Grayling	56	27
M-43, Lansing	22	12
I-75, Auburn Hills	39	17
I-75, Flint	31	12
I-75, Saginaw	36	18
8-Mile Road, Warren	28	14
I-94(4E30), Ann Arbor	25	14
I-94(SMA), Ann Arbor	31	15
I-94(3E30), Ann Arbor	22	10
M-28, Brimley	61	27

Table 4.9 Penetration Test Results of Recovered Paver Binders

Project	"RTFO" Penetration	PAV Penetration
M-32, Lachine	65	35
M-45, Grand Rapids	59	28
Old M-14, Plymouth	Not Sampled	Not Sampled
M-35, Escanaba	52	25
M-52, St. Charles	41	23
M-90, Lexington	40	22
M-50, Brooklyn	49	25
US-31, Elk Rapids	58	34
US-24, Monroe	53	28
I-75, Indian River	51	27
US-27, Grayling	53	28
M-43, Lansing	23	13
I-75, Auburn Hills	33	12
I-75, Flint	27	12
I-75, Saginaw	31	16
8-Mile Road, Warren	26	13
I-94(4E30), Ann Arbor	22	14
I-94(SMA), Ann Arbor	28	12
I-94(3E30), Ann Arbor	22	13
M-28, Brimley	57	26

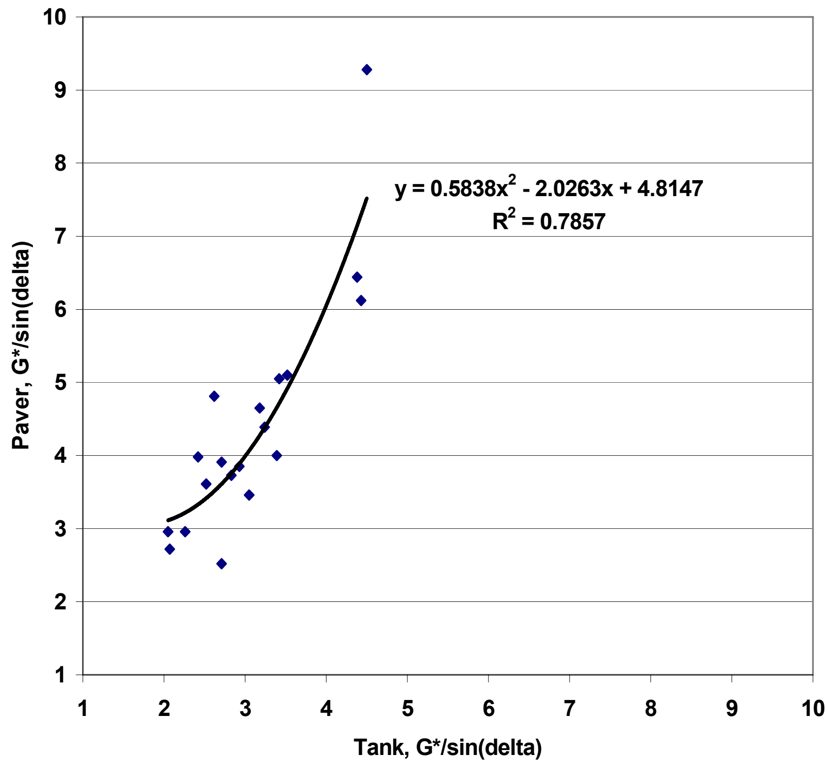


Figure 4.1 Comparison of Tank and Binder G*/sin(delta) of "RTFO" Aged Binders

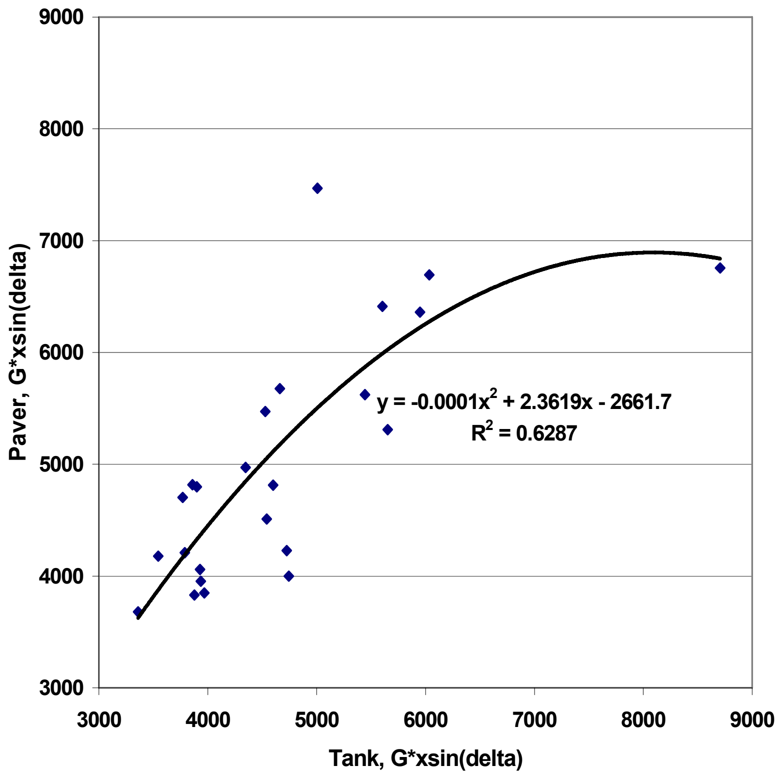


Figure 4.2 Comparison of Tank and Paver G*xsin(delta) of "RTFO" and PAV Aged Binders

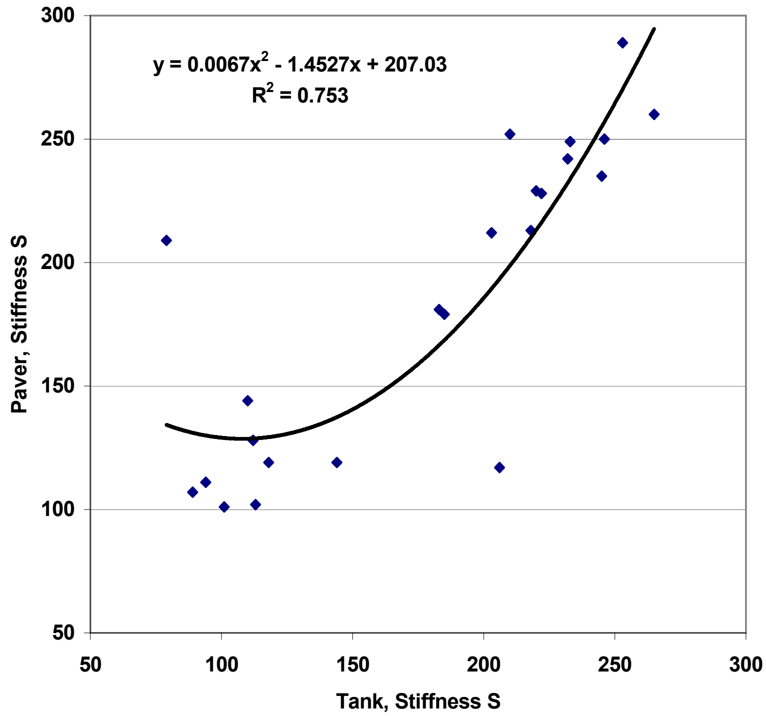


Figure 4.3 Comparison of Tank and Paver Stiffness, S, of "RTFO" and PAV Aged Binders

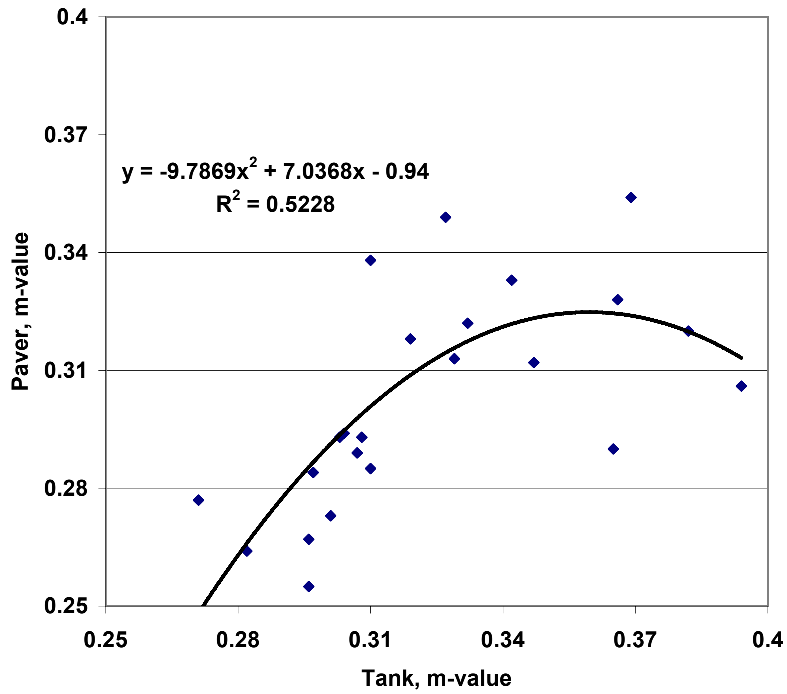


Figure 4.4 Comparison of Tank and Paver M-Values of “RTFO” and PAV Aged Binders

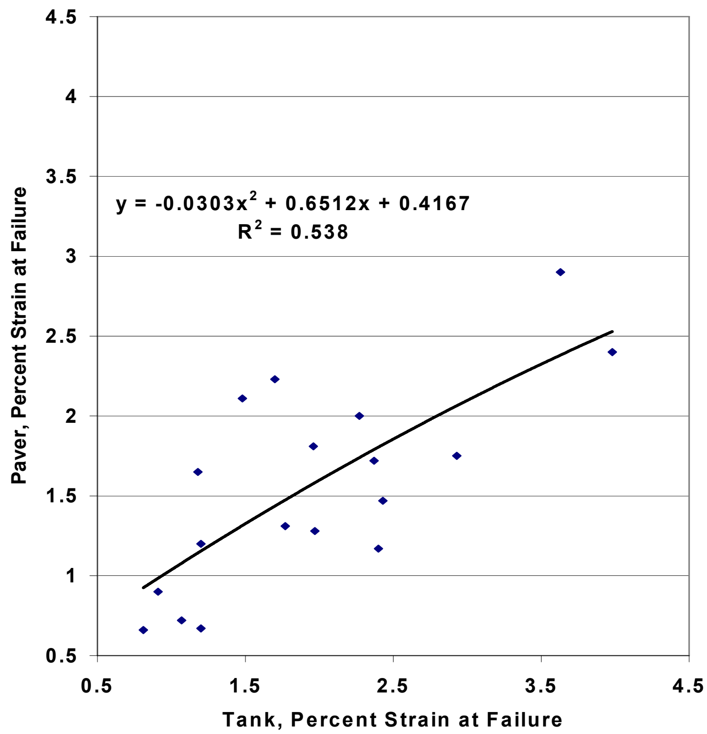


Figure 4.5 Comparison of Tank and Paver Percent Strain at Failure of Direct Tension Test Results of "RTFO" and PAV Aged Binders

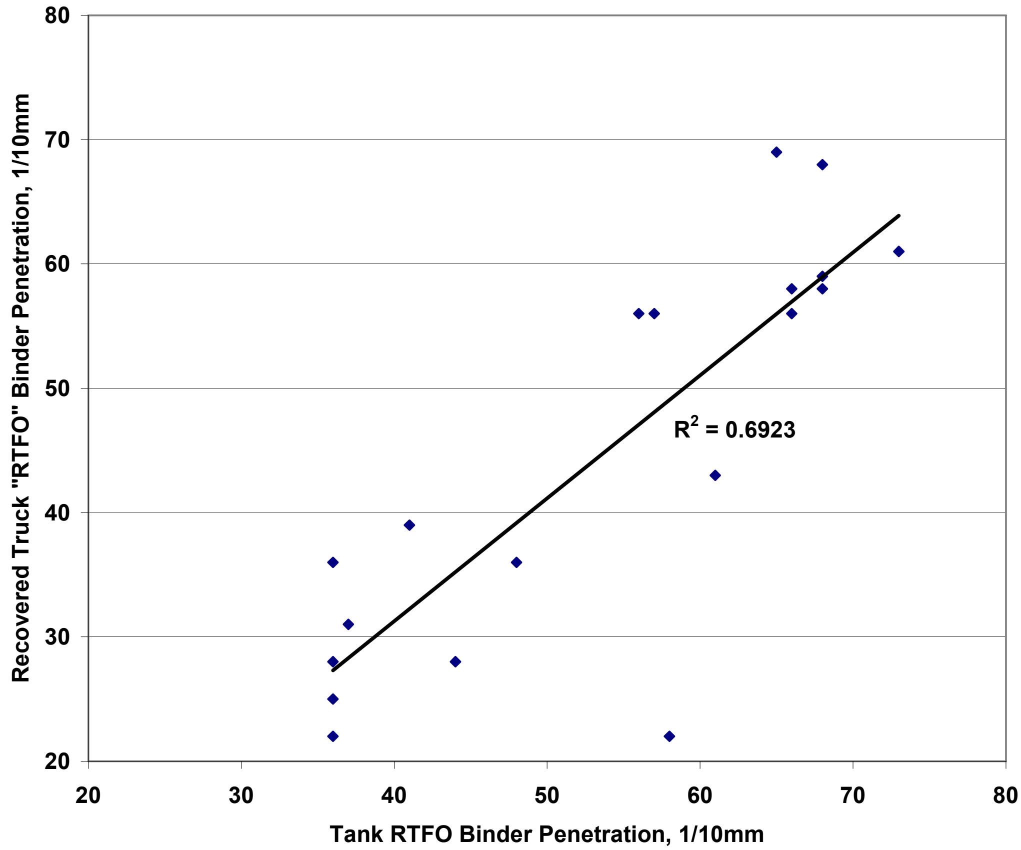


Figure 4.6 Comparison of Tank RTFO and Recovered Truck "RTFO" Penetration Test Results

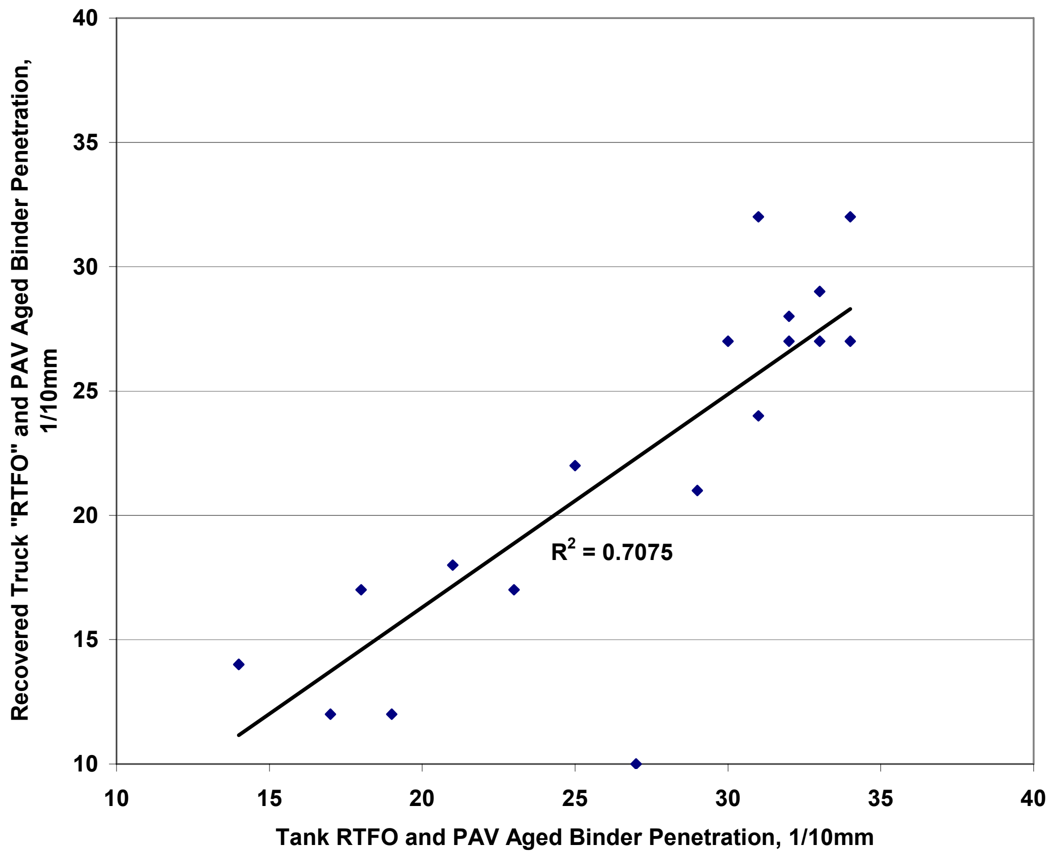


Figure 4.7 Comparison of Tank RTFO and PAV and Recovered Truck "RTFO" and PAV Penetration Test Results

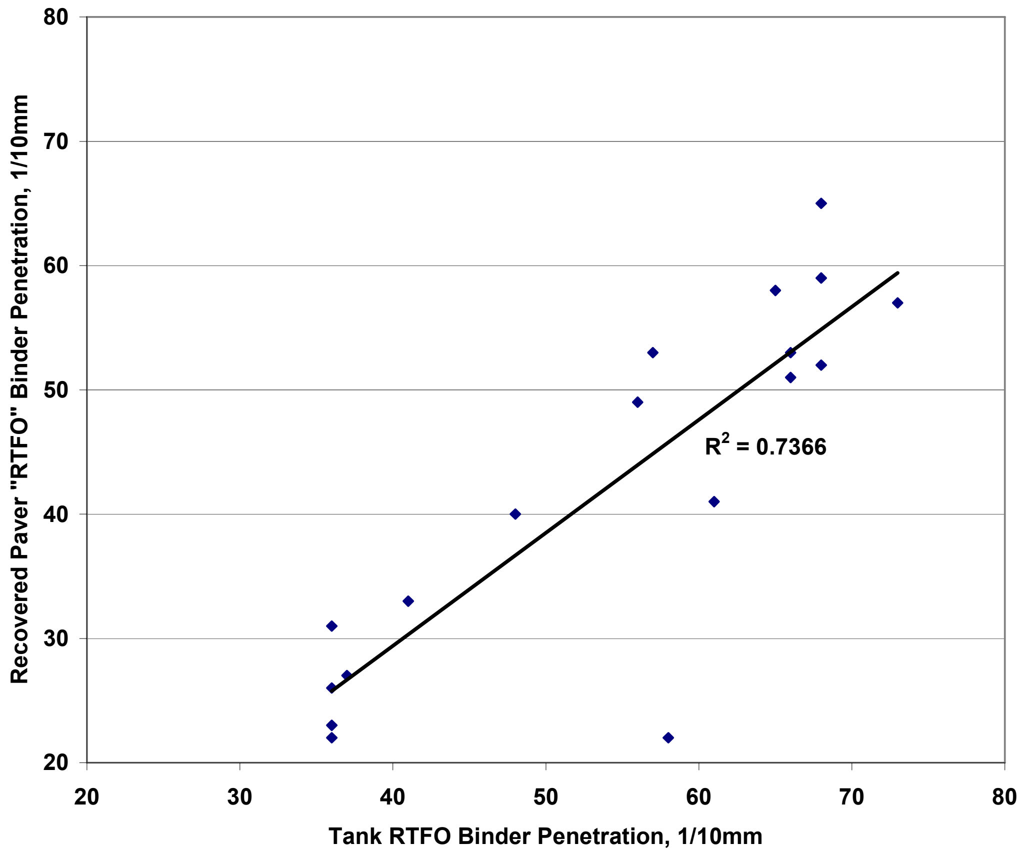


Figure 4.8 Comparison of Tank RTFO and Recovered Truck "RTFO" Penetration Test Results

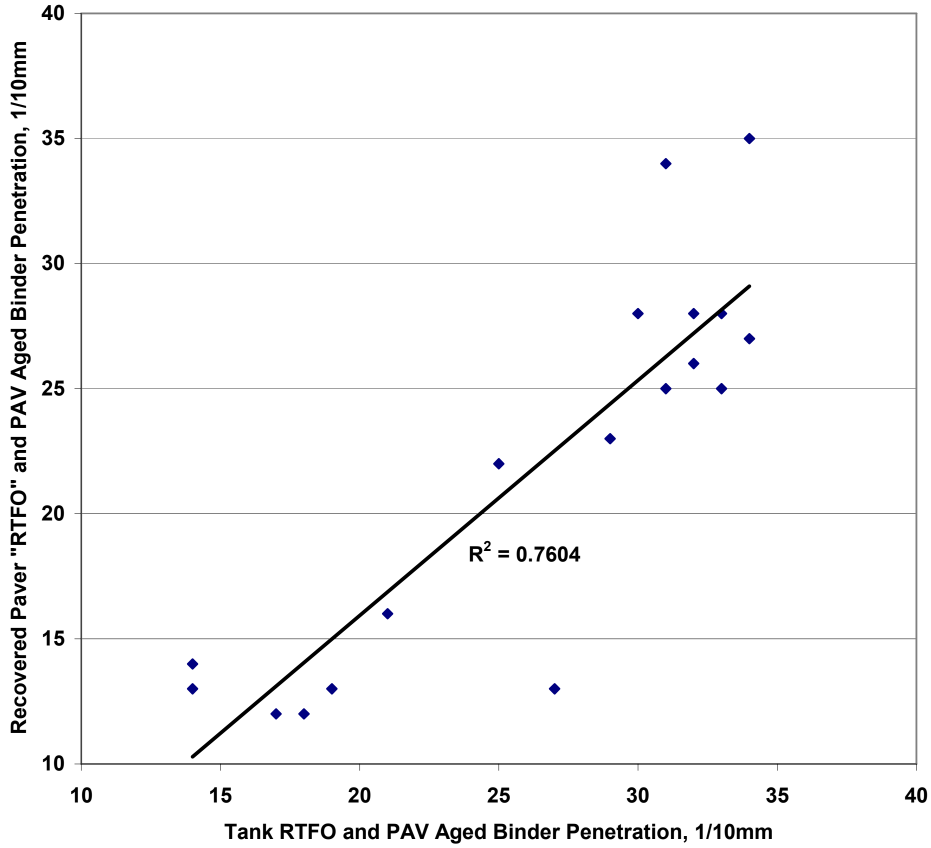


Figure 4.9 Comparison of Tank RTFO and PAV and Recovered Paver "RTFO" and PAV Penetration Test Results

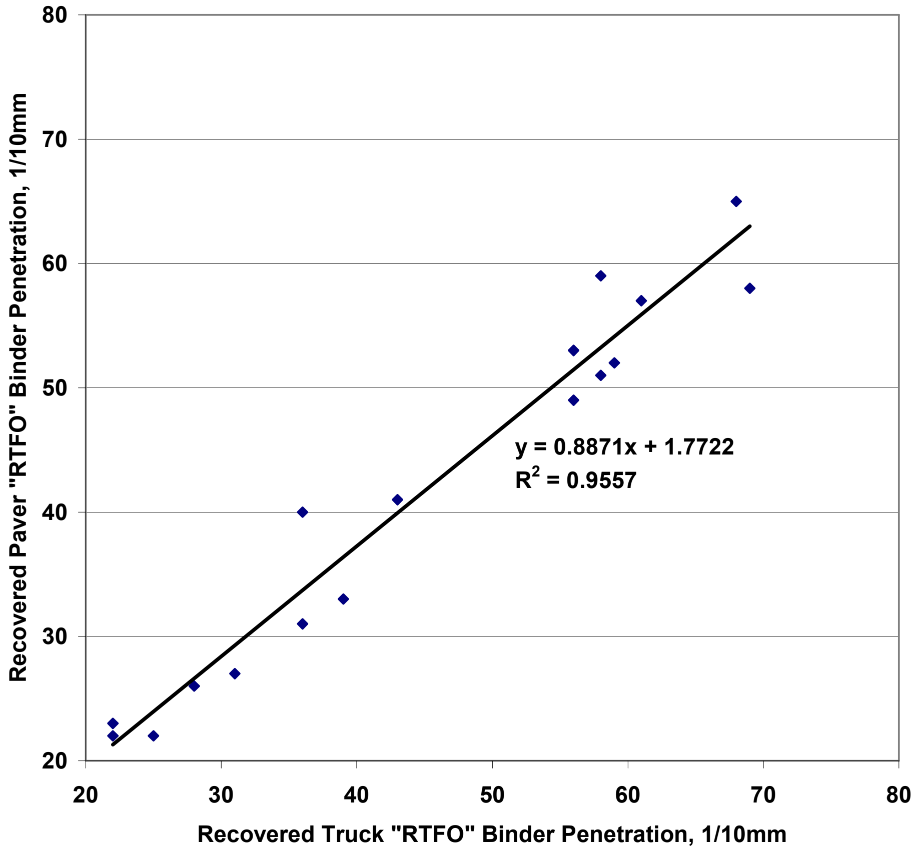


Figure 4.10 Comparison of Recovered Truck "RTFO" and Recovered Paver "RTFO" Penetration Test Results

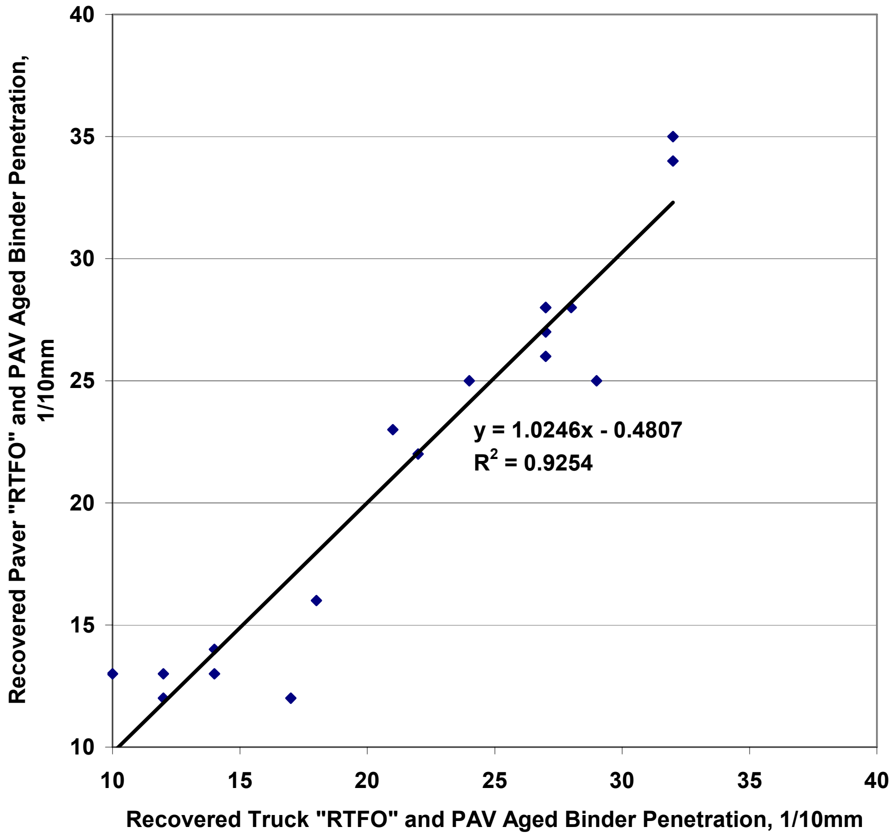


Figure 4.11 Comparison of Recovered Truck "RTFO" and PAV and recovered paver "RTFO" and PAV penetration test results

CHAPTER 5 DESIGN OF EXPERIMENT

5.1 Introduction

Design of experiments need to be developed for statistical testing of hypotheses. It is critical in establishing the design of the experiment to have a firm understanding of the factors under study and any assumptions that are made. Without careful consideration of potentially relevant variables or inappropriate assumptions, a statistical analysis may lead to a biased or unsubstantiated conclusion. The impetus for a design of experiment is to outline in broad terms what are the research questions, which will form the basis of experimental hypotheses.

5.2 Verification of Material Properties

Verification of materials and material properties are necessary prior to performance testing of materials. This is to ensure that significantly different materials were not produced and/or are being evaluated or may explain later inconsistent performance test results. The materials to be verified are aggregate, asphalt binders, and hot mix asphalt using existing property test equipment in accordance with MDOT and AASHTO specifications.

5.2.1 Verification of Aggregate Properties

The verification of aggregate properties will be done according to MDOT and AASHTO testing criteria. The specific aggregate properties that will be verified are:

- Specific gravity,
- Gradation,
- Fine aggregate angularity, and
- Number of crushed faces.

Any significant change in these properties from the material used in the original mixture design will change the volumetric properties of the HMA as well as demand for asphalt binder and thus change the volumetric properties of the HMA. The entire gradation, i.e. every sieve size from the nominal maximum to the percent passing the 0.075mm sieve, will be statistically compared between mixture designs, truck samples and behind the paver samples.

5.2.2 Verification of Asphalt Binder Properties

The binder properties will be verified using Superpave binder equipment following MDOT and AASHTO specifications and provisions. The equipment used to characterize the binders are:

- Rotational Viscosity,
- Dynamic Shear Rheometer (unaged binder),
- Dynamic Shear Rheometer (Rolling Thin Film Oven Aged),
- Bending Beam Rheometer, and
- Direct Tension Test.

A rolling thin film oven and pressure aging vessel will be used to age the binders at the appropriate stages of the binder characterization.

The rationale for characterizing the binder is to examine whether the binder properties are a significant variable in mixture performance. Many in the asphalt industry believe that the binder properties can significantly enhance mixture permanent deformation, fatigue cracking, and thermal cracking performance. Although the researchers are skeptical in the realized benefit of "grade bumping," they do acknowledge that the low temperature properties binders do have a significant effect in mitigating thermal cracking. Thus, it is the intent of the researchers to remain unbiased in the research approach.

Statistical testing of binder properties between tank and recovered samples will be done to examine whether or not there is a difference. Tank samples will be appropriately aged in the rolling thin film oven (RTFO) to correspond with the recovered samples. Statistical comparisons of dynamic shear rheometer test results will be done on the paired samples. Paired samples are ones, tank and recovered, which come from or are used in the same mixture. Both the recovered and RTFO aged samples will be further aged in a pressure aging vessel to simulate the long-term aging of the binders. Statistical comparisons of dynamic shear rheometer, bending beam rheometer and direct tension tester test results will then be conducted. These analyses would be beneficial in evaluating the current specifications as well as determining whether or not recovered binders may be useful in conflict resolution.

5.2.3 Verification of Mixture Designs

The mixture design will be verified using the selected optimum asphalt binder content. This will also provide specimens for performance testing. Thus, the binder contents will not be varied from the optimum selected for design unless the design cannot be verified.

5.2.4 Verification of Volumetric Properties

The volumetric properties of the field produced mixtures will be verified and compared. One of the critical elements will be statistical comparison of mixture volumetric properties at each stage of the production process to the actual design. Additional statistical comparisons between truck samples and behind-the paver samples will be done too.

Mixture properties which will be statistically compared will be maximum theoretical specific gravity, bulk specific gravity at N_{design}, and binder content. In addition to the measured properties, calculated properties will also be compared. Calculated properties will include air voids at N_{design}, voids in the mineral aggregate at N_{design}, voids filled with asphalt at N_{design}, and binder film thickness. The values used in making calculations will include a range based on reliability so that tolerances on the calculations can be examined. An example would be in calculating film thickness, the effective binder content and percent retained on each sieve size are used in the binder film thickness calculation. The effective binder content and percent retained on each sieve size have variability in the truck and behind the paver samples, and these variability's should be captured in the level of reliability of binder film thickness calculation.

5.3 Main Factors for Statistical Analysis: Mixture Design

The primary question to be addressed in this study is: "What is the appropriate level of performance in the Asphalt Pavement Analyzer or other testing apparatus to accommodate Superpave Volumetric Mixture Design?"

Careful consideration of variables need to included in the design of experiment. Factors that need to be considered as factors include:

- The effect of trafficking level (18-kip Equivalent Single Axle Loads), and
- The effect of aggregate size (nominal maximum).

This establishes two main factors to be considered in development of the experimental plan. It is believed that these main factors should be considered in the selection of projects to minimize the bias that may occur if not otherwise examined. Table 5.1 outlines the design of experiment for the main factors. The design outlined is a partial factorial, but allows for inference to be made about the statistical cells that will not have any materials tested. This approach allows for maximum statistical power and balances added costs of testing more material.

Table 5.1 Design of Experiment: Main Mixture Design Factors

		Trafficking Level		
		E3 Mix 1-3 BESALs (millions)	E10 Mix 3-10 BESALs (millions)	E30 & E50 Mixes >30 BESALs (millions)
Nominal Maximum Aggregate Size	19.0mm	XXX		
	12.5mm	XXX		XXX
	9.5mm	XXX	XXX	XXX

X denotes one mixture.

It is reasonable for multiple mixtures to be sampled from the same project. Thus, it is proposed that a total of 18 mixtures from 12 projects be studied.

5.4 Sensitivity of Performance Tests to Current Michigan Department of Transportation Pay Factor Items

Variations in traditional pay factor items should be identified by a performance test for implementation purposes. Traditional pay factor items, with respect to volumetric properties, in Michigan include asphalt binder content, air voids, voids in the mineral aggregate, crushed particle count, theoretical maximum density, and aggregate gradation. Since some of these pay factor items have inter-relationships, it is not necessary to include all as main factors in the experiment. However, the properties will be measured/calculated and included in the analyses. One example of inter-relationships is asphalt binder content and maximum theoretical specific gravity. As the asphalt binder content of a mixture is increased, a decrease in the maximum theoretical specific gravity will occur. In a similar fashion, as the asphalt binder content is increased, the compacted air voids will decrease provided the same compaction effort is applied. Thus, it is necessary to capture the main factors in the design of experiment and identify the other factors which are statistically labeled as interactions. Table 5.2 outlines a design of experiment for the mixtures that will be studied and the main effects. Table 5.3 outlines the full factorial experiment for the 19.0mm mixture. It is proposed that the 12.5mm and 9.5mm mixtures be tested using all of the performance testing equipment outlined for the 19.0mm mixture in Table 5.3 with asphalt binder content as the only pay factor variable.

Table 5.2 Design of Experiment: A Sensitivity Analyses of Pay Factor Items-9.5mm, 12.5mm, and 19.0mm mixtures

		Main Effects		
		Asphalt Binder Content	Air Voids	Gradation
Mixture Size	19.0 mm	TTT LLL UUU	TTT LLL UUU	TTT LLL UUU
	12.5 mm	TTT LLL UUU		
	9.5 mm	TTT LLL UUU		

T denotes one sample to be tested at target value.
L denotes one specimen to be tested at the low value of the specification.
U denotes one specimen to be tested at the upper value of the specification.

Table 5.3 Design of Experiment: A Sensitivity Analyses of Pay Factor Items-19.0mm Mixture

Distress	Performance Test	Main Effects: Asphalt Binder Content	Air Voids	Gradation (75um sieve)
Permanent Deformation	Asphalt Pavement Analyzer	TTT LLL UUU	TTT LLL UUU	TTT LLL UUU
	Simple Shear Testing	TTT LLL UUU	TTT LLL UUU	TTT LLL UUU
Fatigue Cracking	Indirect Tensile Test (MDOT)	TTT LLL UUU	TTT LLL UUU	TTT LLL UUU
	Beam Fatigue	TTT LLL UUU	TTT LLL UUU	TTT LLL UUU
Thermal Cracking	Indirect Tensile Test (Roque)	TTT LLL UUU	TTT LLL UUU	TTT LLL UUU

T denotes one sample to be tested at target value.
L denotes one specimen to be tested at the low value of the specification.
U denotes one specimen to be tested at the upper value of the specification.

For each main effect in Table 5.3, the target value as well as the upper and lower specification will be tested. An example for air voids would be if the target is 4.0%, then the corresponding lower and upper specification values would be 3.5% and 4.5%, respectively. The interaction terms that will be included in the analyses are voids in mineral aggregate, theoretical maximum specific gravity, and crushed particle count. At this time, the 75um sieve will be the only gradation variable studied because it is felt that it is a more critical gradation factor than the 4.75mm and 600um sieves. However, if the statistical analyses of the gradation data in section 3.2.1 demonstrate more variability on other sieves and have a potential to affect performance, then additional gradation variables will be tested and studied. Thus, 135-19.0mm samples, 45-12.5mm samples, and 45-9.5mm samples will be tested.

5.5 Main Factors for Statistical Analysis: Sampling

The issue of pavement performance can be most effectively be addressed in a laboratory setting using tests that are known to yield metrics related to actual field performance of pavements. There are three different places in the design/construction process where either buyers or sellers would like to know the expected performance of the hot mix asphalt (HMA). These three locations are in design, during production, and assessment of payment.

A mix designer would like to establish the performance of mixtures prior to production to provide assurance that the mixture will perform as it is intended. Thus, the mixture would be lab mixed-lab compacted. During production, assessment of the HMA can occur from sampling trucks or from sampling behind the laydown machine, but prior to compaction. In this phase of construction, the samples would be field mixed-lab compacted. Testing of mixtures at this point in the production process acts as a quality control check. The final phase of testing is on mixtures that have been field mixed-field compacted, and acts as the quality assurance of the production process. Previous research has raised questions on the method of sample preparation and its effect on performance metrics. To address this issue, the following design of experiment is proposed in Table 5.4 for each of the 18 mixtures being studied.

Table 5.4 Design of Experiment: Accounting for Sampling Location

		Sampling Location			
Distress	Performance Test	Mixture Design	Truck, Loose Mix	Laydown, Loose Mix	Cores/Slabs
Permanent Deformation	Asphalt Pavement Analyzer	YYY	YYY	YYY	YYY
	Simple Shear Testing	YYY	YYY	YYY	YYY
Fatigue Cracking	Indirect Tensile Test (MDOT)	YYY	YYY	YYY	YYY
	Beam Fatigue	YYY	YYY	YYY	YYY
Thermal Cracking	Indirect Tensile Test (Roque)	YYY	YYY	YYY	YYY

Y denotes a sample to be tested.

The proposed experimental plan in Table 5.4 is for each of the 18 mixtures in Table 5.1. Thus, it is proposed that a total of 1080 samples will be subjected to performance testing. However, if the mixture characterization of the truck samples and the behind-the-paver samples show no difference in any of the four primary verification parameters (Voids in Mineral Aggregate at N_{design}, Theoretical Maximum Density, Air Voids at N_{design}, and Asphalt Binder Content), then there may be limited benefit to conducting performance testing on both of the loose mix sampling locations. Furthermore, if the MDOT quality assurance data and loose mix samples (truck and laydown) do not show significant differences, then the experimental plan could be further simplified by not testing field cores/slabs.

CHAPTER 6 HMA Specimen Preparation and Asphalt Pavement Analyzer Test Methods

6.1 Procurement of Specimens for APA Testing

The State of Michigan has established a need for a performance test to accompany the Superpave mixture design system for identifying rut-prone Hot-Mix Asphalt (HMA) mixtures before they are constructed. The performance test being considered is the Asphalt pavement Analyzer (APA). In this study, 10 different Michigan asphalt wearing courses constructed in the summer of 2000 have been tested in the APA to access the APA's ability to identify rut-prone mixes, the machine's sensitivity to HMA mixture properties linked to permanent deformation, and to explore the possibility of using the APA in a Performance Based Specification (PBS). To complete this study 210 Superpave Gyratory specimens were prepared and tested in the APA. The following chapter summarizes the methods used to prepare these specimens and the test methods and settings used in the APA testing.

6.2 APA Specimen Preparation

The literature has shown that the APA can test either beam or cylindrical specimens. Because the Superpave mix design is based upon cylindrical specimens compacted in the Superpave Gryratory Compactor (SGC) it was decided that the SGC would be used to prepare the APA specimens. The SGC used for this study was manufactured by Pine Instruments and is shown in Figure 6.1. Chapter 3 describes the mixture verification, the process of creating a laboratory mixture that "replicates" the HMA mixture used in the 10 projects. This laboratory mixture design was used to prepare the SGC specimens for APA testing. The steps in creating the APA specimens are summarized are following sections.

Figure 6.1 A Pine Superpave Gyratory Compactor

The first step in processing APA specimens was batching the stockpile aggregates together to create the design aggregate gradation. Each stockpile aggregate was separated into separate size fractions. Particles larger then the No. 4 sieve (2.75 mm) were added separately while all material less then the No. 4 were added together. The final weight of the batched aggregate was 3600 grams, enough aggregate to prepare a 90-100 mm high specimen when mixed with asphalt binder and compacted in the SGC.

After batching, both the aggregate and asphalt binder were put into a convection oven to bring the material up to mixing temperature. Mixing and compacting temperatures are shown in Table 6.1. Upon reaching mixing temperature, the aggregate and liquid asphalt binder were combined. This was done in a Dayton rotary bucket mixer (Figure 6.2). The bucket mixer was used because it is believed that it mixes HMA very similarly to a HMA production plant with a drum mixer. The aggregate was weighed in a tared bucket and the amount of asphalt to be added was calculated using the following equation:

$$Asphalt \quad Mass = Aggregate \quad Weight / \left[(1 - (Asphalt \quad Content / 100) \right] \qquad (4.1)$$

where:

Asphalt Mass = Mass of Asphalt to Add to Heated Aggregate (g)
Aggregate Weight = Mass of Heated Aggregate (g)
Asphalt Content = (Mass Asphalt)/(Total Asphalt Mixture Mass)

This is the point in the process where different amounts of asphalt binder were added to the heated aggregate to prepare specimens with varying asphalt contents. Asphalt contents of all 210 APA specimens are presented in Appendix B. The HMA mixture was mixed until all of the aggregate were visibly well coated. After mixing, the HMA mixture was deposited back into its original pan. Great care was taken to scrape all of the asphalt mix constituents from the bucket into the curing pan. Also, a dummy HMA mixture was mixed before actually mixing APA specimens to properly "butter" the bucket.

Figure 6.2 A Dayton Bucket Mixer

After mixing, the material was allowed to cure at 135 °C for 1.5 hours. After 1.5 hours the convection oven was set at the compaction temperature for a ½ hour to allow the mixture to rise to compaction temperature. The compaction temperature specified on the JMF's were used for laboratory compaction also (Table 6.1). During the 2 -hour curing period the HMA mixtures were stirred once to expose different HMA mixture to the air in the convection oven.

Table 6.1 SGC Mixing and Compaction Temperatures

Project Location and Name	HMA Mixing Temperature (°C)	SGC Compaction Temperature (°C)
Auburn Hills, I-75	164	127
Clarkston, I-75	164	135
Saginaw, I-75	164	135
Lansing, US-43	164	157
Indian River, I-75	164	130
Grayling, US-27	164	130
Brooklyn, M-50	158	143
Monroe, US-24	158	143
Elk Rapids, US-31	158	135
Brimley, M-28	158	139

After curing, the HMA mixture was compacted in a SGC. First, the heated SGC mold with compaction papers was tared out on a floor scale. The HMA mixture was then added to the empty SGC mold. The air void content of each SGC specimen was controlled by calculating the proper height needed to achieve the desired air voids. The following equations were used to find the proper specimen height:

$$BSG_{@Specified\ Air\ Voids} = MTSG - MTSG[1-(Specified\ \ Air\ \ Void/100)] \tag{4.2}$$

$$Height\ of\ SGC\ Specimen = (Correction\ Factor\ \text{X}\ Aggregate\ Weight) / (BSG_{@Specified\ Air\ Voids}\ \text{X}\ 17.6715) \tag{4.3}$$

where:

$\text{BSG}_{\text{@Specified Air Voids}}$	=The Bulk Specific Gravity of a SGC Specimen with the Desired Air Voids (i.e. 4, 8, or 12%)
MTSG	= The Maximum Specific Gravity of the Asphalt Mixture Being Compacted
Specified Air Void	=The Air Void that the SGC Specimen Needs To Be Compacted To (%)
Correction Factor	=($\text{BSG}_{\text{@Specified Air Voids}}$)/(Volumetric Bulk Specific Gravity)
Aggregate Weight	= Mass of Aggregate in Tared SGC Mold (g)
Height of SGC Specimen	=Height to set SGC to Prepare SGC Specimen with Specified Air Voids (mm)

Once the height of the SGC specimen is calculated a heated SGC mold with the HMA mixture is inserted into a SGC. After insertion, the compaction height is entered into the SGC, and the SGC automatically compacts the specimen to the targeted height.

After compaction, the SGC specimen is extruded from the SGC and the specimen is allowed to cool to room temperature. After cooling, the bulk specific gravity of the specimen is determined using the saturated surface dry method (ASTM D2726). The calculated bulk specific gravity of the SGC specimen is then used to verify that the specimen is in fact compacted to the air voids specified. The tolerance for air voids used in this study is ±0.5%. For example, if the SGC specimen compacted was a 4% air void specimen then the actual air voids calculated by the following equation must be between 3.5% to 4.5% air voids to be included in the study. The air void equation is as follows:

$$SGC \quad Specimen \quad Air \quad Voids = (MTSG - BSG_{SSD}) / MTSG \qquad (4.4)$$

where:

SGC Specimen Air Voids = Actual Air Voids of the SGC specimen
MTSG = The Maximum Specific Gravity of the Asphalt Mixture Being Compacted
BSG_{SSD} = Bulk Specific Gravity as Calculated from ASTM D2726

If the SGC specimen air voids were not within ±0.5% of the target air voids, the specimen was discarded and a new specimen is made. APA specimen air void contents for all 210 specimens is shown in Appendix B.

Once the SGC specimens were prepared, they are trimmed to a height of 75 mm, the depth of the APA molds. Cutting of the specimens was done using a rock saw manufactured by Diamond International (Figure 6.3). Care was taken to cut the specimens so that the top and bottom of the specimens were parallel.

Figure 6.3 A Diamond International Rock Saw

6.3 Preliminary APA Test Method

One of the goals of this project was to provide Michigan Department of Transportation (MDOT) with an APA Test Specification for testing HMA mixtures in the Asphalt Pavement Analyzer. Based on the literature review machine settings were selected to produce results that would correlate well with actual field performance. The APA test settings and methods recommended to MDOT and used in this study are summarized in Table 6.2.

Table 6.2 Preliminary APA Machine Settings and Test Methods

Parameter	Specification
Test Temperature, (°C) [*1]	Upper Performance Grade of HMA Mixture Being Tested
Environmental Condition	Dry
Specimen Size, mm	Cylindrical Specimens with 150 mm diameter and 75 mm height
Load, N (lb)	445 (100)
Hose Pressure, kPa (psi)	689 (100)
Wheel Speed, m/sec	0.61
Number of Test Wheel Load Cycles	8000
Laboratory Compaction Device	Superpave Gyratory Compactor
Pretest Specimen Conditioning	4 hours @ Test Temperature
Number of Seating Cycles	50 Cycles

*1 This does not include grade bumping for high volume facilities or slower moving traffic.

The SGC specimens were placed into the APA molds. To keep testing uniform for all specimens it was decided that the top of all SGC specimens should be flush with the top of the APA mold. To do this, Plaster of Paris was placed beneath the specimens so the specimens were flush with the top of the mold. In earlier APA testing at Michigan Tech, it was noticed that if SGC specimens did not fit snugly in the APA molds (i.e. there were gaps present between the SGC specimen and the mold) and that the specimens tend to spread out beneath the APA loading. This would artificially inflate the rut depth measurement. To keep this from happening, Plaster of Paris was poured into the gaps and allowed to harden so that the specimens were completely restrained within the mold. Figure 6.4 shows APA specimens with plaster of paris applied to the sides of the specimens.

Figure 6.4 APA Specimens with Plaster of Paris

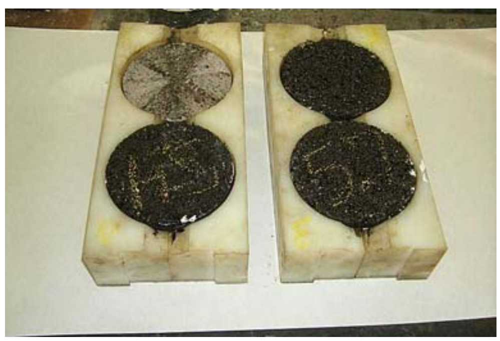


Figure 6.5 APA Molds With and Without a Concrete Spacer

After preparing the test specimens the APA molds with the specimens were conditioned at the test temperature for four hours to allow the specimens to come to test temperature. The molds were either placed into the lower compartment of the APA or in a laboratory oven during this conditioning cycle. After conditioning APA testing was commenced.

Normally each APA mold contains two specimens for testing. The average rut depth of both specimens is then recorded as the APA rut depth. In this study the standard deviation of the three specimens at each asphalt content/air void level was of great importance. The APA does not record each specimen rut depth independently but rather records the average of the two specimens in each APA mold. Thus, another method had to be used so that the rut depth of each individual specimen was recorded and the standard deviation could be calculated. To do this a concrete spacer was placed into one of the specimen holes and only one asphalt specimen was tested in each mold during APA testing (Figure 6.5). The LVDTs measuring the rut depth of the concrete plug were shut off so that only the two LVDTs measuring the rut depth of the asphalt specimen were recorded.

Testing was commenced using the machine settings shown in Table 6.2. The resulting data was recorded in the personal computer accompanying the APA. This data was put into Microsoft Excel format and loaded onto Zip discs. The data was then loaded onto the Civil and Environmental Engineering computer network for analysis using the SAS statistical package.

6.4 Procurement of Specimens for Fatigue Testing

In the summer of 2002 materials were batched from the aforementioned MDOT projects and prepared in order to make beam fatigue specimens. An experimental plan was developed for each job listed in Table 6.1. Specimens were prepared at 4, 8 and 12% air void contents for each job, while the optimum asphalt content from the work done by Hofmann in 2002 was used. Table 6-3 below shows the number of specimens that were tested at the air void contents for each individual job. The following sections describe the proportioning, mixing and compacting procedures for the HMA fatigue samples.

Table 6-3: Targeted Air Void and Asphalt Content Levels for all MDOT Paving Jobs

		Asphalt Content, %		
		< Target	Target	> Target
Air Voids, %	4	Not Tested	XXXXXX	
	8		XXXXXX	
	12		XXXXXX	Not Tested

X = one beam fatigue sample

6.4.1 Batching of HMA Materials

The first step in the sample preparation was to batch out the required amount of aggregate materials for one slab, from which three beams were cut. The job mix formulas (JMF) that were used to batch samples were taken from those verified by Hofmann in 2002. The batch samples were approximately 10,000g each. Appendix H shows the aggregate and asphalt mix batch weights for each job. The target air void levels were 4, 8, and 12%. Two batches were prepared at each air void level for each job. The batching process started with the measurement of the linear kneading compactor volume mold, taking into consideration the final target height of the sample (50mm). The mold dimensions and volume were: (380mm long x 203mm wide x 50mm high) / (1000^3) = 3.86×10-3m^3. The bulk specific gravity of the sample was then calculated using the target air voids and the maximum theoretical specific gravity of the asphalt mix, which was taken from previous research (Hofmann 2002). The mold volume was then multiplied by the bulk specific gravity of the aggregate and by 1,000,000 (to convert units to mm) and then divided by a correction factor (1.03) to estimate the target mix weight for a particular air voids level. Even though careful calculations had been made, adjustments to the batching process were necessary in order to obtain the target air void levels. The calculated mix weight for 4% air voids was found to actually produce an actual adjusted mix weight for the targeted 8% air voids samples. The 12% air voids batches did not need adjustment. The 4% air voids sample weights were calculated by subtracting the adjusted mix weight at 8% air voids from the adjusted mix weight at 12% air voids and adding the calculated mix weight at 4% air voids. See Appendix H for an example of the entire batch calculations process.

6.4.2 Mixing and Compacting of HMA Materials

First, the aggregate and asphalt materials were brought up to the appropriate mixing temperature. The mixing and compacting temperatures were calculated by using the method as described in the Asphalt Institute (2001) in conjunction with the corresponding mixing and compacting viscosities at 165 and 135^{o}C, respectively, which were found by Hofmann in 2002. The designated Superpave values are 0.17±0.02 Pa•s for mixing and 0.28±0.03 Pa•s for compacting. Then, liquid asphalt was added to the aggregate and the sample was thoroughly mixed to ensure uniform coating of the aggregate with the binder. Each sample was then adjusted to the appropriate weights to target the air voids levels. The 8% and 12% air void samples were prepared first by simply removing some material. The excess material was placed into a spare pan. The 4% air voids samples were then prepared. All six samples were then aged for 2-4 hours in an oven at the calculated compacting temperature. This was done to simulate aging that occurs during construction. The type of linear kneading compactor used for this research was a HasDek Slab-Pac™, which is shown in Figure 6.6. While the samples were aging, the molds and any necessary equipment needed for compaction were heated and brought to compaction temperature as well. The samples were then compacted. After compaction they were allowed to cool down to room temperature.

Figure 6.6: HasDek Slab-Pac© Linear Kneading Compactor (Shown with molds)

The samples' bulk specific gravities were then measured according to ASTM D 2726 (Standard Test Method for Bulk Specific Gravity and Density of Non-Absorptive Compacted Bituminous Mixtures). Finally, each samples' air voids were calculated using the maximum theoretical and bulk specific gravities. Each sample was then cut into three beams. The beam dimensions were 50mm high x 63mm wide by 385mm long, all being +/- 5mm. The following pictures illustrate what the compacted and cut samples looked like as shown in Figure 6.7. Any pertinent data for the beams such as gradation and the specimen batching tables can be found in Appendix H.

Figure 6.7: Compacted (M-35 Escanaba) and Cut (I-75 Grayling) HMA Samples

6.5 Specimen Testing in the Beam Fatigue Apparatus

Each asphalt specimen was tested in the beam fatigue apparatus (Figure 6.8) at a different microstrain level. The intent was to produce a range of termination cycles for each job to see how a mix's performance varied with microstrain at different air void contents and then later on to quantify certain mix variables through regression analysis. All tests were run in accordance with AASHTO TP8, Standard Test Method for Determining the Fatigue Life of Compacted Hot Mix Asphalt Subjected to Flexural Bending. The flexural bending machine used is called the UTM 21© and was made by Industrial Process Controls (IPC) in Melbourne, Australia.

All tests were run in the constant strain mode; so that the maximum deflection of the beam was held constant with each successive load cycle, while the load placed on the specimen gradually decreased. The constant maximum strain level was monitored with two linear variable differential transducers (LVDTs) that measure the deflection of the beam as well as the change in length between the inner and outer gauge lengths. The beam was flexed for a number of cycles. The termination cycle was defined as the cycle at which 50% of the initial stiffness was achieved.

Figure 6.8: IPC UTM 21© Beam Fatigue Apparatus

The results were then plotted to get a preliminary idea of how each type of mix reacted to the test with regard to air void and microstrain level. At this stage other variables such as initial stiffness and initial modulus were also included as potential predictor variables for statistical modeling. A summary of the beam fatigue test data is presented in Appendix I.

CHAPTER 7 RESULTS

7.1 Hot Mix Asphalt and Aggregate Characterization of Truck and Paver Samples

The characterization of hot mix asphalt (HMA) and all of its constituents were done on truck and paver samples. The constituents of the HMA characterized were aggregate and asphalt binder. The aggregate and HMA characterization is summarized in this chapter, while characterization of asphalt binders was summarized previously in this report.

7.2 Experimental Plan

A review of the experimental plan for field sampling is provided to demonstrate the experimental approach and the factors considered in the research plan. The project locations were randomly picked from throughout the State of Michigan. Careful coordination between the Bituminous Traveling Mixture Inspectors, the Construction and Technology Office, contractors, and consultants and the research team was maintained throughout the field sampling portion of the project. Table 7.1 details the project locations where samples were obtained, satisfying the experimental plan.

7.3 Method of Sampling

The method of obtaining truck and paver samples was previously described previously in this report, but a brief review of the sampling procedures is described in sections 7.3.1 and 7.3.2 for truck and behind the paver sampling, respectively.

7.3.1 Truck Sampling of Mixtures

In general, eight trucks were sampled in a single day’s production at the HMA plant after being loaded. Five-5gallon buckets of material were sampled from each truck, resulting in a total of 40-5gallon buckets of HMA. However, there were instances where this amount of sampling (number of trucks) could not be done because of impending weather. Nonetheless, an equal amount of material was sampled from each project by increasing the number of buckets sampled from the remaining trucks.

7.3.2 Behind the Paver Sampling of Mixtures

Sampling from behind the paver occurred on 19 of the 20 projects. The Old M-14 project did not have any behind the paver samples because weather closed the project down. In lieu of behind the paver sampling, a stockpile at the plant was provided after the weather passed. On the remaining projects, five behind the paver locations were sampled on 15 projects with plates. The differences for sampling on the four projects were as follows:

The I-75 (Flint) and I-94 (Ann Arbor, SMA) projects utilized material transfer devices (windrows) and it was felt that there was a possibility of the plates being picked up and could cause significant damage to the laydown equipment. This resulted in the mats being sampled with pan shovels diagonal to the paving lane. Due to safety issues on the two remaining I-94 projects in Ann Arbor, a 3E30 and a 4E30 mix were of concern as the samples had to be manually moved to exit ramps which resulted in three rather than five sampling locations. In summary, 15 of the 20 projects were sampled behind the paver in the same manner using plates.

7.3.3 Characterization of Hot Mix Asphalt Samples

Tables 7.2 through 7.21 summarize the characteristics of the truck and paver samples from all of the projects and also provide a summary of the project's mix design characteristics. The characterization of the mixture and aggregate properties was done in accordance with AASHTO test specifications except for the crushed particle count for coarse aggregate as noted. The test specifications followed were:

- Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens (Method A): AASHTO T 166-00,
- Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures: AASHTO T 209-99,
- Quantitative Extraction of Bitumen from Bituminous Paving Mixtures (Method A): AASHTO T 164-97,
- Recovery of Asphalt from Solution by Abson Method: AASHTO T 170-00,
- Mechanical Analysis of Extracted Aggregate: AASHTO T 30-93,
- Uncompacted Void Content of Fine Aggregate: AASHTO T 304-96, and
- Determining the Percentage of Crushed Fragments in Gravels: Pennsylvania DOT Test Method 621.

The volumetric calculations of voids in the mineral aggregate (VMA) and effective binder contents (P_{be}) used the aggregate specific gravity design values. The assumed aggregate specific gravity values also are then translated into the calculation of voids filled with asphalt (VFA) as this calculation is dependent upon VMA.

7.4 Comparison of Field Samples Characteristics to Design Values

Comparison of aggregate and mixture characteristics from the two field sampling locations are made as well as comparison of field to design characteristics. Specifically, aggregate and HMA characteristics of truck and paver samples are compared to determine whether or not there is a relationship between the two. The two sampling locations' characteristics are also compared to design characteristics and examined with the specification tolerances. The comparisons are made in the following three sections: asphalt binder content comparisons, aggregate characteristic comparisons, and HMA volumetric comparisons. The reason for differentiating the asphalt binder content from the HMA volumetric comparisons is based on the binder content being a measurement, whereas the other volumetric properties are calculations based on other measurements. Furthermore the discussion follows the steps in which the materials were processed, e.g., binder content determination, gradation and aggregate characteristic measurements, and volumetric calculations.

7.4.1 Asphalt Binder Content Comparisons

Binder content comparisons are made in Figure 7.1 between average truck and paver sampling locations for each project. The unity line for the two sampling locations is shown on the figure. The correlation between the two sampling locations is 0.86, which is very good. This high correlation would be expected and indicated that the handling of the mixtures between the load out at the plant and the placement is in general very good across the State of Michigan. The dashed line in Figure 7.1 is the best fit line for the data and is shown to have a slope less than the unity line. The slope being less than the unity line indicates that asphalt binder is disappearing during the haul and placement time or potentially a combination of additional aging and more asphalt absorption is occurring. If more aging is occurring, this could indicate a loss of the lighter elements in the asphalt being lost during the aging, resulting in a loss in binder content during the haul and placement time. A second possibility could be the added absorption of the asphalt into the aggregate, yet the recovery process is not as effective as that of the absorbed asphalt binder.

Figure 7.2 compares the truck and paver binder contents to the job mix formula (JMF) binder contents. Logically, the trend for both the truck and paver binder contents are closer to the lower tolerance band of the acceptance region. Significant savings over a project can be achieved by a contractor producing mixtures with lower binder contents, as the binder is the most expensive component of the bulk HMA. One important note on the tolerance band is that the construction specification of +/-5% is based on the running average of five test results and the results of this study are averages from four different stationing/sampling locations for each project. The correlations for the truck and paver sampling locations compared to the JMF are 0.76 and 0.67, respectively. The trend lines for the paver sampling location are more likely to be out of specification than the truck sampling location as evidence by the paver trend line being closer to the lower bound of the specification tolerance or acceptance region.

7.4.2 Aggregate Characteristic Comparisons

Graphical comparisons of the aggregate characteristics are made in Figures 7.3 through 7.12. Select sieve sizes (#4, #8, #30, and #200) are reviewed as these are the sieves used in MDOT's specifications for verification of mixture designs. Figure 7.3 demonstrates that there is a very strong correlation, 0.98, between the percent passing the #4 sieve for the truck and paver sampling locations. Comparison of the percent passing the #4 sieve to the JMF and the design acceptance region in Figure 7.4 shows that all of the standard mixes were within the design specification tolerance of +/-4% for construction. The one nonstandard mix, denoted as an SMA (stone matrix asphalt), was not within the running average tolerance of +/-5%.

Similar to the #4 sieve size, the correlation between the truck and paver sampling locations for the percent passing the #8 is 0.98 as shown in Figure 7.5.

Figure 7.6 demonstrates that all of the standard mixes are within the design specification tolerances of +/-4%, which is excellent. The SMA is outside the design tolerance similar to the #4 sieve.

Figure 7.7 demonstrates a strong correlation, 0.99, between the truck and paver sampling locations for the percent passing the #30 sieve.

Figure 7.8 demonstrates that nearly all of the mixes are within the design specification tolerance of +/-3%, regardless of the sampling location. The one exception is the SMA mix on I-94 is within the design tolerance for the paver sampling location and just outside of the design tolerance for the truck sampling location. The last sieve size used for design specification is the #200 sieve.

Figure 7.9 shows a strong correlation of 0.93 between the truck and paver sampling locations for the percent passing the #200 sieve. In general the paver sampling locations tend to have more fines, material passing the #200 sieve, than the truck sampling location. This would be expected because additional handling of the mixture occurs with the paver samples than truck samples and thus paver samples are more likely to experience more breakdowns from the additional handling. Comparisons of the percent passing the #200 sieve between truck and paver sampling locations to the JMF illustrates there are two projects that are clearly outside of the acceptance region of the running average of five test results, +/-0.6% from the JMF, for both the truck and paver sampling locations. A third project has the percent passing the #200 for the truck sampling location within the acceptance region while the paver sampling location is outside of the acceptance location. Again, most of the projects have an average of four test results. Only the I-75 project in Indian River is well outside the specification tolerance.

The last aggregate quality compared to the design limits is the fine aggregate angularity in

Figure 7.12. Overall the comparison between the truck and paver sampling locations had a reasonable correlation of 0.67. It is important to point out that no bias has been established for this test by AASHTO, and thus the variability for the test results and associated tolerances are not known. Based on the design specifications, the E10 and E30 mixes should have a fine aggregate angularity (FAA) of 45% and the E3 mixes should have an FAA of 40%. The Michigan Department of Transportation uses the FAA specification values as a minimum, which applies to both design and construction. Figure 7.12 shows the two levels of FAA for design and demonstrates the 45% FAA for the E10 and E30 mixes are not being achieved for most projects (six out of nine) for both the truck and paver sampling locations. The production and placement process would tend to change the aggregate characteristics and could lead to different values of FAA than design values. The FAA design value of 40% of for the E3 mixes were met for seven of the nine mixes demonstrating that the design specification is being largely met by recovered field samples. The two projects that did not achieve an FAA of 40% for both truck and paver sampling locations, one met the criteria based on the truck sampling location and did not meet the criteria for the paver sampling location, while vice versa was the case for the other project. Nonetheless both projects were very close to meeting the criteria as the non-conforming test values are 39.5% or greater.

7.4.3 HMA Volumetric Comparisons

(TMD and Gmb; verification of TMD and Gmb; research projects compacted to Ndesign rather than in MDOT's QC/QA process)

Figures 7.13 through 7.21 show the comparison of truck and paver sampling location volumetric properties to the JMF design values. The Michigan Department of Transportation uses the following volumetric criteria for design:

- Fines to effective binder content ratio,
- Percent air voids at N_{design},
- Voids in the Mineral Aggregate (VMA),
- Voids Filled with Asphalt (VFA).

However, the Michigan Department of Transportation uses only the percent air voids of the compacted mat and the voids in the mineral aggregate as specification criteria for payment.

Figure 7.13 demonstrates a strong correlation of 0.92 between the truck and paver sampling location for the fines to effective binder content (F/P_{be}). However when the paver and truck sampling locations for the F/P_{be} are compared to the design values in

Figure 7.14, there are not good correlations (0.50 for truck; 0.43 for paver). Comparisons of percent air voids at N_{design} for the truck and paver sampling locations show a rather poor relationship between the two as the correlation is 0.59 in Figure 7.15. The specification tolerance of +/-0.5%, which defines the acceptance region, is not being met by most of the mixes. The majority of the mixes outside of the acceptance region tend to have lower air voids. One of the issues associated with the lower air voids at N_{design} is that during the design phase of mixtures the air voids at N_{design} are back-calculated based on the ratio of the specimen heights at N_{design} and N_{max} and the corresponding bulk specific gravity of the samples at N_{max}. The method of back-calculating the percent air voids at N_{design} assumes a linear relationship, which is in fact not true for most mixes. This non-linear relationship is illustrated in Figure 7.16.

The VMA data is reviewed in Figures 7.17 through 7.19. Figure 7.17 shows a very good correlation of 0.91 for VMA between the truck and paver sampling locations. However, the figure does also illustrate a drop of 0.3% in the VMA from the truck to the paver sampling location and is consistent whether a mix has a high or low VMA as the best fit curve is parallel to the solid unity line. In general most mixes, 11 of the 17, are within the specification tolerances or acceptance region. Review of Figure 7.18 shows that there are four projects that more than 0.5% below the minimum VMA for the respective nominal maximum aggregate size and are indicated by an oval around the mixes. Finally for the VMA comparisons, Figure 7.19 shows the comparisons between the truck and paver sampling locations to the JMF design values. The correlations, 0.64 for the truck and 0.65 for the paver sampling locations, are reasonable considering the aggregate specific gravity values used in the calculation are the assumed design values. Again the unity line in Figure 7.19 represent the design values, and the offset of the truck and paver locations moving further away from the target design values illustrates the breakdown of the aggregate on average leading to lower VMA as the mixes are handled more.

The last volumetric characteristic calculated and compared is the voids filled with asphalt (VFA). The comparison between truck and paver sampling locations for VFA is shown in Figure 7.20. A correlation of 0.58 is shown and is rather poor. Comparison of the truck and paver sampling location values of VFA to the JMF values clearly demonstrates the field characteristic has no relationship to the design values as the correlation are nearly 0.

Table 7.1 Experimental Plan and Project Locations

Nominal Maximum Aggregate Size	Trafficking Level		
	E3 Mix 1-3 BESALs (millions)	E10 Mix 3-10 BESALs (millions)	E30 & E50 Mixes >30 BESALs (millions)
19.0mm	M-32, Lachine M-45, Grand Rapids Old M-14, Plymouth		I-94, Ann Arbor[1]
12.5mm	M-35, Escanaba M-52, St. Charles M-90, Lexington		I-94, Ann Arbor I-94, Ann Arbor (SMA) Eight Mile Road, Warren
9.5mm	M-50, Brooklyn US-31, Elk Rapids US-24, Monroe	I-75, Indian River I-75, Grayling Saginaw St., Lansing	I-75, Auburn Hills I-75, Flint I-75, Saginaw

[1]This project/mix was sampled prior to a change in the experimental plan.

Table 7.2 Aggregate and Hot Mix Asphalt Characteristics for M-32 (Lachine)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	99.5
12.5	89.4	91.7	86.9
9.5	81.5	80.9	73.4
4.75	47.4	48.2	44.7
2.36	31.6	28.4	27.7
1.18	19.3	17.9	18.1
0.600	13.0	12.0	12.4
0.300	7.7	8.4	8.8
0.150	5.5	6.0	6.2
0.075	4.5	4.7	4.8
Mixture Characteristics			
Binder Content, %	5.1	5.0	4.9
%Gmm at N_{design}	96.0	96.7	97.1
VMA, %	13.7	13.2	12.8
VFA, %	70.8	75.2	77.5
F/P_{be}	1.07	1.15	1.21
Fine Aggregate Angularity	45.9	43.9	44.1
Percent Crush Count, 1 Face	98.4	98.4	97.5
Number of Samples	---	4	4

Table 7.3Aggregate and Hot Mix Asphalt Characteristics for M-45 (Grand Rapids)

	Design	**Truck**	**Paver**
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	98.3	99.5	100.0
12.5	89.6	88.7	85.8
9.5	85.7	87.3	83.9
4.75	74.0	76.1	72.5
2.36	48.5	50.1	48.6
1.18	35.7	35.6	35.2
0.600	27.6	27.4	27.4
0.300	16.0	17.3	17.3
0.150	6.1	6.9	6.9
0.075	4.1	3.7	3.7
Mixture Characteristics			
Binder Content	5.1	4.9	4.6
%Gmm at N_{design}	96.0	96.3	96.1
VMA, %	13.7	13.1	12.8
VFA, %	69.7	72.1	69.4
F/P_{be}	1.01	0.97	1.05
Fine Aggregate Angularity	42.2	40.4	39.6
Percent Crush Count, 1 Face	92.4	82.1	85.0
Number of Samples	---	4	4

Table 7.4 Aggregate and Hot Mix Asphalt Characteristics for Old M-14 (Plymouth)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	---
19.0	99.7	99.6	---
12.5	87.5	81.8	---
9.5	75.4	68.4	---
4.75	37.6	36.0	---
2.36	26.1	23.7	---
1.18	19.1	17.6	---
0.600	14.4	13.7	---
0.300	10.2	10.0	---
0.150	6.3	6.3	---
0.075	4.0	4.5	---
Mixture Characteristics			
Binder Content	5.1	4.5	---
%Gmm at N_{design}	96.0	95.2	---
VMA, %	13.7	13.3	---
VFA, %	70.8	64.1	---
F/P_{be}	1.00	1.28	---
Fine Aggregate Angularity	46.2	43.1	---
Percent Crush Count, 1 Face	96.1	98.6	---
Number of Sample	---	2	—

Table 7.5 Aggregate and Hot Mix Asphalt Characteristics for M-35 (Escanaba)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	96.1	98.3	98.6
9.5	86.0	91.4	93.0
4.75	67.3	70.4	72.5
2.36	55.1	57.7	59.6
1.18	44.1	46.1	48.0
0.600	35.1	34.3	35.6
0.300	21.6	18.6	19.3
0.150	6.9	8.1	8.3
0.075	5.4	5.6	5.4
Mixture Characteristics			
Binder Content	5.6	5.1	5.6
%Gmm at N_{design}	96.0	96.4	96.4
VMA, %	14.7	14.2	14.4
VFA, %	73.4	75.0	74.9
F/P_{be}	1.18	1.22	1.16
Fine Aggregate Angularity	44.5	42.2	41.4
Percent Crush Count, 1 Face	88.1	85.5	83.1
Number of Samples	---	4	4

Table 7.6 Aggregate and Hot Mix Asphalt Characteristics for M-52 (St. Charles)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	94.0	91.8	90.8
9.5	88.6	87.7	86.6
4.75	76.3	73.8	72.6
2.36	54.5	56.8	55.8
1.18	40.0	42.7	42.0
0.600	28.9	30.9	30.5
0.300	17.3	18.9	18.7
0.150	8.4	9.1	9.1
0.075	5.3	5.8	5.8
Mixture Characteristics			
Binder Content	5.5	5.5	5.2
%Gmm at N_{design}	96.0	95.3	95.8
VMA, %	14.4	15.6	14.7
VFA, %	71.9	70.1	71.8
F/P_{be}	1.11	1.23	1.29
Fine Aggregate Angularity	43.7	42.4	43.4
Percent Crush Count, 1 Face	95.6	97.3	98.0
Number of Samples	---	4	4

Table 7.7 Aggregate and Hot Mix Asphalt Characteristics for M-90 (Lexington)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	99.1	95.0	95.1
9.5	89.6	88.6	88.1
4.75	74.9	77.1	76.9
2.36	56.2	55.7	55.8
1.18	38.6	39.4	40.2
0.600	26.8	28.1	29.1
0.300	16.5	17.3	17.8
0.150	8.7	9.0	9.2
0.075	5.6	5.3	5.7
Mixture Characteristics			
Binder Content	6.0	6.2	6.2
%Gmm at N_{design}	96.0	96.7	97.1
VMA, %	16.0	15.9	15.7
VFA, %	75.0	79.0	81.4
F/P_{be}	1.10	0.99	1.05
Fine Aggregate Angularity	48.1	44.4	46.0
Percent Crush Count, 1 Face	96.5	99.8	100.0
Number of Samples	---	4	4

Table 7.8 Aggregate and Hot Mix Asphalt Characteristics for M-50 (Brooklyn)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	99.4
9.5	99.6	99.7	97.9
4.75	89.0	82.0	78.1
2.36	67.0	59.1	55.8
1.18	43.5	39.9	37.9
0.600	29.0	27.3	26.0
0.300	18.8	17.9	17.0
0.150	9.7	10.7	10.3
0.075	5.9	6.5	6.4
Mixture Characteristics			
Binder Content	6.8	6.1	6.0
%Gmm at N_{design}	96.0	95.0	96.6
VMA, %	16.6	15.8	15.0
VFA, %	75.8	68.6	77.3
F/P_{be}	1.08	1.37	1.39
Fine Aggregate Angularity	46.2	45.1	44.1
Percent Crush Count, 1 Face	87.0	92.0	90.6
Number of Samples	---	4	4

Table 7.9 Aggregate and Hot Mix Asphalt Characteristics for US-31 (Elk Rapids)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	99.9
9.5	98.0	98.4	97.8
4.75	66.1	65.0	66.1
2.36	35.0	36.2	37.3
1.18	22.2	22.5	23.0
0.600	18.1	18.3	18.8
0.300	9.5	10.4	10.9
0.150	5.3	5.9	6.2
0.075	4.5	4.9	5.2
Mixture Characteristics			
Binder Content	6.5	6.1	5.8
%Gmm at N_{design}	96.0	97.0	97.0
VMA, %	16.0	14.7	14.1
VFA, %	75.4	79.7	79.0
F/P_{be}	0.85	0.98	1.11
Fine Aggregate Angularity	40.7	39.8	40.3
Percent Crush Count, 1 Face	99.0	99.2	98.6
Number of Samples	---	4	4

Table 7.10 Aggregate and Hot Mix Asphalt Characteristics for US-24 (Monroe)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	99.6
12.5	100.0	98.9	98.2
9.5	97.0	93.9	93.4
4.75	61.1	63.1	59.9
2.36	36.0	35.9	33.5
1.18	23.4	22.0	20.3
0.600	15.4	14.0	12.6
0.300	9.1	8.6	8.4
0.150	6.1	5.5	6.2
0.075	4.8	4.5	5.2
Mixture Characteristics			
Binder Content	6.2	6.2	6.2
%Gmm at N_{design}	96.0	95.4	95.2
VMA, %	15.2	15.3	15.6
VFA, %	73.8	69.6	69.4
F/P_{be}	1.01	0.95	1.08
Fine Aggregate Angularity	43.3	42.0	42.6
Percent Crush Count, 1 Face	99.7	99.9	99.7
Number of Samples	---	4	4

Table 7.11 Aggregate and Hot Mix Asphalt Characteristics for I-75 (Indian River)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	99.9
9.5	97.2	97.0	95.5
4.75	67.6	68.0	63.7
2.36	38.0	38.5	37.7
1.18	23.9	25.2	25.6
0.600	17.1	17.8	18.3
0.300	9.8	11.2	11.6
0.150	5.6	7.1	7.4
0.075	4.0	5.6	5.8
Mixture Characteristics			
Binder Content	6.3	5.7	5.7
%Gmm at N_{design}	96.0	96.9	97.5
VMA, %	15.4	14.1	13.4
VFA, %	74.0	77.8	80.9
F/P_{be}	1.00	1.25	1.32
Fine Aggregate Angularity	46.0	41.7	41.7
Percent Crush Count, 1 Face	99.0	98.8	98.9
Number of Samples	---	4	2

Table 7.12 Aggregate and Hot Mix Asphalt Characteristics for US-27 (Grayling)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	99.8
9.5	97.0	97.5	96.3
4.75	65.5	66.6	62.5
2.36	43.5	45.5	42.6
1.18	29.0	30.5	29.5
0.600	20.0	19.9	19.8
0.300	9.5	10.9	11.2
0.150	6.4	5.6	6.1
0.075	3.7	4.1	4.6
Mixture Characteristics			
Binder Content	6.3	5.9	5.7
%Gmm at N_{design}	96.0	96.8	97.1
VMA, %	15.4	14.3	13.6
VFA, %	74.9	77.4	78.9
F/P_{be}	1.00	0.88	1.04
Fine Aggregate Angularity	46.3	43.6	42.7
Percent Crush Count, 1 Face	99.3	99.1	99.1
Number of Samples	---	4	4

Table 7.13 Aggregate and Hot Mix Asphalt Characteristics for M-43 (Lansing)

	Design	**Truck**	**Paver**
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	99.9	99.6
9.5	99.6	99.9	99.1
4.75	82.2	78.8	79.9
2.36	58.9	56.1	58.0
1.18	42.9	40.6	42.0
0.600	32.3	29.6	30.7
0.300	19.4	17.1	17.5
0.150	9.7	7.5	7.6
0.075	5.0	4.1	4.2
Mixture Characteristics			
Binder Content	6.0	5.4	5.5
%Gmm at N_{design}	96.0	95.9	96.3
VMA, %	15.6	14.6	14.4
VFA, %	74.3	72.2	74.3
F/P_{be}	0.99	0.92	0.93
Fine Aggregate Angularity	45.0	42.3	42.4
Percent Crush Count, 1 Face	88.2	86.5	85.7
Number of Samples	---	4	4

Table 7.14 Aggregate and Hot Mix Asphalt Characteristics for I-75 (Auburn Hills)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	100.0	99.9
9.5	99.0	98.8	98.6
4.75	71.0	71.5	70.8
2.36	37.1	39.1	39.0
1.18	21.6	23.1	23.9
0.600	14.4	15.6	15.8
0.300	10.1	10.8	11.4
0.150	7.0	7.3	7.8
0.075	5.6	5.7	5.7
Mixture Characteristics			
Binder Content	6.2	5.7	5.7
%Gmm at N_{design}	96.0	95.4	95.6
VMA, %	16.0	15.8	15.4
VFA, %	75.0	70.8	71.1
F/P_{be}	1.10	1.20	1.30
Fine Aggregate Angularity	45.3	43.5	43.8
Percent Crush Count, 1 Face	99.0	98.2	98.5
Number of Samples	---	4	4

Table 7.15 Aggregate and Hot Mix Asphalt Characteristics for I-75 (Flint)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	98.9	98.7
9.5	97.5	96.0	95.2
4.75	70.6	70.8	68.7
2.36	42.6	43.5	42.1
1.18	27.3	27.6	26.9
0.600	18.1	18.1	17.9
0.300	12.7	12.1	12.2
0.150	8.2	7.7	7.8
0.075	5.3	5.1	5.3
Mixture Characteristics			
Binder Content	6.0	5.5	5.4
%Gmm at N_{design}	96.0	95.8	95.8
VMA, %	15.7	15.1	14.8
VFA, %	74.7	72.3	71.9
F/P_{be}	1.10	1.16	1.22
Fine Aggregate Angularity	48.2	44.0	43.5
Percent Crush Count, 1 Face	99.9	98.4	98.6
Number of Samples	---	4	4

Table 7.16 Aggregate and Hot Mix Asphalt Characteristics for I-75 (Saginaw)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	100.0	98.1	97.2
9.5	93.7	94.5	92.2
4.75	83.2	84.7	80.9
2.36	61.9	62.3	57.7
1.18	41.6	44.8	41.4
0.600	30.3	32.3	30.3
0.300	19.2	20.2	19.3
0.150	8.8	8.6	8.7
0.075	5.7	5.3	5.6
Mixture Characteristics			
Binder Content	5.8	5.9	5.7
%Gmm at N_{design}	96.0	96.4	97.2
VMA, %	15.7	15.6	14.4
VFA, %	74.0	77.2	80.3
F/P_{be}	1.13	1.03	1.15
Fine Aggregate Angularity	46.4	46.0	43.5
Percent Crush Count, 1 Face	100.0	100.0	100.0
Number of Samples	---	4	4

Table 7.17 Aggregate and Hot Mix Asphalt Characteristics for 8-Mile Road (Warren)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	99.0	98.9	99.4
9.5	87.3	90.3	90.0
4.75	55.7	58.3	58.6
2.36	29.0	29.3	30.2
1.18	18.8	18.4	18.9
0.600	14.0	13.2	13.5
0.300	10.0	9.7	10.0
0.150	6.6	6.3	6.4
0.075	4.8	4.2	4.3
Mixture Characteristics			
Binder Content	5.3	5.3	5.4
%Gmm at N_{design}	96.0	96.0	96.6
VMA, %	15.3	14.8	14.5
VFA, %	70.6	72.7	76.8
F/P_{be}	1.00	0.92	0.93
Fine Aggregate Angularity	47.1	45.7	45.2
Percent Crush Count, 1 Face	96.7	98.5	98.0
Number of Samples	---	4	4

Table 7.18 Aggregate and Hot Mix Asphalt Characteristics for I-94 (Ann Arbor, 4E30)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	99.3	97.2	96.8
9.5	87.2	82.5	84.2
4.75	47.1	45.6	47.4
2.36	29.3	28.9	29.5
1.18	20.3	20.4	20.5
0.600	14.8	14.9	15.1
0.300	10.3	10.6	10.8
0.150	6.9	7.1	7.3
0.075	4.8	5.1	5.3
Mixture Characteristics			
Binder Content	5.2	4.7	4.8
%Gmm at N_{design}	96.0	97.4	97.5
VMA, %	14.7	12.8	12.8
VFA, %	72.9	79.5	80.7
F/P_{be}	1.08	1.29	1.30
Fine Aggregate Angularity	46.5	44.7	44.7
Percent Crush Count, 1 Face	97.7	98.2	97.6
Number of Samples	---	4	2

Table 7.19 Aggregate and Hot Mix Asphalt Characteristics for I-94 (Ann Arbor, SMA)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0
12.5	96.7	97.0	97.3
9.5	79.0	83.1	79.8
4.75	35.8	36.0	33.1
2.36	17.8	20.8	19.3
1.18	13.7	16.8	15.7
0.600	11.8	15.1	14.1
0.300	10.7	13.9	13.1
0.150	9.5	11.7	11.0
0.075	8.3	7.7	7.2
Mixture Characteristics			
Binder Content	6.6	6.0	5.7
%Gmm at N_{design}	96.0	96.8	95.7
VMA, %	17.8	16.4	16.9
VFA, %	77.4	80.3	74.5
F/P_{be}	1.41	1.46	1.43
Fine Aggregate Angularity	46.4	---	---
Percent Crush Count, 1 Face	99.7	100.0	100.0
Number of Samples	---	4	4

Table 7.20 Aggregate and Hot Mix Asphalt Characteristics for I-94 (Ann Arbor, 3E30)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	99.7	99.3	98.4
12.5	87.5	83.6	84.0
9.5	79.4	72.9	73.6
4.75	37.6	40.3	38.6
2.36	26.1	26.5	25.4
1.18	19.1	19.3	18.8
0.600	14.4	14.8	14.5
0.300	10.2	11.0	10.7
0.150	6.3	7.1	7.1
0.075	4.7	5.4	5.1
Mixture Characteristics			
Binder Content	5.0	4.5	4.4
%Gmm at N_{design}	96.0	98.0	98.1
VMA, %	13.7	11.4	11.3
VFA, %	71.0	82.5	82.8
F/P_{be}	1.15	1.49	1.46
Fine Aggregate Angularity	46.2	44.3	42.2
Percent Crush Count, 1 Face	96.1	98.7	98.5
Number of Samples	---	4	1

Table 7.21 Aggregate and Hot Mix Asphalt Characteristics for M-28 (Brimley)

	Design	Truck	Paver
Gradation, Percent Passing			
Sieve Size, mm			
25.0	100.0	100.0	100.0
19.0	100.0	99.7	100.0
12.5	100.0	99.1	98.9
9.5	93.4	93.0	91.9
4.75	67.1	67.3	64.3
2.36	53.0	53.1	50.9
1.18	43.9	43.5	42.0
0.600	31.4	31.5	30.9
0.300	19.0	13.4	13.7
0.150	9.3	6.7	7.0
0.075	4.3	4.3	4.4
Mixture Characteristics			
Binder Content	6.1	5.9	5.7
%Gmm at N_{design}	96.0	95.0	95.5
VMA, %	16.4	16.9	16.5
VFA, %	75.6	70.8	72.5
F/P_{be}	0.70	0.83	0.90
Fine Aggregate Angularity	44.1	-	-
Percent Crush Count, 1 Face	79.5	88.8	88.0
Number of Samples	---	4	4

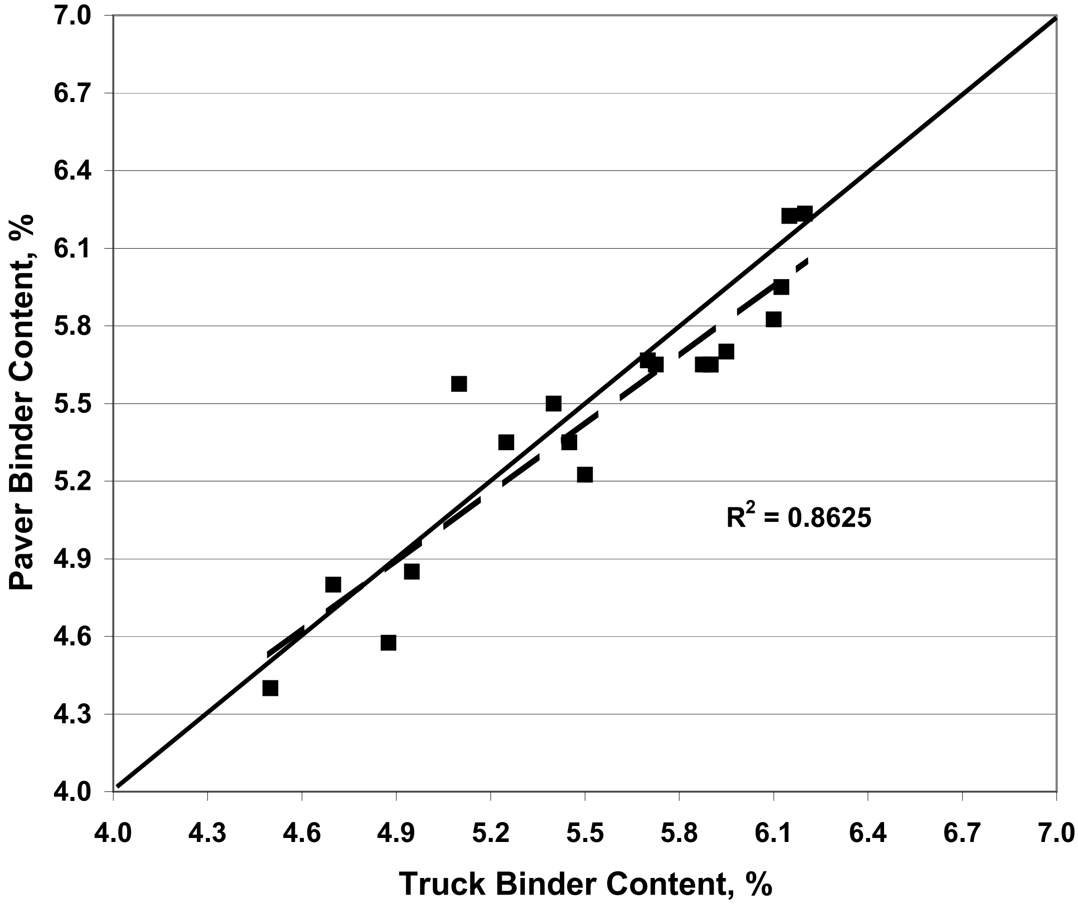


Figure 7.1 Comparison of Truck and Paver Binder Contents

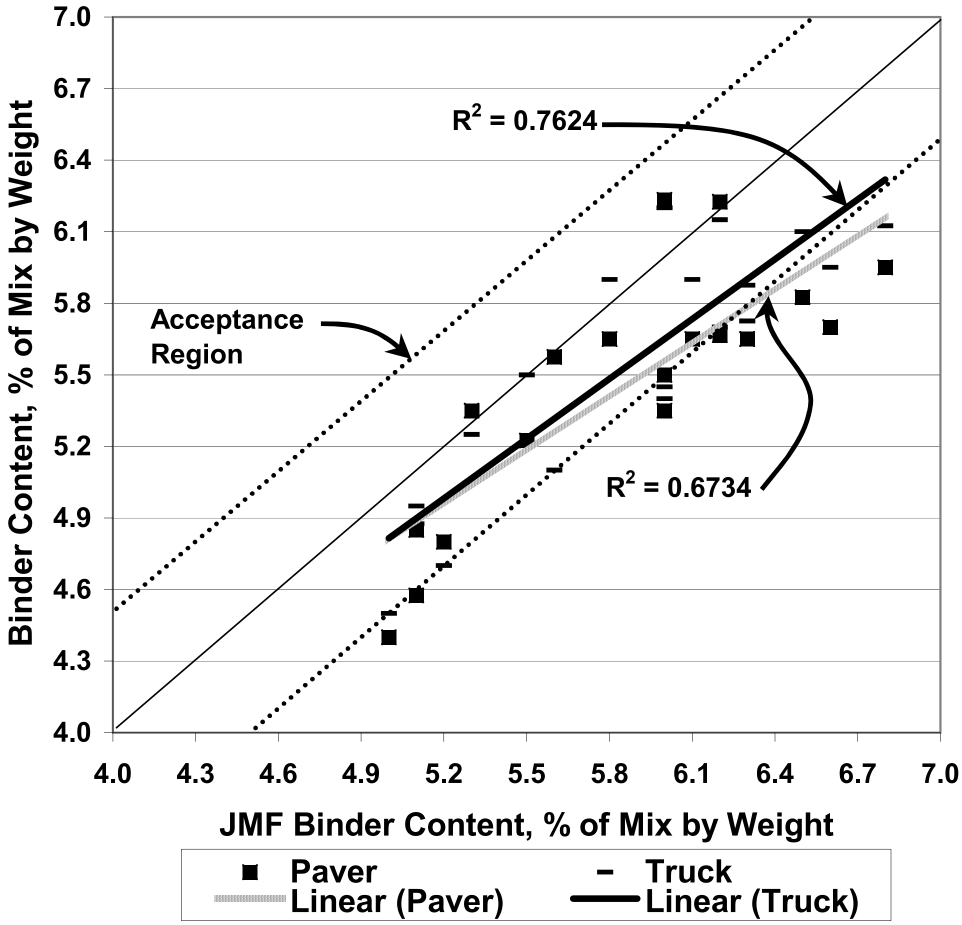


Figure 7.2 Comparison of Truck and Paver Binder Contents to JMF Binder Contents

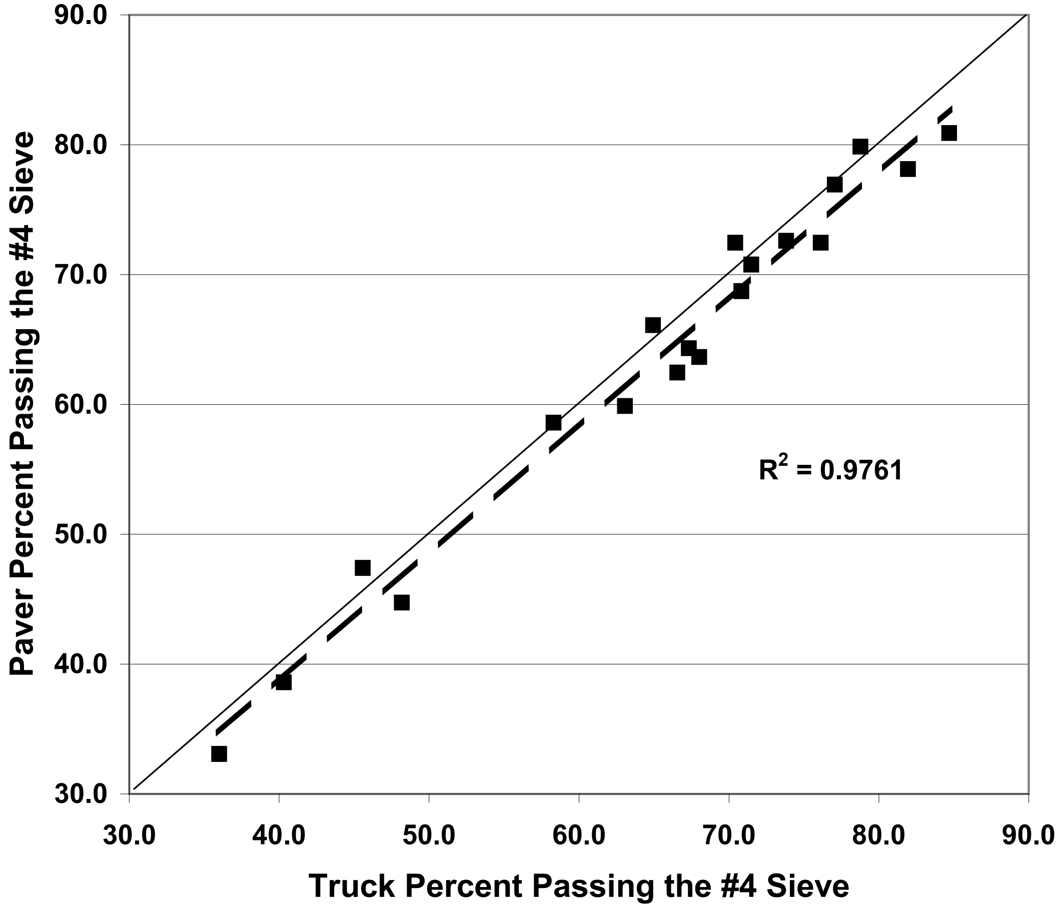


Figure 7.3 Comparison of Truck and Paver Percent Passing #4 Sieve

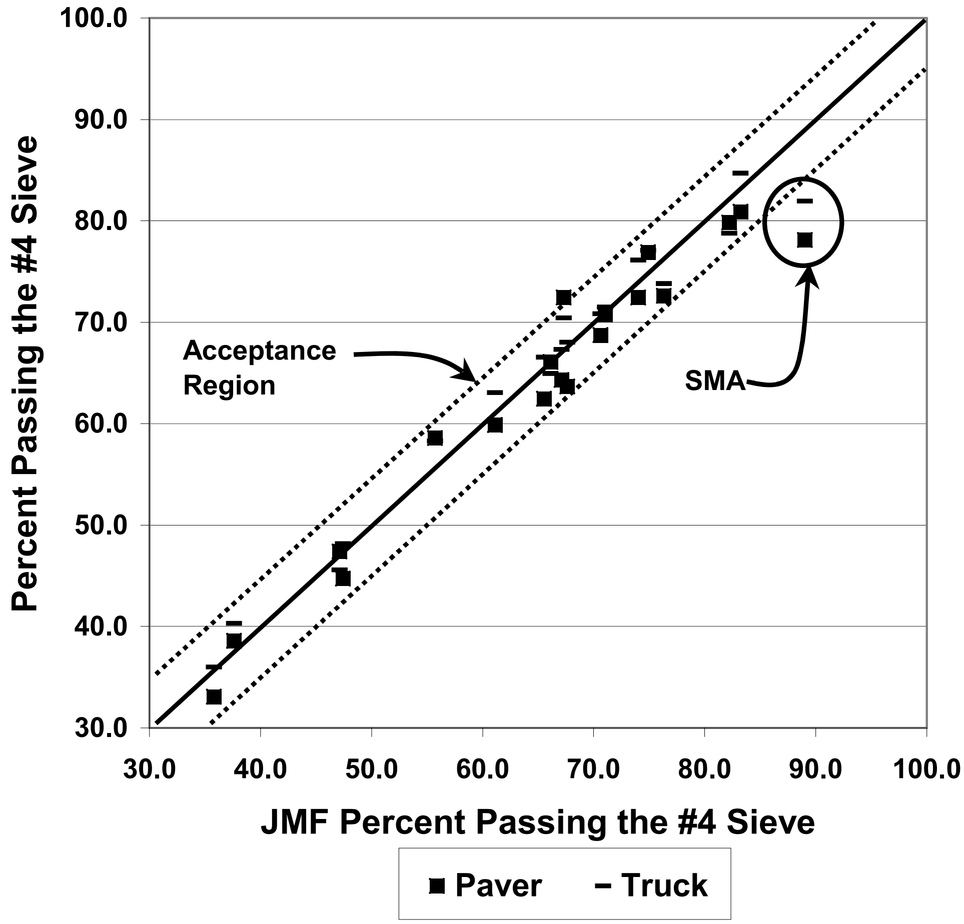


Figure 7.4 Comparison of Truck and Paver #4 to JMF #4 Design Values

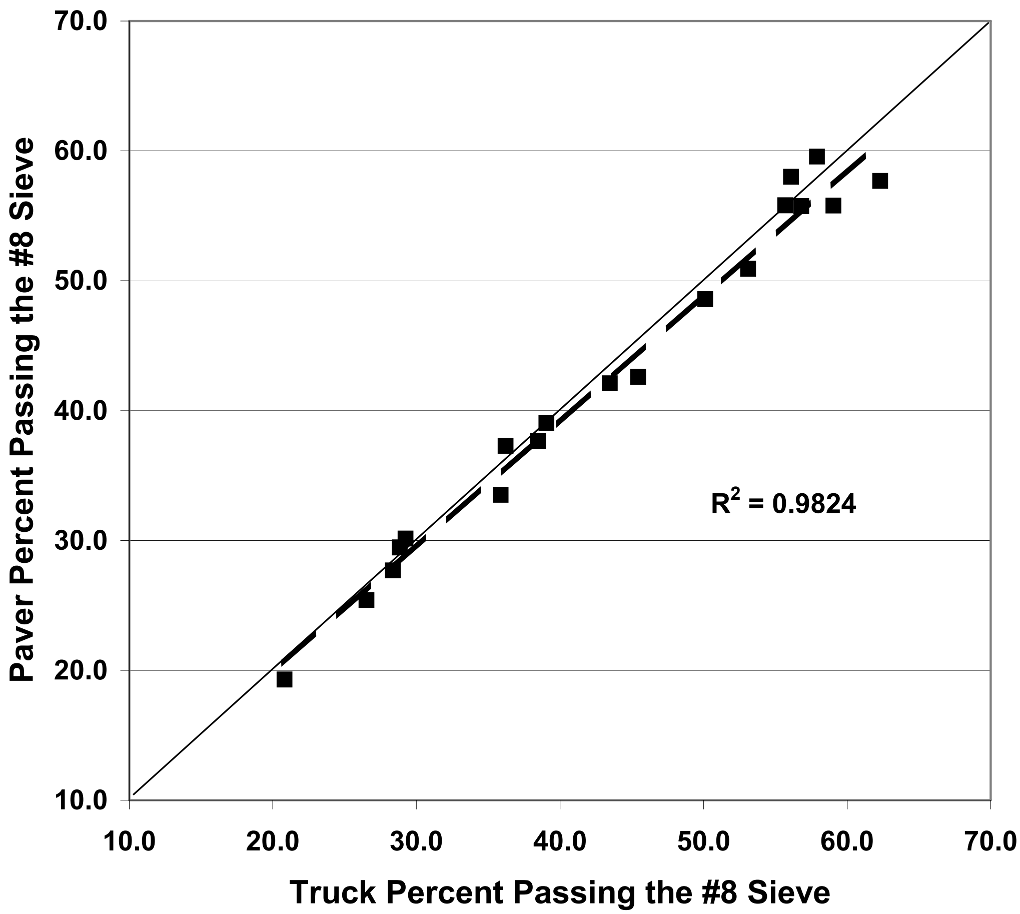


Figure 7.5 Comparison of Truck and Paver Percent Passing #8 Sieve

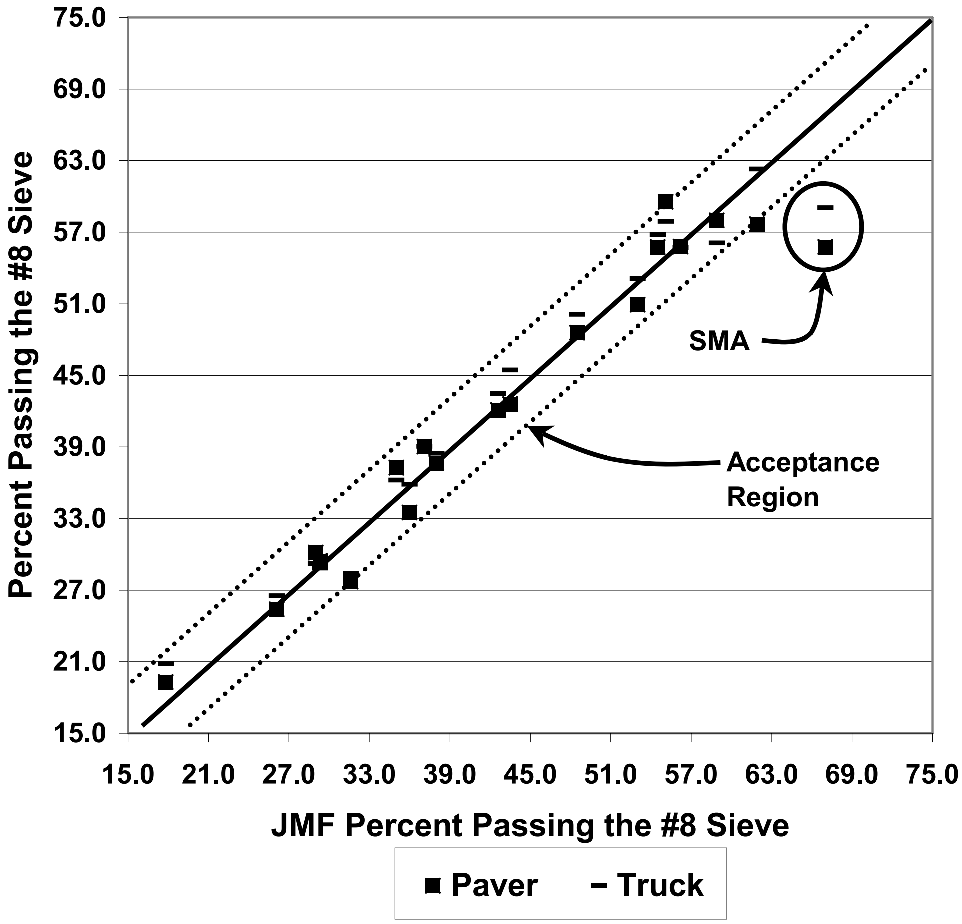


Figure 7.6 Comparison of Truck and Paver #8 to JMF #8 Design Values

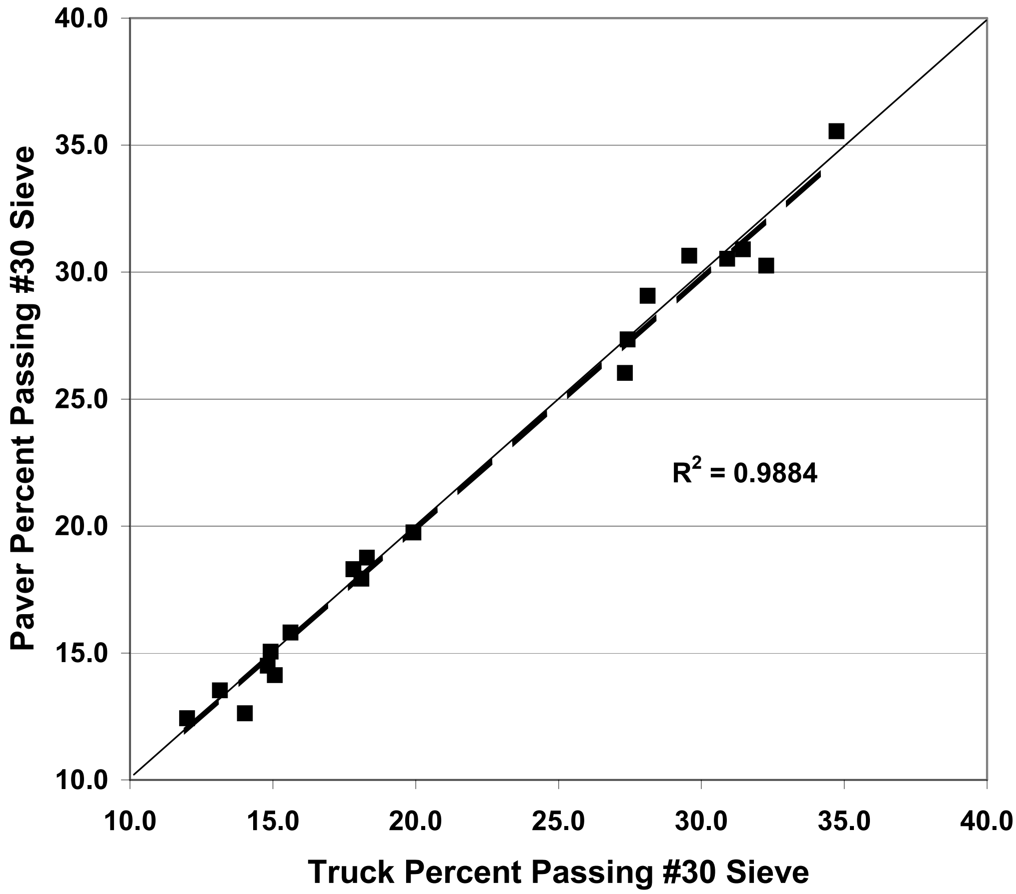


Figure 7.7 Comparison of Truck and Paver Percent Passing #30 Sieve

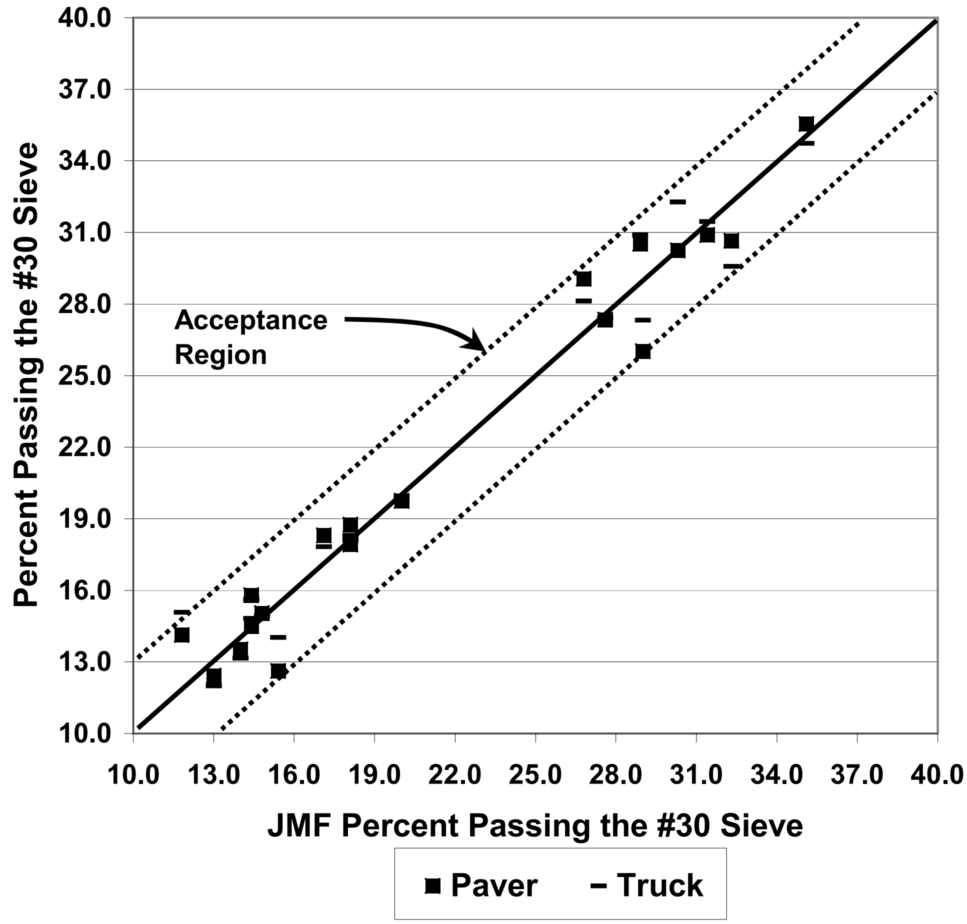


Figure 7.8 Comparison of Truck and Paver #30 to JMF #30 Design Values

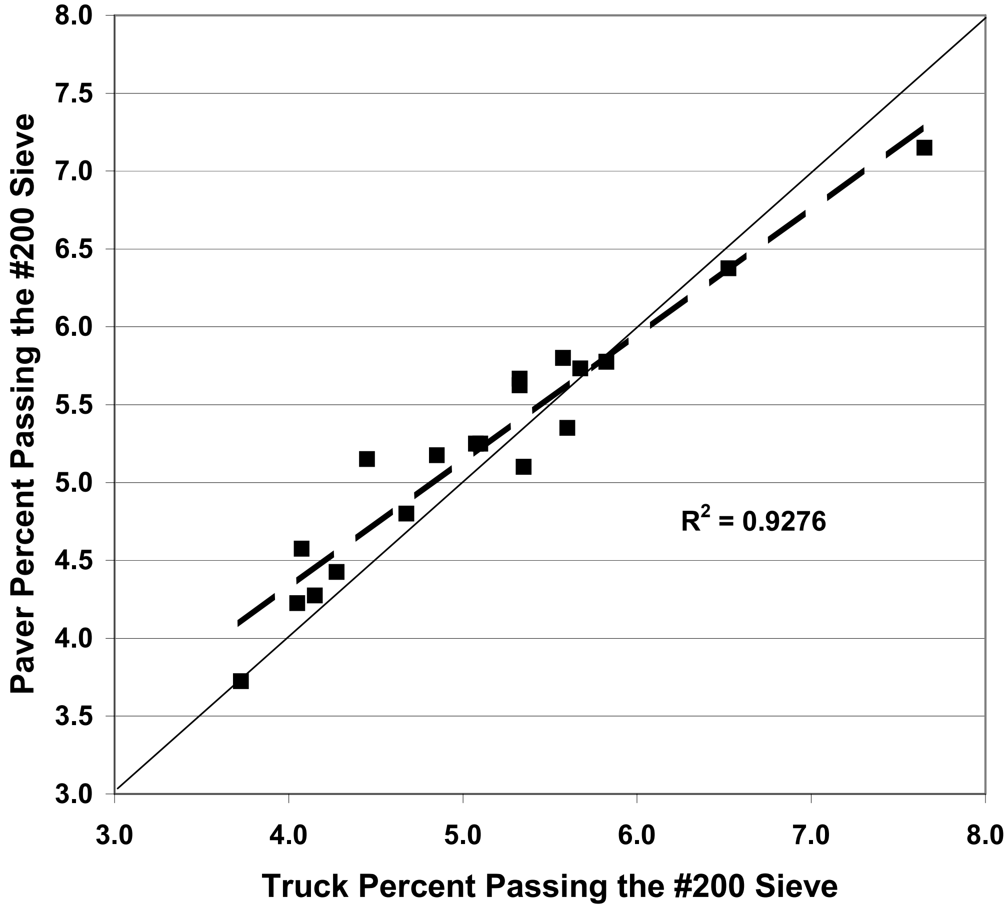


Figure 7.9 Comparison of Truck and Paver Percent Passing #200 Sieve

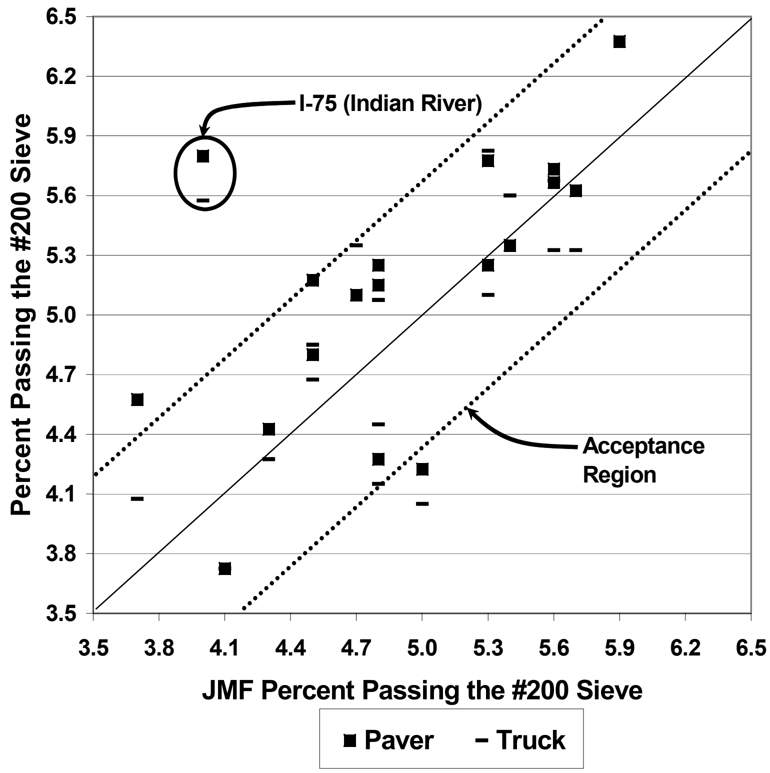


Figure 7.10 Comparison of Truck and Paver #200 to JMF #200 Design Values

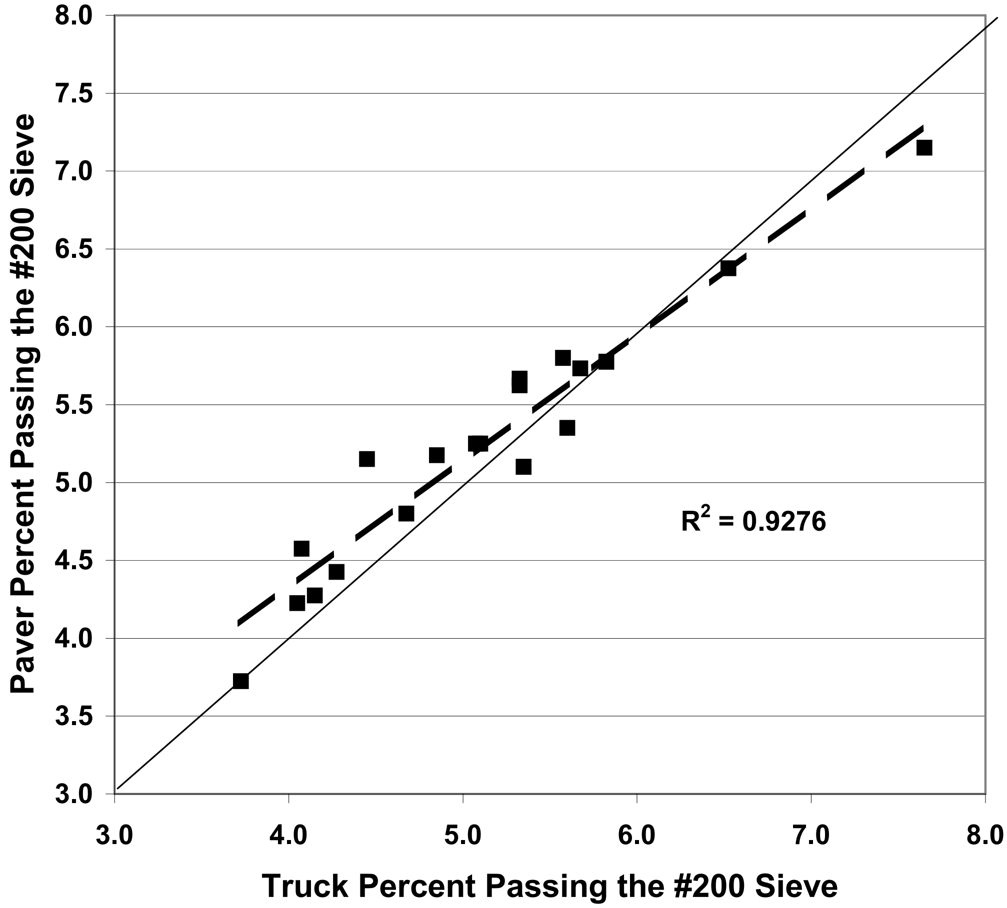


Figure 7.11 Comparison of Truck and Paver Percent Passing #200 Sieve

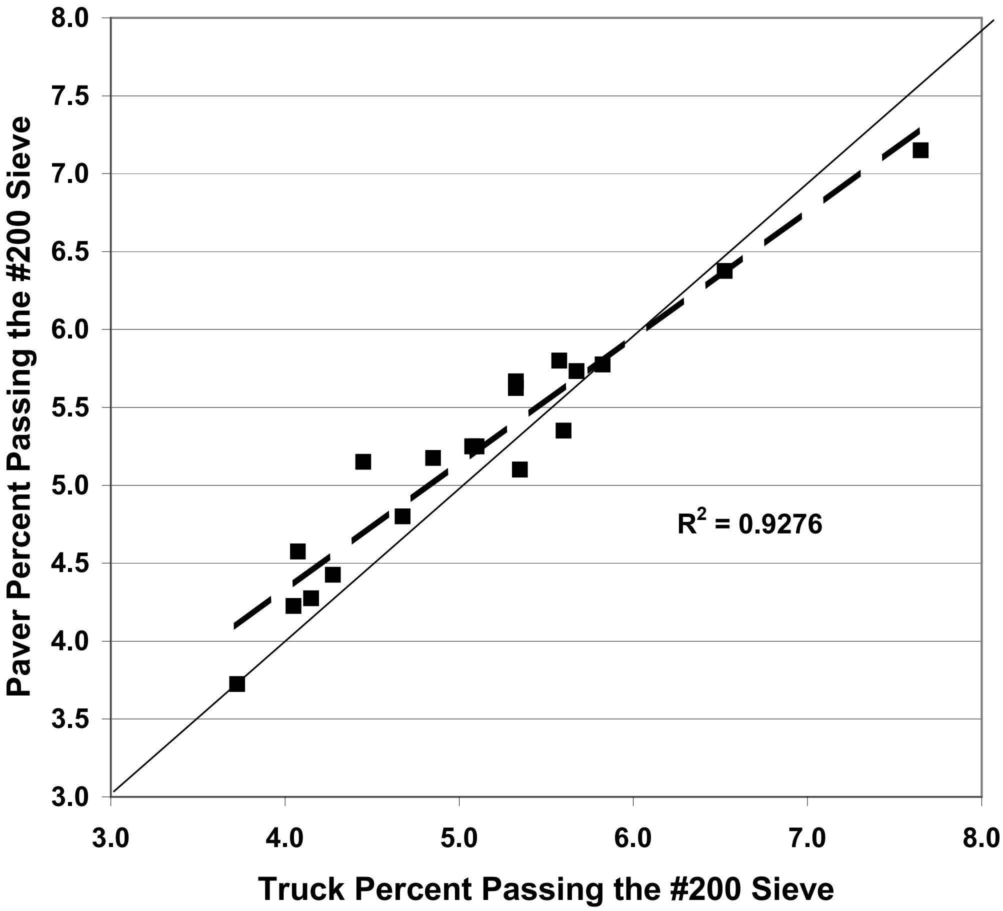


Figure 7.12 Comparison of Truck and Paver Fine Aggregate Angularity

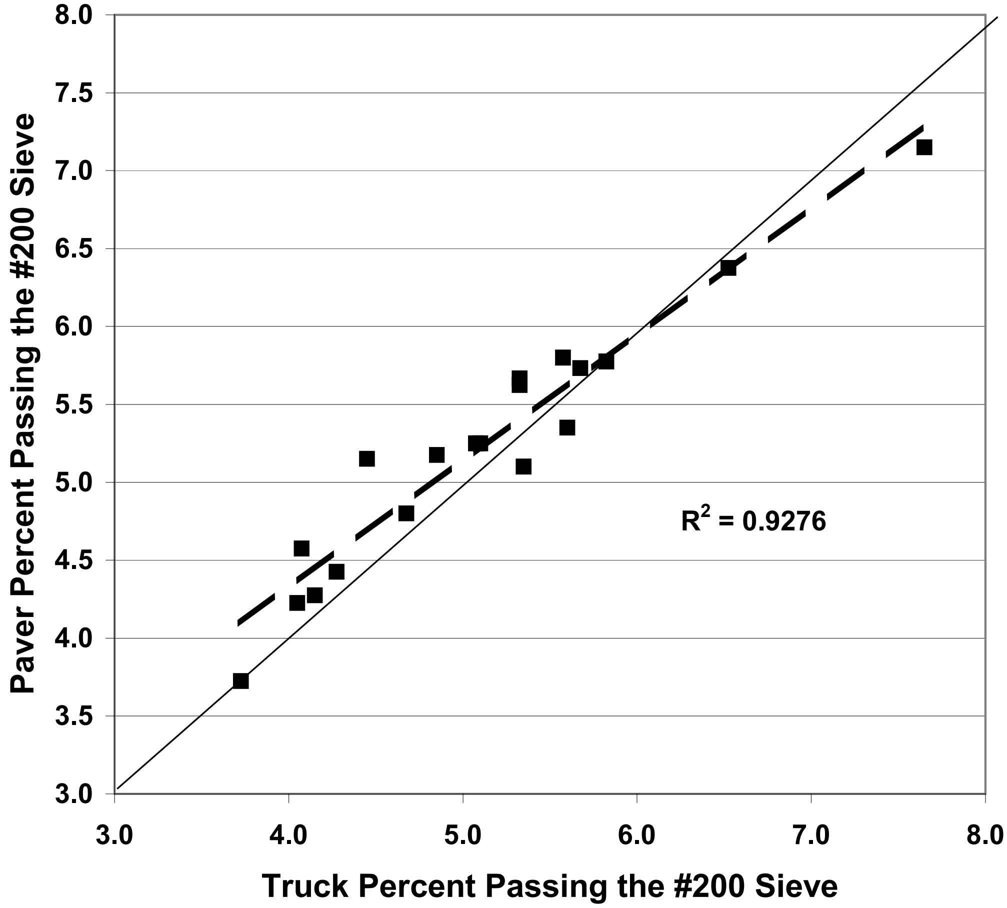


Figure 7.13 Comparison of Truck and Paver Fines to Effective Binder Content Ratios

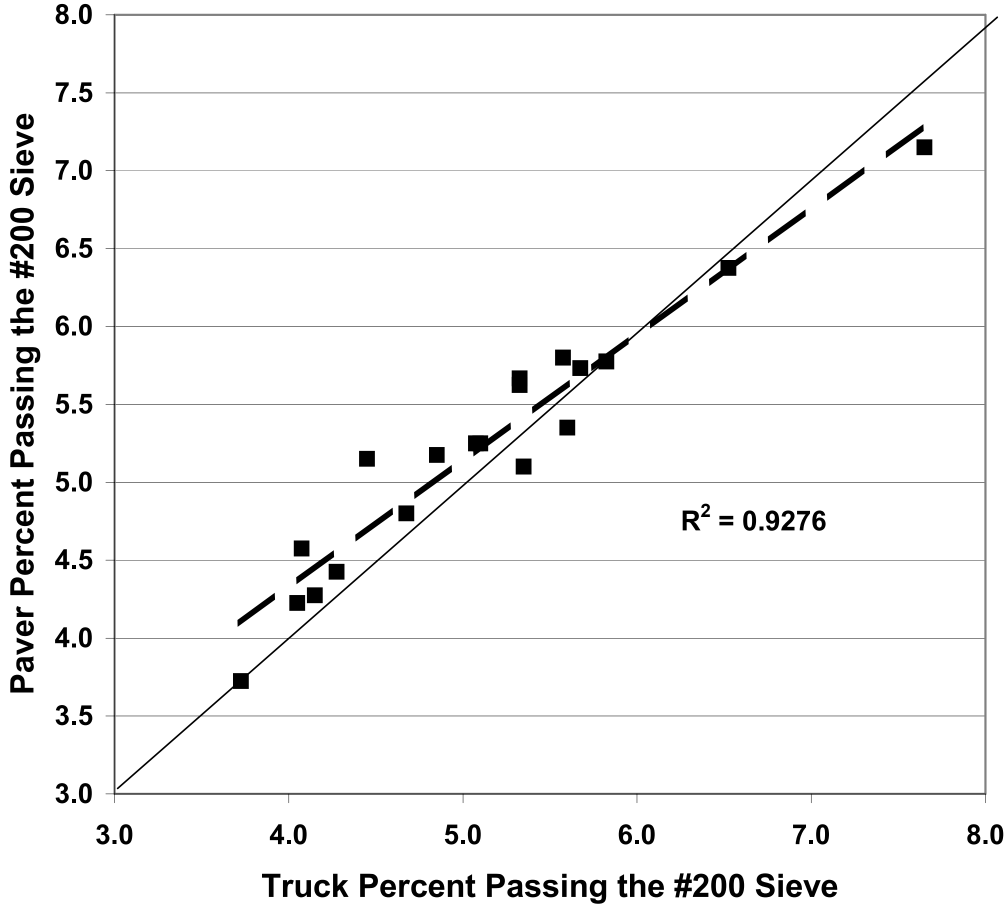


Figure 7.14 Comparison of Truck and Paver Effective Binder Ratios to JMF Design Values

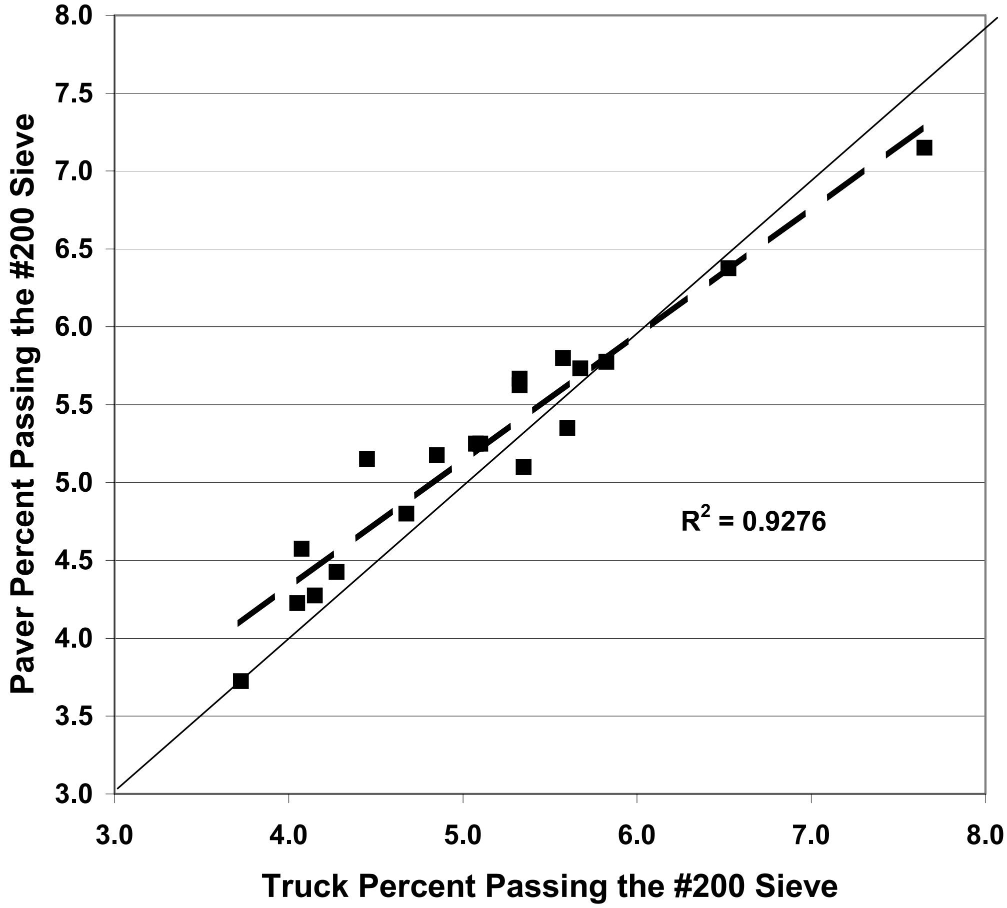


Figure 7.15 Comparison of Truck and Paver Percent Air Voids at N_{design}

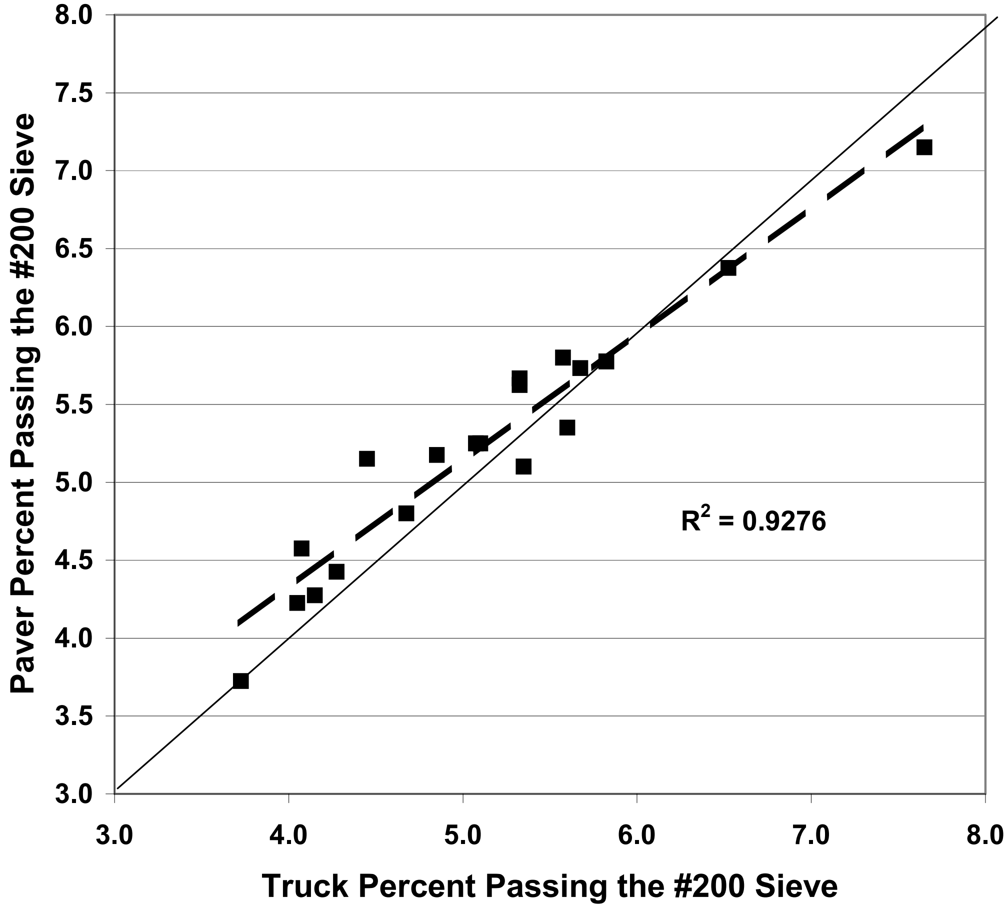


Figure 7.16 The Non-Linear Effects of Back-Calculating Percent Air Voids at N_{design}

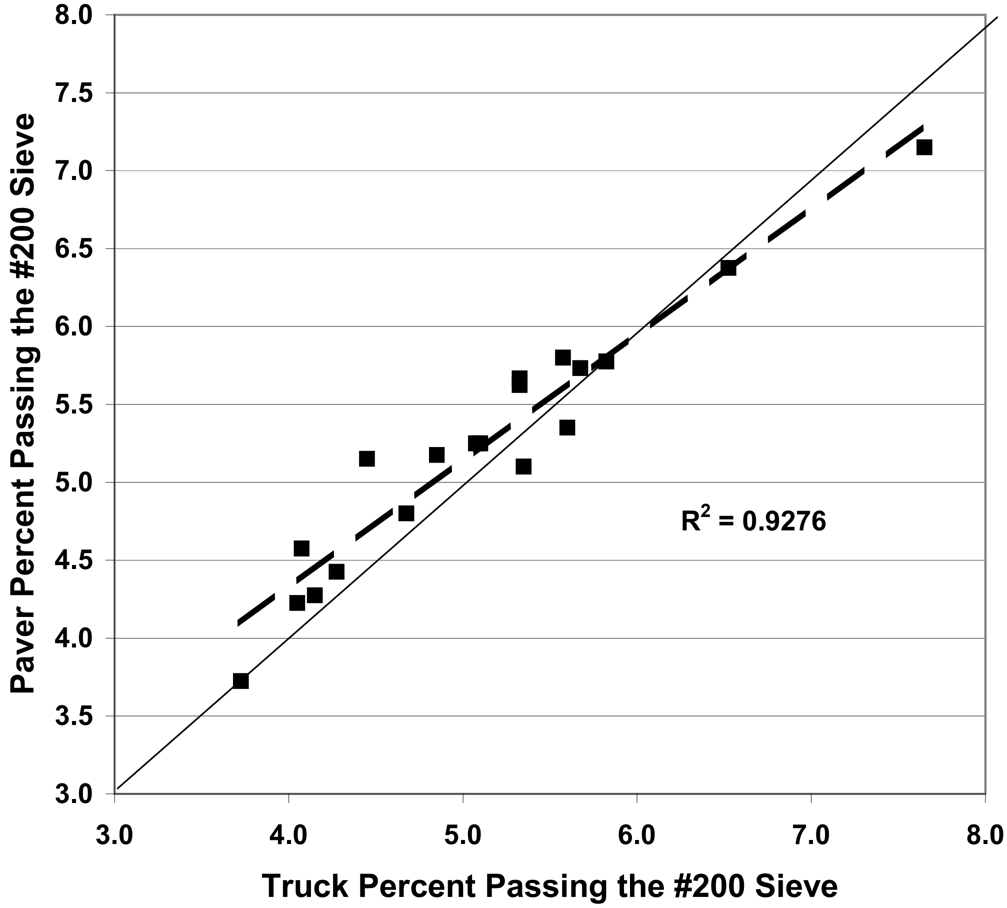


Figure 7.17 Comparison of Truck and Paver Voids in the Mineral Aggregate

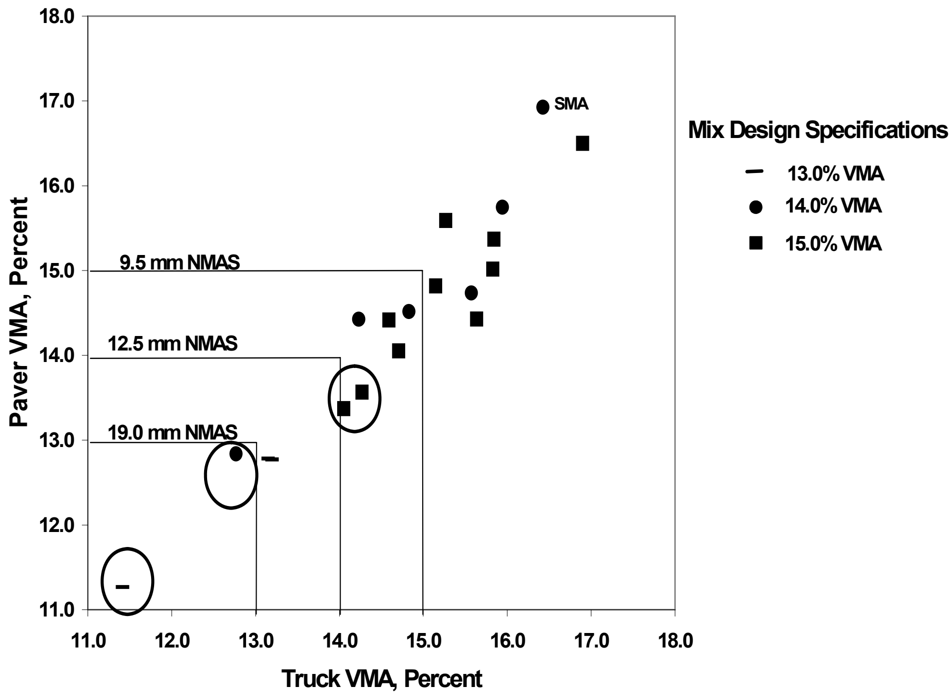


Figure 7.18 Comparison of Truck and Paver Voids in the Mineral Aggregate and the Effect of Nominal Maximum Aggregate Size

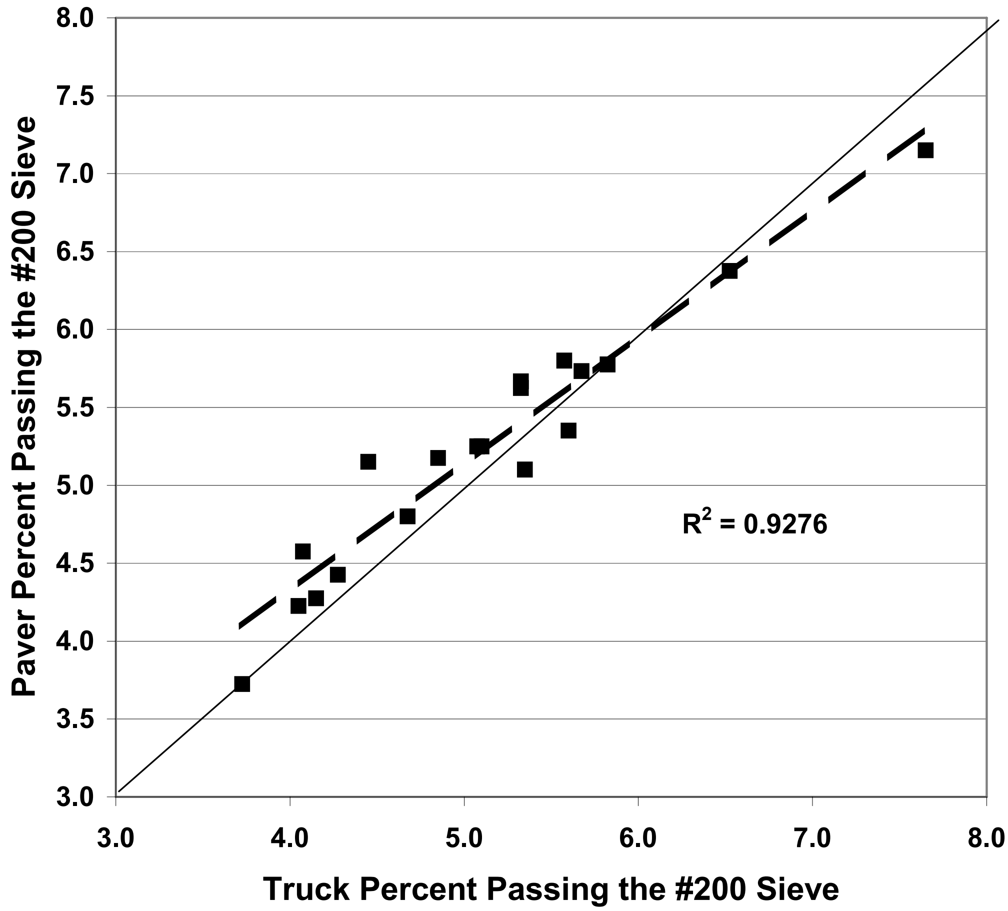


Figure 7.19 Comparison of Truck and Paver Voids in the Mineral Aggregate and JMF Design Values

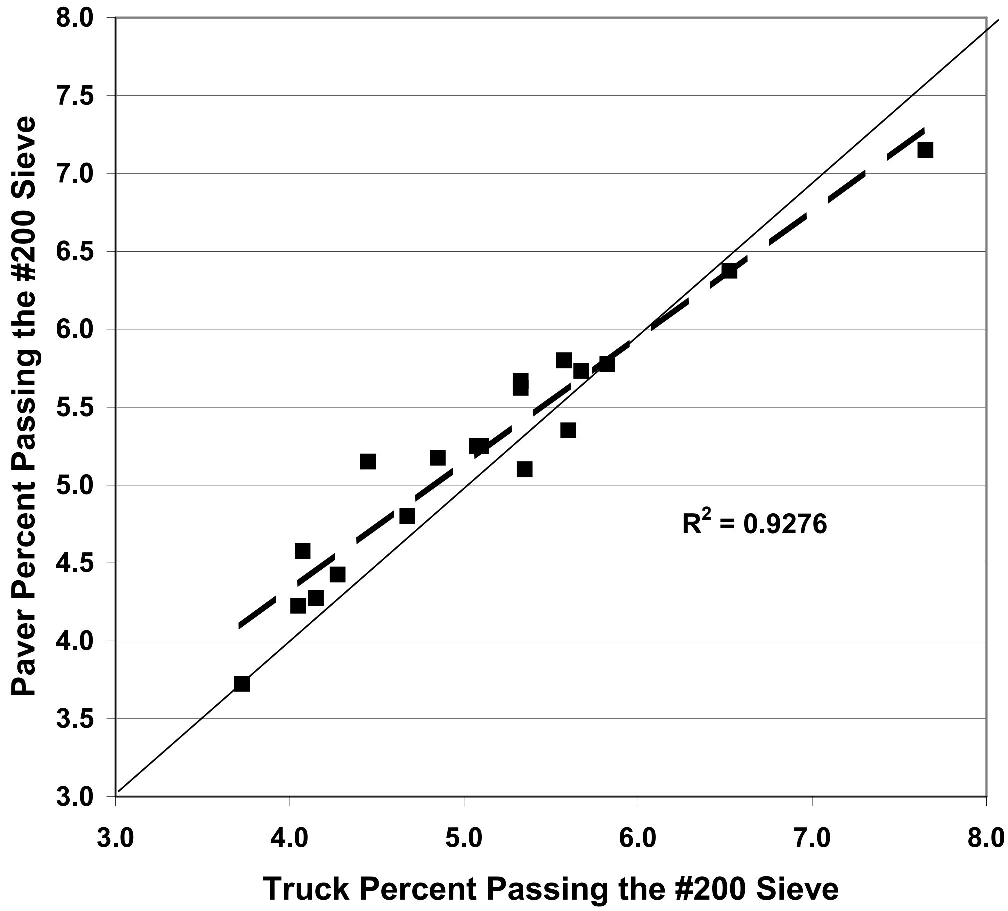


Figure 7.20 Comparison of Truck and Paver Voids Filled with Asphalt

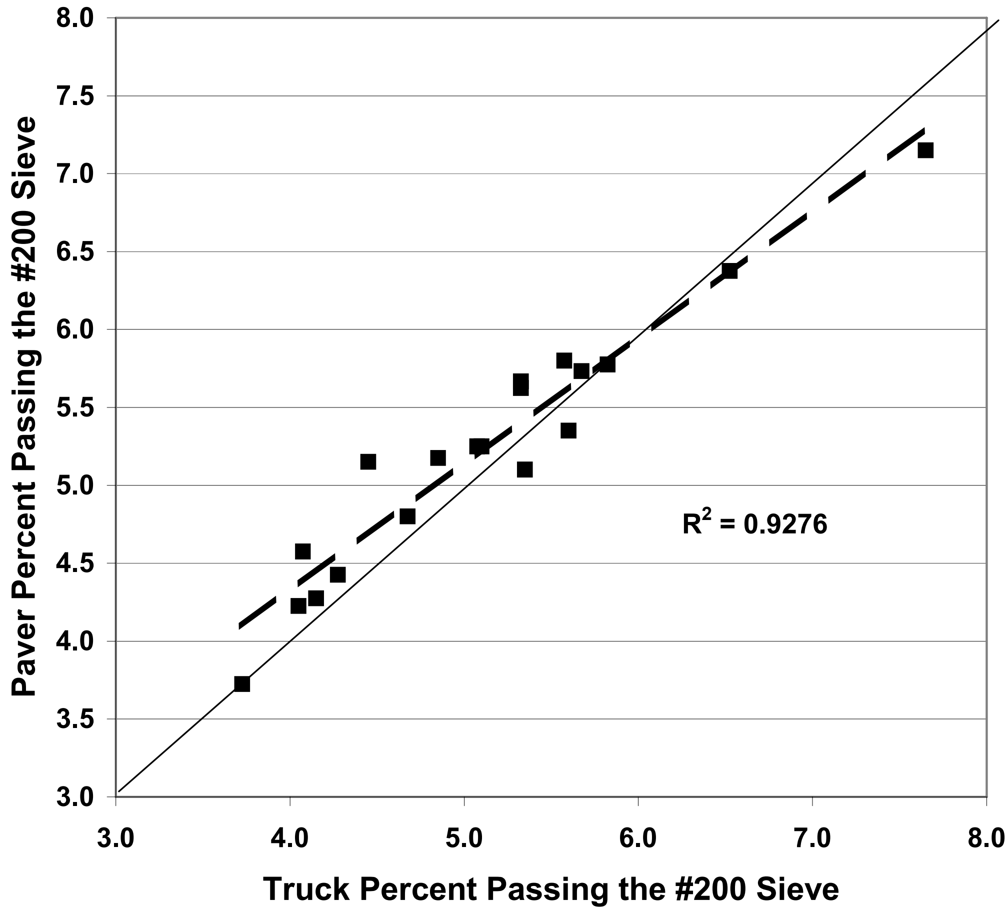


Figure 7.21 Comparison of Truck and Paver Voids Filled with Asphalt to JMF Design Values

7.5 An Empirical Rut Prediction Model

The APA is an empirical performance test. The value measured, the rut depth, cannot be used as a basis for a mechanistic model. In the past, the APA rut depth at 8000 cycles has been used as to identify rut-prone HMA mixtures before they are used in the field. This is done by establishing a pass/fail rut depth. For example, based upon past experience some state highway agencies have established a rut depth of 5mm as the dividing point between rut-prone and rut resistant HMA mixtures. Hence, no HMA mixtures with an APA rut depth of 5 mm or greater would be constructed in the field. No attempts have been identified in the literature review to use the APA to predict how many 80-kN Equivalent Single Axle Loadings (ESALs) an HMA pavement can be loaded with until failure. This chapter presents the following:

Section 7.7: A methodology of converting APA rut depth and APA cycles to field rut depth and 80-kN ESALs.

Section 7.8: The creation of a PBS based upon the methods presented in Section 7.7.

Section 7.9: A preliminary Performance Based APA Specification for Michigan.

7.6 Converting APA Test Performance to Field Performance

The wheel loading in the APA is supposed to simulate a wheel loading on an in-service pavement while the rut created is supposed to be similar to the rut created by trafficking on in-service pavements. In this section, a method of converting the APA rut depth and the number of APA load cycles to actual pavement rut depth and ESALs will be presented.

7.6.1 Determination of an APA Rut Depth that is Equivalent to Rutting Failure On an In-service HMA Pavement

To determine an APA rut depth that is equal to failure on an in-service pavement, a pavement failure rut depth must first be determined. Barksdale (1972) found that for pavements with a 2% crown (typical for the United States) rut depths of 0.5 in. (12.5 mm) are sufficiently deep to hold enough water to cause a car traveling 50 mph to hydroplane. The rut depth referred to by Barksdale is the total rut depth, not the downward rut depth (Figure 7.22). According to pavement rut depth taken from Westrack (FHWA, 1998) a 12.5 mm total rut depth is approximately equivalent to a downward rut depth of 10 mm (Figure 7.23). From APA data also taken from Westrack pavements it can be determined that a 10 mm downward rut depth on an in-service pavement correlates well with a 7 mm rut depth in the APA (Figure 7.24). Based upon these correlations, an APA failure rut depth of 7 mm will be used in establishing an empirical model.

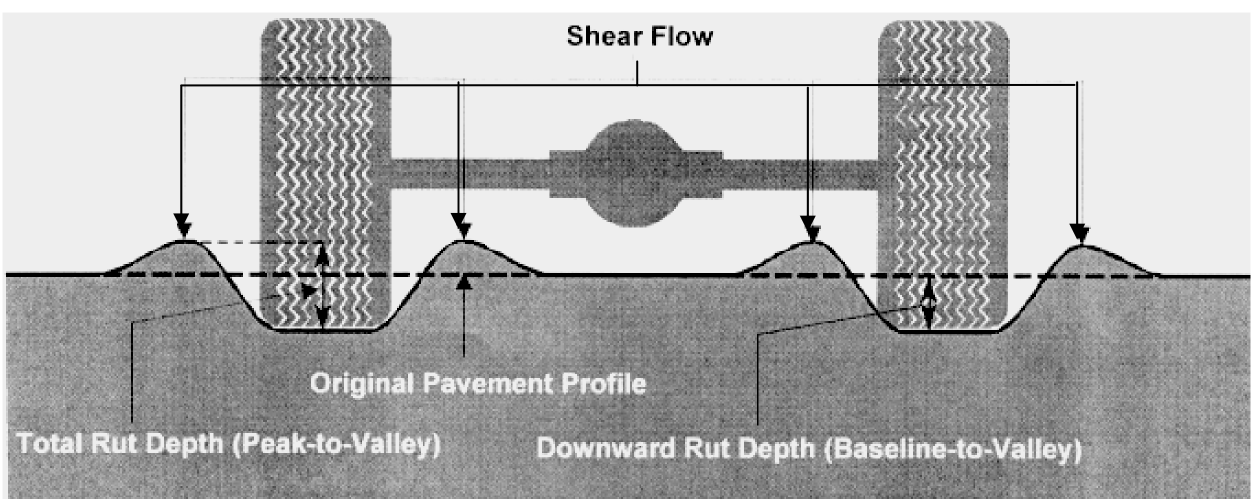


Figure 7.22 Downward Versus Total Rut Depth

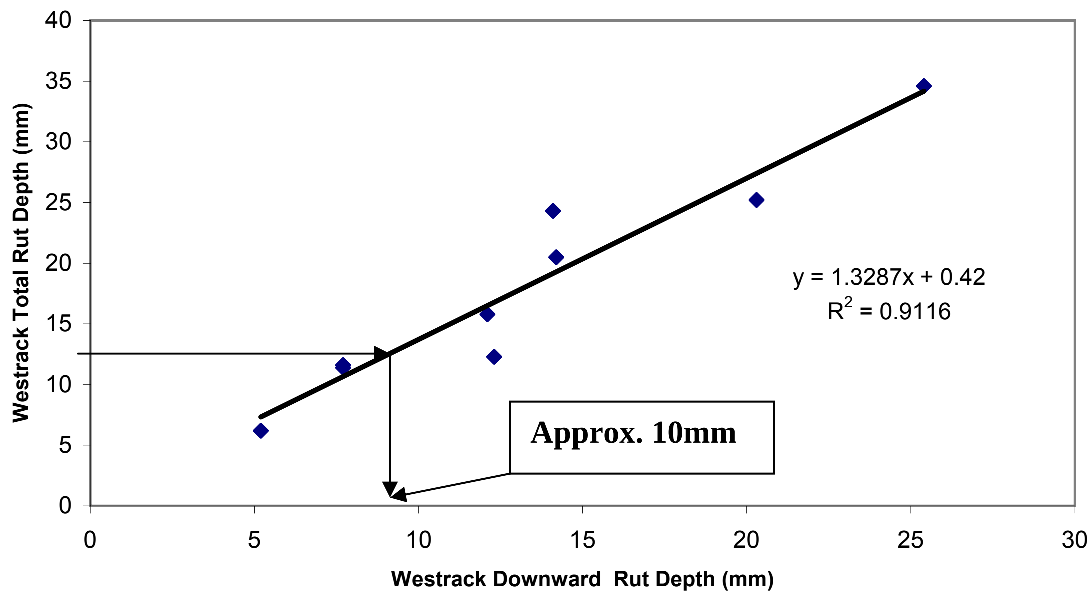


Figure 7.23 Westrack Total Rut Depth vs. Westrack Downward Rut Depth (Williams and Prowell, 1999)

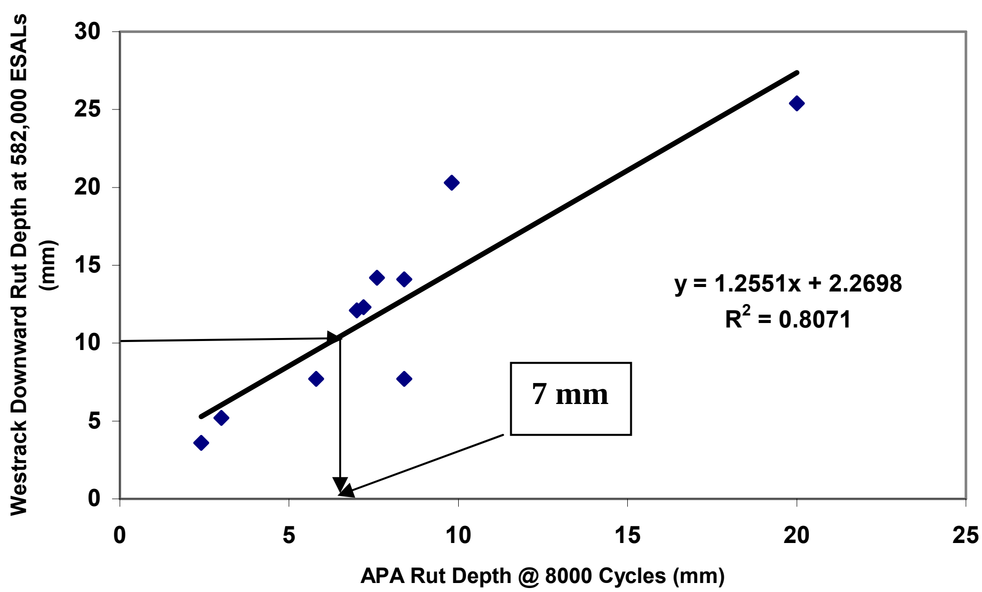


Figure 7.24 Westrack Downward Rut Depth vs. APA Rut Depth (Williams and Prowell, 1999)

7.6.2 Determination of How Many 80-kN ESALs are equal to one APA cycle

The WesTrack experiment provided a unique opportunity to compare APA results with a full-size pavement testing facility where both the loading and temperature were known. APA test specimens were taken directly from the wheel paths of the test track before truck loading and were tested at 60 °C - nearly the same as the average high pavement temperature of 57.53 °C @ 12.7 mm depth (Williams and Prowell, 1999). As can be seen from Figure 7.24, the WesTrack pavement rut depths correlated very well with the APA test specimens taken from WesTrack.

Although the Westrack and APA test temperatures are nearly the same, the number of ESALs per APA cycle cannot be found simply by dividing 582,000 ESALs by 8000 cycles (Figure 5.3). This is because the trucks that loaded WesTrack traveled slower then ordinary trucks on highways and the wheel wander of the WesTrack trucks was tighter then ordinary truck traffic. Both truck speed and wheel wander have to be corrected as follows before the amount of rutting ESALs per APA cycles can be determined.

First, the WesTrack trucks traveled at 65 kph, which is slower then ordinary truck traffic, which travels approximately 100 kph at highway speeds. Because of the viscoelastic nature of asphalt cement the longer loading time caused by slow moving trucks causes increased HMA pavement damage. Haddock et al. (1998), in a study by Purdue University conducted on the Indiana Department of Transportation's (INDOT) accelerated pavement testing (APT) facility, developed a relationship between rut depth and truck speed (Figure 7.25). According to the figure and assuming a HMA pavement of high density, a truck traveling at 65 kph does approximately 12% more pavement damage then does a truck traveling 100 kph.

Secondly, the Westrack trucks, because of their guidance system, wandered less then a ordinary trucks on standard 12-foot lanes. Wheel wander refers to the fact that trucks tend to "wander" about the traffic lane rather then staying exactly in the center of the lane. This wheel wander tends to distribute the truck loadings over a wider pavement area and consequently reduce the depth of ruts that single wheel path traffic would create. From past experience it has been shown that trucks tend to wander over a width of 460 mm when traveling on a 12-foot traffic lane (Lee et el, 1983). The WesTrack Trucks wandered over a width of 127 mm. A decrease in wheel wander causes the truck loads to be distributed over a smaller pavement area and consequently causes more pavement damage. Haddock et al (1998) developed a relationship between wheel wander and rut depth using the INDOT APT, which had a transverse mechanism to include wheel wander. Figure 7.26 shows the relationship between rut depth and wheel wander. Based on the equation of the high-density curve (Equation 7.1), the increased amount of rutting caused by the WesTrack trucks can be determined

$$\begin{aligned} Increased\ \ Damage &= (Rut\ Damage\ at\ 460\ mm\ Wander - Rut\ Damage\ at\ 127\ mm\ Wander) \\ &\quad /(Rut\ Damage\ at\ 127\ mm\ Wander) \\ &= \left(8.2422e^{(-0.0014*127mm)} - 8.2422^{(0.0014*460mm)}\right)/(8.2422e^{(-0.0014*460mm)}) \\ &= 0.594 \end{aligned} \tag{7.1}$$

The WesTrack loaded trucks did 59.4% more damage then ordinary trucks as a result of differences in wheel wander. The previous paragraphs show that the Westrack trucks did more damage per loading then do ordinary trucks. The following equation shows how many ordinary truck ESALs the Westrack Truck ESALs were actually equal to because the decreased truck speed and wheel wander:

$$\begin{aligned} Ordinary\ \ Truck\ \ ESALs &= (582{,}000\ \ ESALs)*(Wander\ \ Adjustment)*(Speed\ \ Adjustment) \\ &= (582{,}000\ \ ESALs)\ *\ (1.594)\ *\ (1.12) \\ &= 1{,}039{,}033\ \ ESALs \end{aligned} \tag{7.2}$$

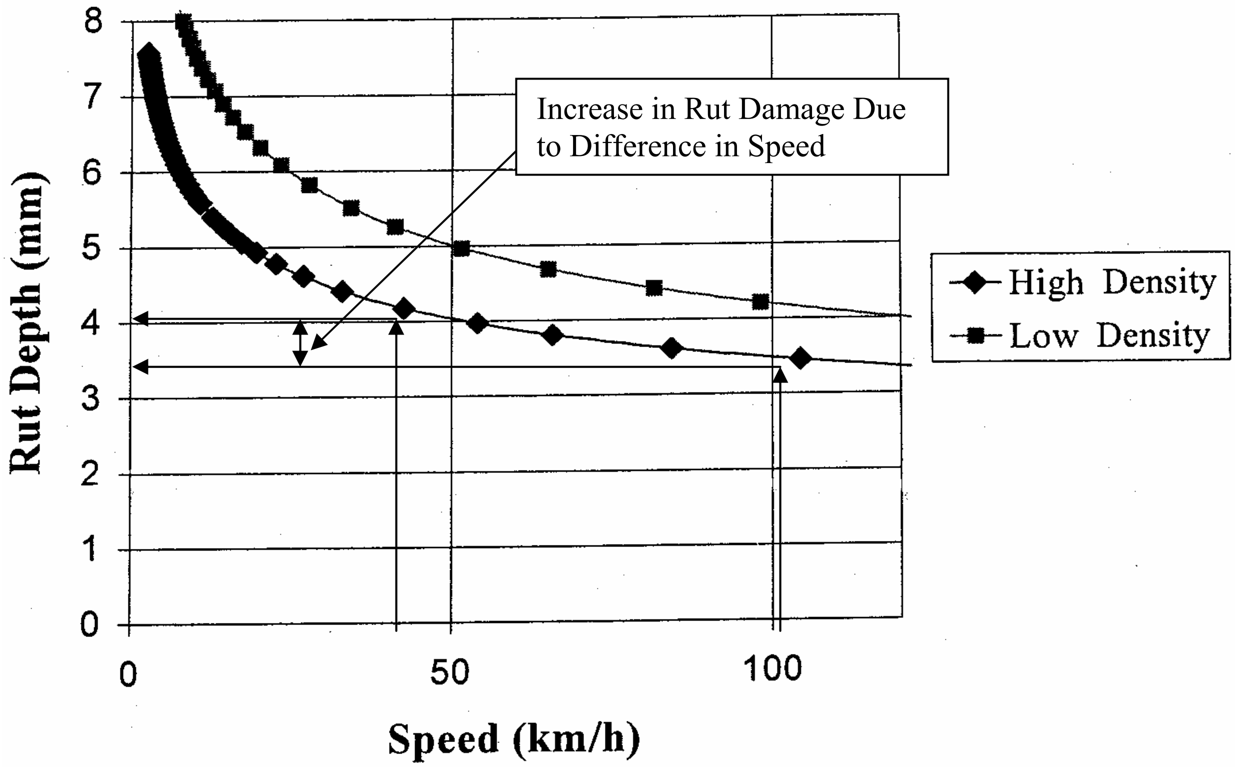


Figure 7.25 The Effect of Speed on Rut Damage (Haddock et al., 1998)

The amount of 80-kN ESALs per APA cycle is calculated as follows:

$$ESALs \quad per \quad APA \quad Cycle = (1{,}039{,}033 \quad ESALs)/(8{,}000 \quad APA \quad Cycles) \quad (7.3)$$
$$= 129.9 \quad ESALs \quad per \quad APA \quad Cycle$$

Based on the previous equation it is estimated that one APA cycle is approximately 129.9 80-kN ESALs. APA testing is typically done at the temperature of the high Performance Grade (PG) of the binder in the HMA mixture, or approximately the highest pavement temperature the HMA mixture will see in-service. Because of this fact, one APA cycle is equal to 129.9 rutting ESALs. Since not all truck loadings occur during times when HMA pavements experience rutting (i.e.

when pavement temperatures approach the upper PG) any PBS would utilizing the APA have to be adjusted to include only rutting ESALs. This is done in the following section.

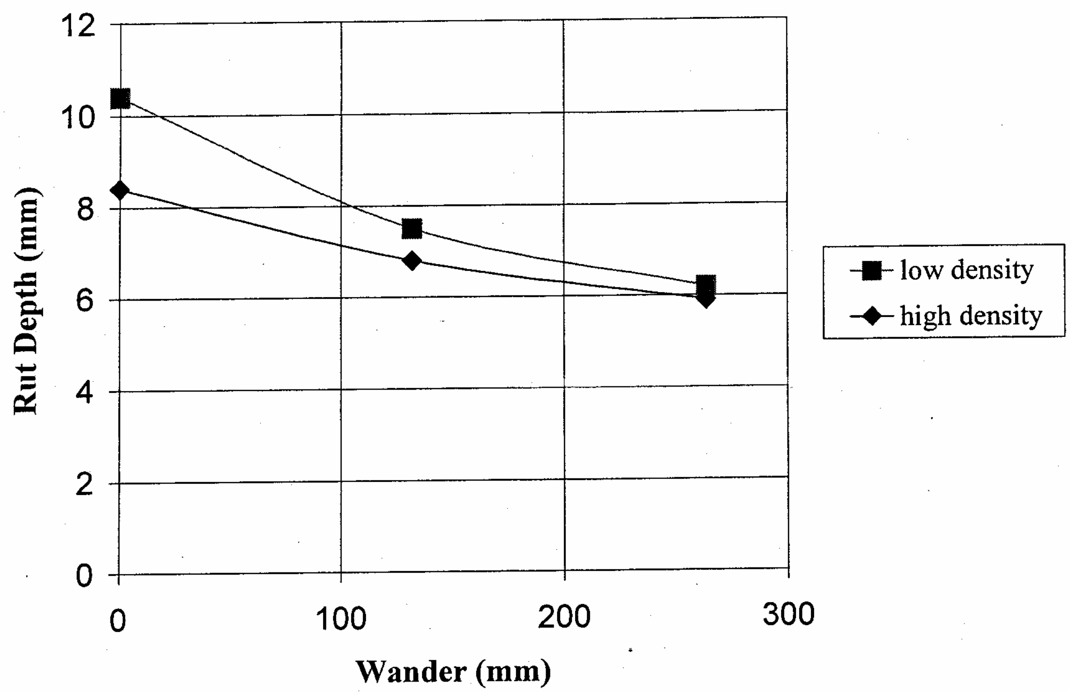


Figure 7.26 The Effect of Wander on Rut Damage (Haddock et al., 1998)

The philosophy of a PBS is to design and construct a HMA pavement that will provide a required level of service (Harvey and Tsai, 1997). In the case of rutting, a PBS should provide an HMA pavement that will not rut to failure within the design life of the HMA pavement. According to the Superpave mixture design system, the design life of an HMA pavement is the traffic volume expected during a 20-year period. For example, a 5E3 HMA mixture is expected to experience 3,000,000 ESALs in its 20-year design life.

As described in the previous two sections, the APA rut depth can be related empirically to in-service pavement rut depth while APA cycles can be converted empirically to 80-kN ESALs. This can serve as the basis for a PBS utilizing the APA and a graphical representation of a PBS using APA data is shown in Figure 7.27. For example, making the simplifying assumption that all ESALs are rutting ESALs, a 5E3 HMA pavement must exhibit less then 7 mm rut at (3,000,000 ESALs)/(129.9 ESALs/APA Cycle) = 23,095 APA Cycles. If a specimen exhibited less then 7 mm of rutting in 23,095 APA cycles this would provide the required performance for a 5E3 HMA mixture. The next section presents a method of calculating the number of rutting ESALs that occur in a Superpave 20-year design life.

7.7 The Development of a Empirical Rut Prediction Model for Michigan

Since asphalt binder viscosity decreases with increasing temperature, HMA rutting occurs when pavement temperatures are above average, particularly in the summer months. More specifically, work done by Mahboub and Little (1988) stated the following assumptions could be made based on Texas HMA pavements:

- Permanent deformation occurs daily over the time interval from 7:30 a.m. to 5:30 p.m.,
- Permanent deformation occurs only in the period from April to October, inclusive, and
- Measurable permanent deformation does not occur at air temperatures below 50 °F (10 °C).

The Superpave 20-year design life includes all ESAL loadings during the entire 20-year design life. Based on the above assumptions, the number of ESALs in the 20-year design life needs to be adjusted to only the ESALs when rutting occurs, or “rutting ESALs”, if a PBS using the APA is to be developed. This process of making this conversion was developed and the steps are summarized in Figure 7.28.

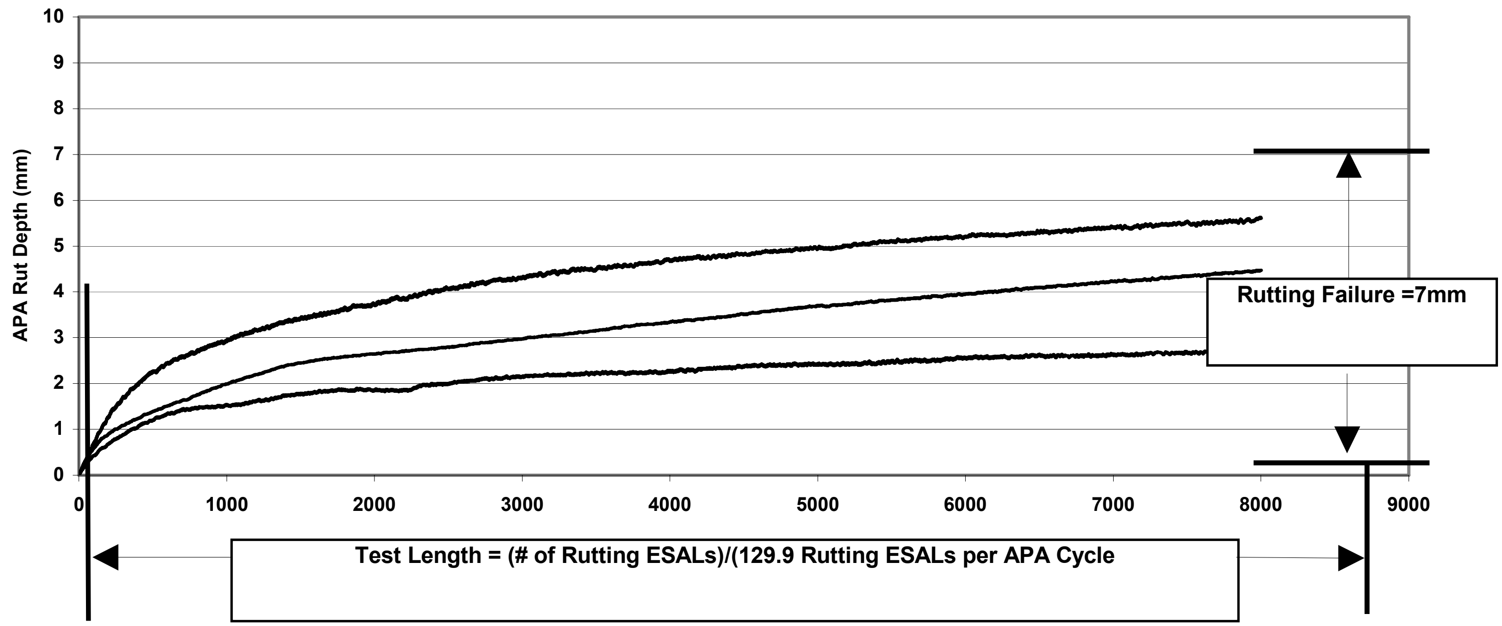


Figure 7.27 A Graphical Representation of a PBS Specification for the APA

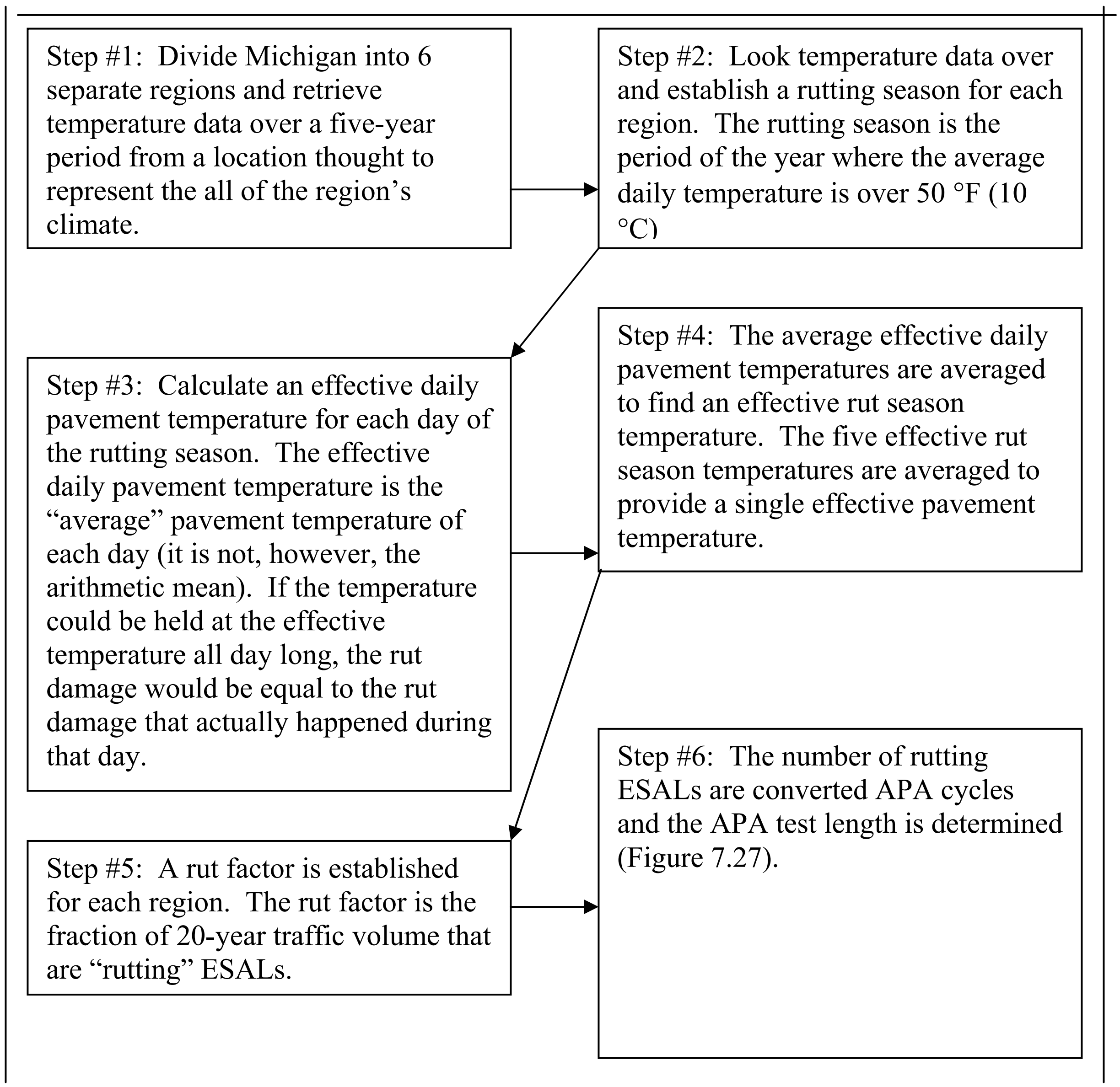


Figure 7.28 A Summary of the Steps Taken to find the Amount of Rutting ESALs during a Superpave 20-year Design Life

7.7.1 Step #1. Dividing Michigan into Six Regions and Retrieving Weather Data for Each Region

Based on the size of the State of Michigan and the differing climates throughout the state, it was decided to develop different PBS criteria for different portions of the state where the climate may differ. This is because the rate that ruts form is very dependent on pavement temperature. Care was taken to try to keep all of each of the Michigan Department of Transportation regions in one region. The state was divided into six regions as shown in Figure 7.29. The sections include the Superior West, Superior East, North, Bay, Grand/Southwest, and University/Metro sections.

A location with a National Oceanic and Atmospheric Administration weather station near the center of each section was selected and weather data was collected over a five-year span (1997-2001) for each location. It is assumed that this location is representative of the entire section. The locations are shown in Figure 7.30.

7.7.2 Step #2. Establishing the Length of the Rutting Season for Each Region

The National Oceanic and Atmospheric Administration's web site was visited and the weather data, including daily minimum and maximum temperatures, was downloaded. The temperature data was reviewed and the length of the rutting season was estimated. The rutting season is the length of the season where temperatures are consistently above 50 °F (10 °C) and the length of each region's rutting seasons are shown in Table 7.22.

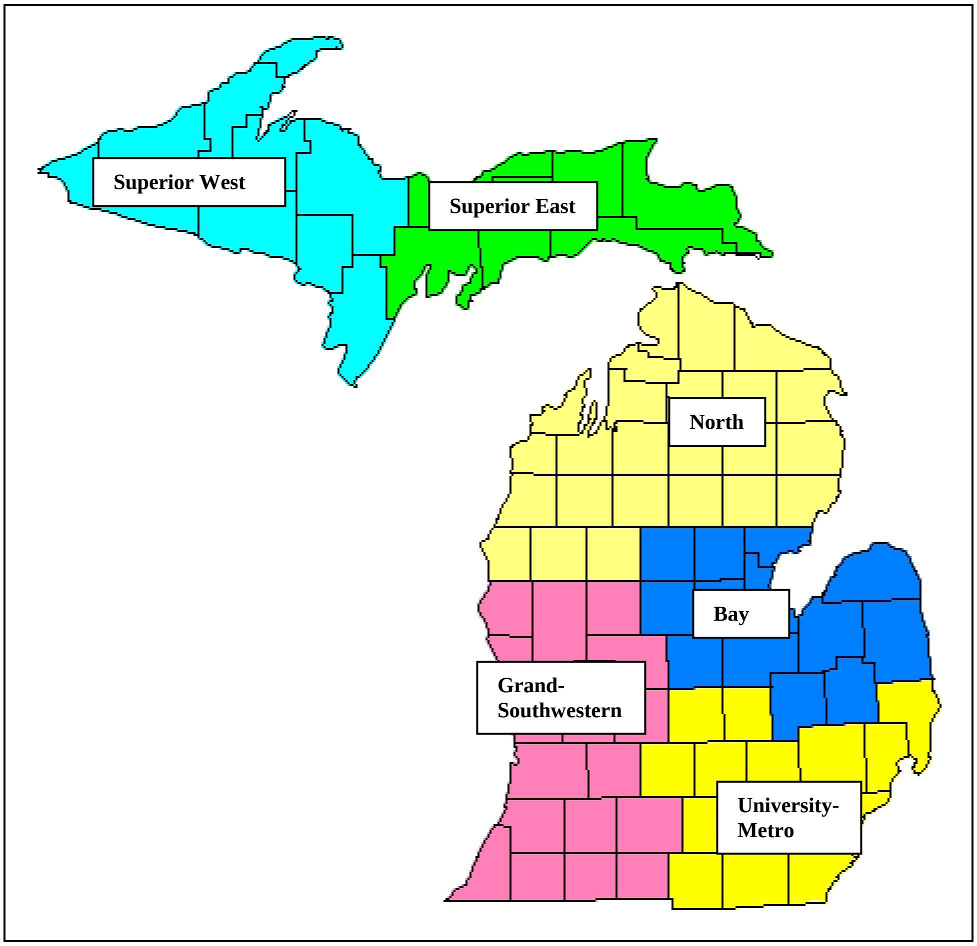


Figure 7.29 Six Regions of Michigan Chosen for PBS

Table 7.22 Length of Rutting Seasons in Each Region

Region	Dates of Rutting Season	Length of Rutting Season in Days
Superior West	April 1 – October 31	214
Superior East	April 1 – October 31	214
North	April 1 – October 31	214
Bay	March 15 – November 15	246
Grand-Southwestern	April 1 – October 31	214
University-Metro	March 1 – November 30	275

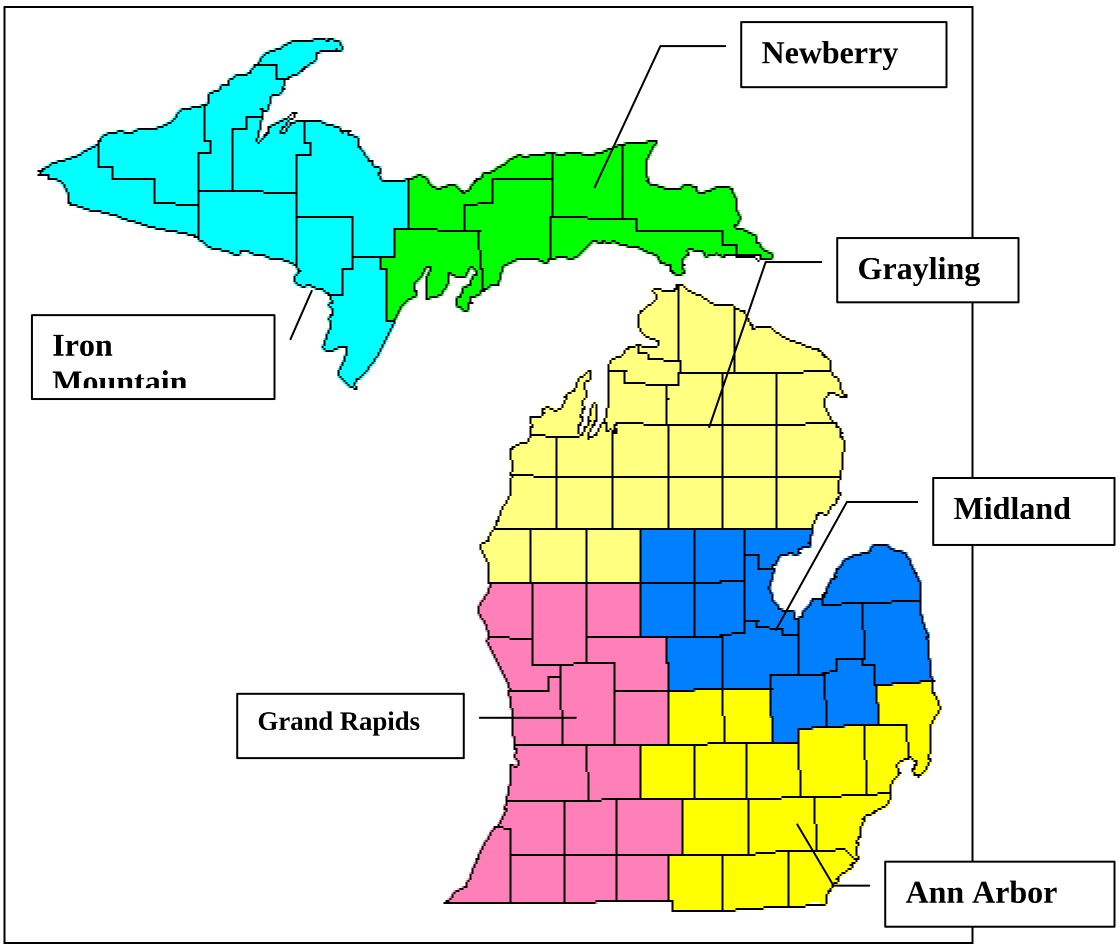


Figure 7.30 Locations Where Weather Data was Collected for Each Region

After establishing the rutting season, the minimum and maximum temperature from each day was recorded into Microsoft Excel and the pavement temperature at 20 mm depth corresponding to these temperatures was found using the equations established by Superpave (Asphalt Institute, 1996):

$$T_{Surface} = T_{Air} + 24.4 + 0.2289(latitude) - 0.00618(latitude)^2 \quad (7.4)$$

$$T_{@\,20\,mm\,depth} = T_{Surface}\,(1 - 0.063d + 0.007d^2 - 0.0004d^3) \quad (7.5)$$

where:

$T_{Surface}$ = Temperature at Pavement Surface (°C)

$T_{@\,20\,mm\,depth}$ = Temperature of Pavement 20 mm Beneath the Pavement Surface (°C)

d = Depth beneath Pavement Surface (mm)

It should be noted that this equation assumes that the sun is shining. It does not take into account cloudy days.

7.7.3 Step #3. Finding the Daily Effective Pavement Temperature

HMA during the rutting season is not always rut susceptible. During the night or on cooler days the rate that rutting occurs slows drastically. To help establish exactly how many ESALs were rutting ESALs each day it was decided that a daily effective pavement temperature would be calculated for each day. The daily effective pavement temperature is the "average" pavement temperature of each day (it is not, however, the arithmetic mean). It is hypothesized that if the temperature could be held at the effective temperature all day long, the rut damage would be equal to the rut damage that actually happened during that day.

HMA pavement rutting is a function of the HMA mixture stiffness. A stiffer HMA mixture recovers more shear deformation elastically then a less stiff mixture does. As an HMA mixture becomes warmer its stiffness decreases and its susceptibility to rutting increases (Shell, 1978). In this study, HMA rut damage is assumed to directly linked to HMA mixture stiffness.

Figure 7.31, reproduced from the Shell Pavement Design Guide (1978), shows a Mixture Stiffness (N/m^2) vs. Temperature (°C) relationship for a typical dense graded HMA with asphalt binder penetration of 50. The Rolling Thin Film Oven (RTFO) penetrations were found as stated in ASTM D5-95 and are summarized in Table 7.23. The Shell guide did not provide a equation to fit the data. The data was entered into a Microsoft Excel spreadsheet and a 5th-order polynomial was calculated by Microsoft Excel to produce an equation of a line that matched the Shell data. This equation is shown on Figure 7.31 and will be referred to as the Shell curve. A multi-step process was utilized to find the effective daily pavement temperature. First, the area below the Shell curve between the high and low daily pavement temperatures was found by integrating the Shell equation as follows:

$$\int_{T_{Low}}^{T_{High}} (29T^5 - 5936T^4 + 383782T^3 - 3559753T^2 - 483605726T + 13294493097)dT \tag{7.6}$$

$$\begin{aligned}
&= ((29/6)T_{High}{}^6 - (5936/5)T_{High}{}^5 + (383782/4)T_{High}{}^4 - (3559753/3)T_{High}{}^3 \\
&\quad - (483605726/2)T_{High}{}^2 + 13294493097T_{High}) - ((29/6)T_{Low}{}^6 - (5936/5)T_{Low}{}^5 \\
&\quad + (383782/4)T_{Low}{}^4 - (3559753/3)T_{Low}{}^3 - (483605726/2)T_{Low}{}^2 + 13294493097T_{Low}) \\
&= \mathit{Area\ Beneath\ Shell\ Curve}
\end{aligned}$$

where:

$\mathrm{T_{High}} = \text{Daily Maximum Temperature (°C)}$
$\mathrm{T_{Low}} = \text{Daily Mininum Temperature (°C)}$

After finding the area beneath the Shell curve the average daily temperature (T_{Mid}) was taken as the temperature that would divide the area beneath the Shell curve in half as follows:

$$0.5 * Total\ Area = \int_{T_{Low}}^{T_{Mid}} (29T \ -5936T^4 + 383782T^3 - 3559753T^2 - 483605726T + 13294493097)dT \tag{7.7}$$

$Sovling\ \ for\ T_{Mid}\ Yields\ the\ Following\ Equation:$

$$\begin{aligned} &0.5 * Total\ Area - \ ((29/6)T_{Low}{}^6 - (5936/5)T_{Low}{}^5 + (383782/4)T_{Low}{}^4 - (3559753/3)T_{Low}{}^3 \\ &- (483605726/2)T_{Low}{}^2 + 13294493097T_{Low}) = ((29/6)T_{Mid}{}^6 - (5936/5)T_{Mid}{}^5 \\ &\quad + (383782/4)T_{Mid}{}^4 - (3559753/3)T_{Mid}{}^3 - (483605726/2)T_{Mid}{}^2 + 13294493097T_{Mid}) \end{aligned} \tag{7.8}$$

where:

$\mathrm{T_{Low}} = \mathrm{Daily\ Minimum\ Temperature}\left(°\mathrm{C}\right)$
$\mathrm{T_{Mid}} = \mathrm{The\ Temperature\ that\ Divides\ the\ Total\ Area\ in\ Half}\ \left(°\mathrm{C}\right)$

The preceding equation, a 6th-order polynomial was solved for T_{Mid} using MathCad. T_{Mid} is the daily effective pavement temperature, meaning that if a pavement was set at T_{Mid} for the entire day the rutting damage would be equal to that which happens when the pavement temperature fluctuates between T_{Max} and T_{Min}. The daily effective temperature was calculated for every day of the rutting seasons in 1997 through 2001.

Table 7.23 HMA Project RTFO Aged Penetrations (ASTM D5-95)

HMA Project	RTFO Aged Penetration (0.1 mm)
Brimley M-28	73.0
Elk Rapids US-31	65.0
Monroe US-24	57.0
Brooklyn M-50	56.0
Grayling US-27	66.0
Indian River I-75	66.0
Lansing M-43	36.0
Auburn Hills I-75	41.0
Clarkston I-75	37.0
Saginaw I-75	36.0

7.7.4 Step #4. Finding an Average Effective Pavement Temperature Based on Five Years of Weather Data From Each Region

Now that the effective daily pavement temperatures (T_{Mid}) are known it is necessary to know the effective pavement temperature for the entire rutting season. This is done as follows:

$$T_{Eff\ Rut\ Season} = \left(\sum_{n=1}^{n} T_{Mid}\right) / n \qquad (7.9)$$

where:

n = Number of Rutting Days per Year

T_{Mid} = Daily Effective Pavement Temperature (°C)

If the temperature during the entire rut season was held at $T_{eff\ Rut\ Season}$ it is hypothesized that the amount of rut damage occurring would be equal to the rut damage that actually occurred at actual pavement temperatures during the rutting season. The $T_{Eff\ Rut\ Season}$ was found for five rutting seasons. The average effective pavement rutting temperature over five years is

$$T_{Eff,\ five\ years} = \left(\sum_{n=1}^{n} T_{effective\ rut\ season\ temp}\right)/n \qquad (7.10)$$

where:

n = Number of Rutting Seasons (n = 5)

$T_{eff\ rut\ season}$ = Effective Rut Season Pavement Temperature (°C)

designated as $T_{Eff,\ five\ years}$ and is found as follows:

$T_{Eff,\ five\ years}$ is the average effective pavement temperature over five rutting seasons.

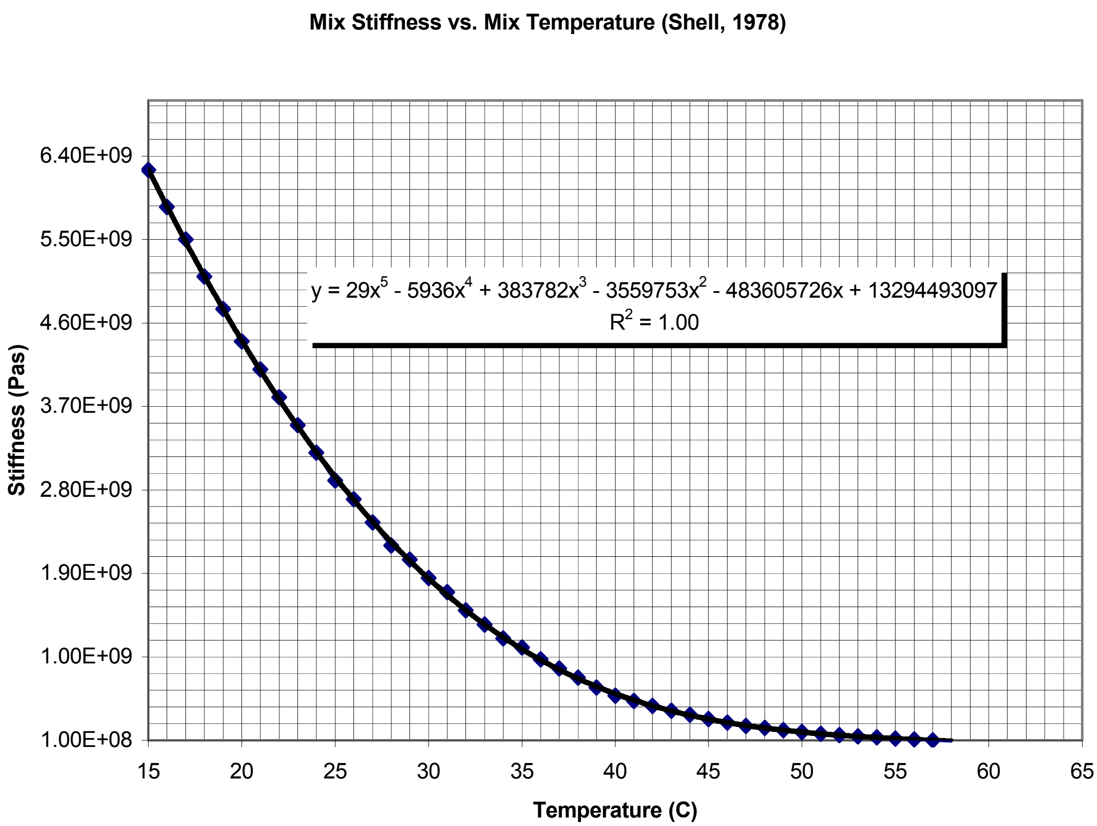


Figure 7.31 HMA Mixture Stiffness vs. Temperature (Shell, 1978)

7.7.5 Step #5 and #6. Establishing a Rut Factor and the Amount of Rutting ESALs During a Superpave 20-year Design Life

The effective pavement rutting temperature calculated from a five-year span of weather data can be used to determine approximately how many of the Superpave 20-year design ESALs are actually rutting ESALs. Once this is done, pavement performance can be predicted using APA data in conjunction with the method of converting APA data to field data presented in Section 5.3. To do this a rut factor is established. A rut factor is defined as follows:

$$Rutting_{ESALs} = Total_{ESALs} * \left(Rutting\ Days / 365 \right) * RF \tag{7.11}$$

where:

$$\begin{aligned}
&\text{Total}_{\text{ESALs}} &&= \text{Superpave 20-year traffic volume} \left(\text{80-kN ESALs} \right) \\
&\text{Rutting Days} &&= \text{Length of Rutting Season} \\
&\text{RF} &&= \text{The Fraction of ESALs during the Rutting Season that are Rutting ESALs}
\end{aligned}$$

To establish the Rutting Factor (RF) a graph was developed based upon the Shell (1978) HMA mixture stiffness vs. Temperature Data (Figure 7.32). Once again, Microsoft Excel was used to fit a curve to a line. Using this 3^{rd}-order polynomial equation, Equation 5.12 taken from Figure 7.32, the rut factor corresponding to the effective HMA mixture stiffness can be found.

$$HMA_{Eff\ Mix\ Siffness} = 2310273548(RF)^3 + 6175895490(RF)^2 - 5829181398(RF) \qquad (7.12)$$

where:

$HMA_{Eff\ Mix\ Stiffness}$ = The Effective Mixture Stiffness Corresponding to $T_{Eff,\ five\ years}$ found using Equation in Figure 5.10 (Shell, 1978)

RF = The Rutting Factor

The Rut Factor, RF, can be used to find the amount of rutting ESALs occurring in the Superpave 20-year design life using Equation 7.11.

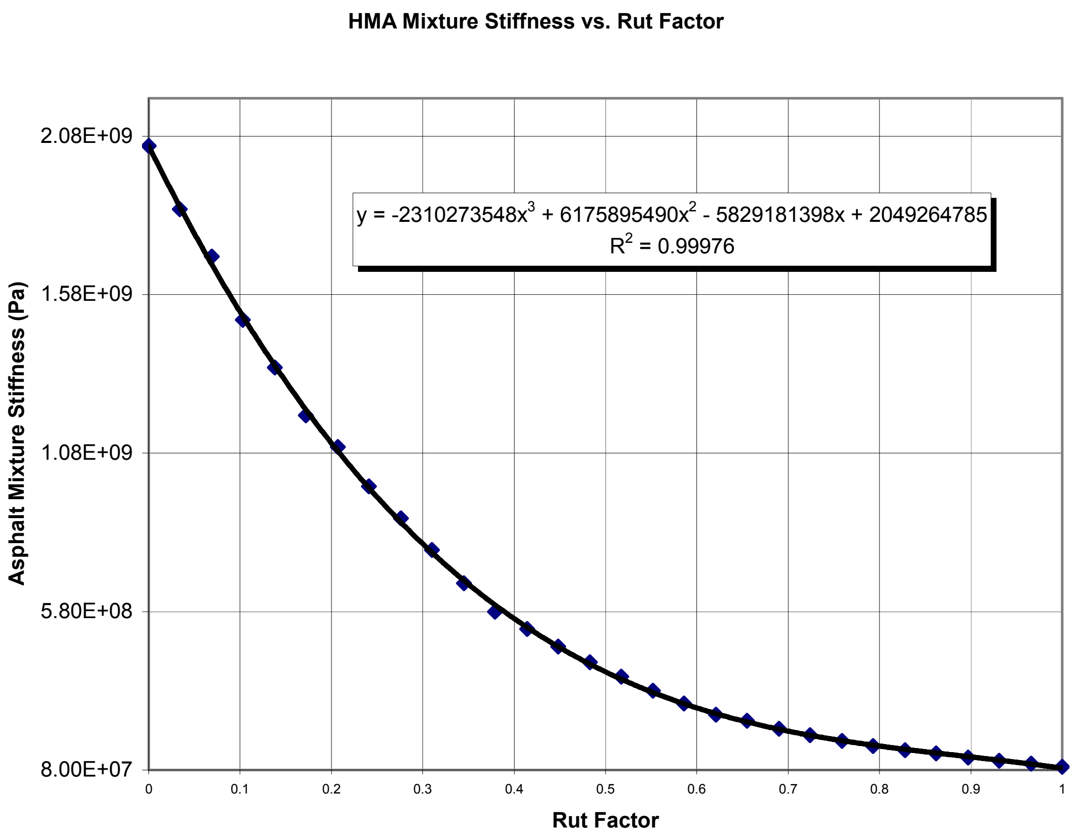


Figure 7.32 HMA Mixture Stiffness vs. Rut Factor (from Shell, 1978)

7.8 A Preliminary PBS for Michigan

As stated in the previous section a PBS based upon APA data must include an APA rut depth failure criterion as well as the test length representing the HMA pavements design life, in terms of ESALs. Sections 7.7 presents a method of converting APA rut depth and cycles to in-service pavement rut depth and ESALs. In section 7.8, a method of finding the amount of rutting ESALs that occur in the Superpave 20-year design life was presented. Based upon these findings, Performance Based APA Specifications were created for all six Michigan regions. As mentioned, a PBS based on APA data must include both a test length (in terms of APA cycles) and a failure rut depth criterion. The rut depth criterion is summarized first, followed by the test length.

The failure criterion for an APA specimen was set at 7 mm based upon data gathered at WesTrack, but this criterion should be adjusted to consider APA testing variability. This rut criterion adjustment is based upon the following factors (Williams and Prowell, 1999):

- The level of confidence,
- The variance or standard deviation,
- The sample size,
- And the specification limit.

A method established by Williams and Prowell (1999) to develop an APA pass/fail rut depth criteria taking the preceding factors into account. The rut depth criterion is set using the small-sample confidence for a one-tail test (Mendenhall and Sincich, 1989) as follows:

$$Maximum\ Rut\ Depth = y + t_{Alpha}(S/\sqrt{n}) \tag{7.13}$$

where:

y = mean APA rut depth at 8000 cycles (mm)

t_{alpha} = confidence limit

S = sample standard deviation (mm)

n = number of APA Specimens in sample

A maximum APA mean rut depth of three APA specimens can be calculated by rearranging Equation 5.13 and substituting values into the equation as follows:

$$\begin{aligned} y &= Maximum\ Rut\ Depth - t_{alpha}(S/\sqrt{n}) \\ &= 7\,mm - 2.353(1\,mm/\sqrt{3}) \\ &= 5.64\,mm \end{aligned} \tag{7.14}$$

where:

7mm = maximum allowable APA rut depth based on Figure 5.3

2.353 = $t_{0.05}$ (Mendenhall and Sincich, 1989)

1mm = standard deviation based on 7 mm rut depth (Figure 5.12)

3 = sample sized proposed to be used in an APA specification

An APA average rut depth of 5.64 mm ensures with 95% confidence that the HMA being tested does not rut more then 7 mm in the APA. This is based on a sample size of three APA specimens.

The test length for the Michigan PBS is calculated using Equation 7.11. A preliminary Performance Based APA specification for Michigan HMA pavements is presented in Table 7.24.

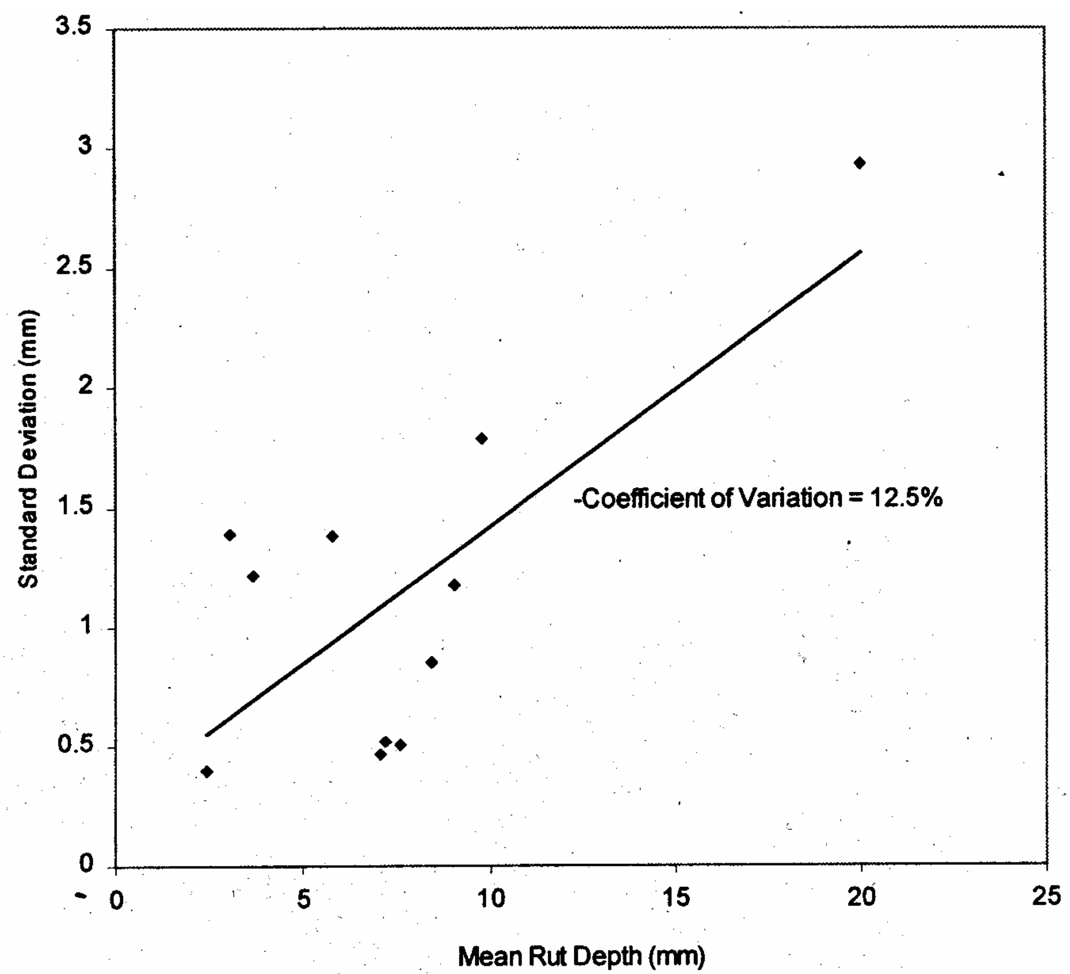


Figure 7.33 Mean Rut Depth vs. Standard Deviation Based on Data Taken from Westrack , Data Point = 3 Specimens (Williams and Prowell, 1999)

Table 7.24 A Preliminary Performance Based APA Specification for Michigan

Region	Traffic Level	Rutting Days per Year	Rutting Factor	18-Kip ESALs on Rutting Days	Rutting ESALs	APA Test Length (APA Cycles)	APA Failure Criteria (mm)[1]
Superior West	E3	214	0.084	1,783,333.00	149,800.00	1,158.00	5.64
	E10			5,944,444.00	499,333.00	3,859.00	5.64
	E30			17,833,333.00	1,498,000.00	11,577.00	5.64
Superior East	E3	214	0.071	1,783,333.00	126,617.00	978.00	5.64
	E10			5,944,444.00	422,056.00	3,262.00	5.64
	E30			17,833,333.00	1,266,167.00	9,785.00	5.64
North	E3	214	0.067	1,783,333.00	119,483.00	923.00	5.64
	E10			5,944,444.00	398,278.00	3,078.00	5.64
	E30			17,833,333.00	1,194,833.00	9,234.00	5.64
Bay	E3	246	0.167	2,050,000.00	342,350.00	2,646.00	5.64
	E10			6,833,333.00	1,141,167.00	8,819.00	5.64
	E30			20,500,000.00	3,423,500.00	26,457.00	5.64
Grand/Southwest	E3	214	0.201	1,783,333.00	358,450.00	2,770.00	5.64
	E10			5,944,444.00	1,194,833.00	9,234.00	5.64
	E30			17,833,333.00	3,584,500.00	27,701.00	5.64
University/Metro	E3	275	0.147	2,291,667.00	336,875.00	2,603.00	5.64
	E10			7,638,889.00	1,122,917.00	8,678.00	5.64
	E30			22,916,667.00	3,368,750.00	26,034.00	5.64

[1]The APA Failure Criterion is Based on the Mean APA Rut Depth of Three APA Specimens

7.9 Asphalt Pavement Analyzer Test Results

Ten separate 9.5-mm nominal maximum aggregate wearing course mixtures were sampled during the 2000 construction season. The HMA was sampled from throughout the State of Michigan and included all four Superpave traffic levels (i.e. E1, E3, E10, and E30). These project's mix designs were recreated in the laboratory and tested in the APA to determine the following:

- To determine the usefulness of the empirical model developed in the previous section,
- To determine the effect that changing asphalt content and air voids has on APA performance,
- Develop a regression model to predict APA rut depth, and
- Perhaps most importantly, the APA data presented in this Chapter will be correlated with future in-service pavement performance to assess the APA's usefulness in predicting the performance of Michigan HMA pavements.

Two types of APA data will be analyzed. The first, the APA rut depth at 8000 cycles, is used industry wide as a indication of whether or not an HMA mixture will perform in the field. The second is the amount of APA cycles needed to achieve a rut depth of 7 mm. As shown previously, a 7 mm APA rut depth correlated with an in-service HMA rutting failure . The previous section also presents a method of converting APA cycles to 80-kN ESALs. Based on this, it is thought that the number of APA cycles needed to achieve a 7 mm rut depth can be converted to how many ESALs an in-service pavement could withstand before failure.

7.10 APA Test Results

This section summarizes the APA results from HMA specimens created using materials from 10 Michigan HMA paving projects. Two performance measures are presented. Section 7.7.1 summarizes the APA rut depths at 8000 cycles, a pass/fail criterion used throughout the United States to identify rut prone HMA. Section 7.7.2 presents a performance measure that has not been documented in the past. It is the APA cycles needed to create a 7 mm rut (or APA cycles to failure). This is the criterion used in the Performance Based Specification (PBS) presented in the previous section. The APA cycles to failure results can be used to access the feasibility of this PBS. In addition to the results shown in this chapter, APA rut depths at 1000 cycle increments for all 210 test specimens is presented in Appendix B.

7.10.1 APA Rut Depths at 8000 Cycles Results

Most State Highway Agencies that utilize the APA set a pass/fail criterion for the APA rut depth at 8000 cycles. The APA rut depth at 8000 cycles was recorded for HMA from all 10 projects and the data is summarized in Table 7.25 through Table 7.34. Each cell in the following test matrices contains the mean APA rut depth at 8000 cycles of three APA specimens. The standard deviation of the three specimens was calculated and is shown in parenthesis beneath the mean rut depth.

Table 7.25 APA Rut Depths at 8000 Cycles, Brimley M-28

Project Name:	**Brimley M-28**	**Air Voids**		
Traffic Level:	**5 E 1**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	10.25 (0.77)	11.21 (0.6)
	Opt. AC	8.95 (0.73)	10.56 (0.73)	13.69 (1.17)
	Opt. AC +0.5%	9.27 (1.71)	12.94 (0.68)	**N/A**

Table 7.26 APA Rut Depths at 8000 Cycles, Elk Rapids US-31

Project Name:	**Elk Rapids US-31**	**Air Voids**		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	7.22 (2.18)	11.81 (1.40)
	Opt. AC	4.85 (0.58)	7.76 (1.17)	12.61 (3.00)
	Opt. AC +0.5%	7.02 (3.03)	8.96 (2.04)	**N/A**

Table 7.27 APA Rut Depths at 8000 Cycles, Monroe US-24

Project Name:	**Monroe US-24**	**Air Voids**		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	4.17 (0.76)	3.92 (1.12)
	Opt. AC	3.5 (1.14)	4.18 (0.28)	6.03 (0.80)
	Opt. AC +0.5%	5.19 (1.35)	8.16 (0.74)	**N/A**

Table 7.28 APA Rut Depths at 8000 Cycles, Brooklyn M-50

Project Name:	Brooklyn M-50	Air Voids		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	6.34 (2.00)	7.33 (1.26)
	Opt. AC	5.17 (1.47)	8.38 (0.54)	7.16 (1.09)
	Opt. AC +0.5%	6.32 (0.83)	8.12 (1.40)	**N/A**

Table 7.29 APA Rut Depths at 8000 Cycles, Lansing, US-43

Project Name:	Lansing US-43	Air Voids		
Traffic Level:	**5 E 10**	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	**N/A**	3.23 (0.45)	6.76 (0.71)
	Opt. AC	3.38 (0.64)	5.57 (1.26)	8.96 (0.94)
	Opt. AC +0.5%	3.44 (0.08)	5.47 (2.10)	**N/A**

Table 7.30 APA Rut Depths at 8000 Cycles, Indian River I-75

Project Name:	Indian River I-75	Air Voids		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	9.33 (2.42)	10.88 (0.47)
	Opt. AC	8.31 (1.20)	11.24 (1.89)	12.63 (1.94)
	Opt. AC +0.5%	11.56 (0.21)	14.32 (0.51)	**N/A**

Table 7.31 APA Rut Depths at 8000 Cycles, Grayling US-27

Project Name:	**Grayling US-27**	**Air Voids**		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	4.44 (0.38)	6.4 (2.06)
	Opt. AC	3.19 (0.26)	4.27 (1.45)	6.74 (1.11)
	Opt. AC +0.5%	3.89 (1.17)	5.62 (0.52)	**N/A**

Table 7.32 APA Rut Depths at 8000 Cycles, Auburn Hills I-75

Project Name:	**Auburn Hills I-75**	**Air Voids**		
Traffic Level:	**5 E 30**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	5.24 (2.15)	6.1 (1.00)
	Opt. AC	4.88 (0.79)	7.86 (1.77)	9.69 (1.28)
	Opt. AC +0.5%	5.38 (1.02)	7.63 (0.93)	**N/A**

Table 7.33 APA Rut Depths at 8000 Cycles, Clarkston I-75

Project Name:	**Clarkston I-75**	**Air Voids**		
Traffic Level:	**5 E 30**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	6.24 (2.40)	10.81 (0.08)
	Opt. AC	6.23 (1.64)	9.04 (2.04)	14.54 (1.72)
	Opt. AC +0.5%	6.44 (2.64)	11.34 (0.80)	**N/A**

Table 7.34 APA Rut Depths at 8000 Cycles, Saginaw I-75

Project Name:	**Saginaw I-75**	**Air Voids**		
Traffic Level:	5 E 30	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	5.53 (1.07)	11.19 (0.27)
	Opt. AC	4.16 (1.92)	6.5 (1.67)	11.37 (0.24)
	Opt. AC +0.5%	4.89 (0.99)	8.21 (2.12)	**N/A**

In addition to recording all of the APA rut depth data from each individual project the data from each traffic level was averaged and is presented in Table 7.35 through Table 7.37. This was done so the mean APA rut depth and standard deviation at different traffic levels can be analyzed and trends in the data can be identified. An average for 5E1 mixtures is not presented since there is only was only one 5E1 HMA mixture tested, Brimley M-28.

Table 7.35 Average APA Mean Rut Depths, 5E3 HMA Mixtures

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	5.91 (1.65)	7.69 (1.26)
	Opt. AC	4.51 (1.06)	6.77 (0.66)	8.6 (1.63)
	Opt. AC +0.5%	6.18 (1.74)	8.41 (1.39)	**N/A**

Table 7.36 Average APA Mean Rut Depths, 5E10 HMA Mixtures

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	5.67 (1.08)	8.01 (1.08)
	Opt. AC	4.96 (0.70)	7.03 (1.53)	9.44 (1.33)
	Opt. AC +0.5%	6.3 (0.49)	8.47 (1.04)	**N/A**

Table 7.37 Average APA Mean Rut Depths, 5E30 HMA Mixtures

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 30**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	5.67 (1.87)	9.37 (0.45)
	Opt. AC	5.09 (1.45)	7.8 (1.83)	11.87 (1.08)
	Opt. AC +0.5%	5.57 (1.55)	9.06 (1.28)	**N/A**

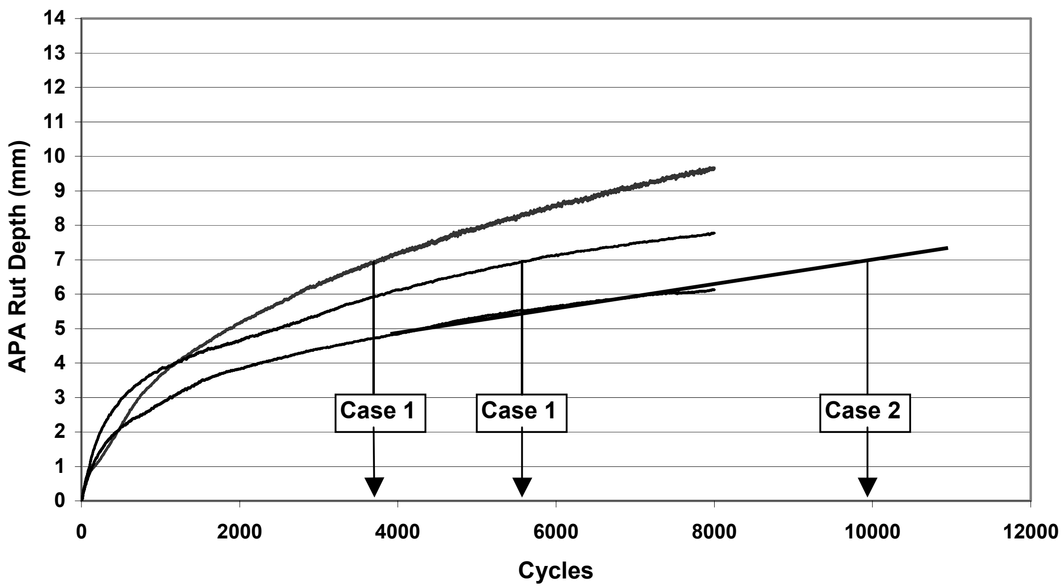


Figure 7.34 Method Used to Find the Number of APA Cycles Until Failure The Amount of APA Cycles to Reach the Failure Rut Depth in the Asphalt Pavement Analyzer

7.10.2 The Amount of APA Cycles to Reach the Failure Rut Depth in the Asphalt Pavement Analyzer

The number of cycles in the APA to achieve a rut depth of 7mm (or APA cycles to failure) is important in this study to test the effectiveness of the empirical pavement prediction model proposed in the previous chapter. Recording the APA cycles until failure was done in two different ways. First, if the specimen rutted more then 7 mm in the 8000 cycle test, the number of APA cycles where the specimen rutted 7mm was determined (Figure 7.34) and is illustrated as Case 1. Case 2, where the APA specimen did not rut 7mm, the APA curve was extrapolated out to 7mm rut depth (Figure 7.34). This extrapolation was done by extending the creep curve outwards to 7 mm. The creep portion of the APA curve is assumed to be where permanent shear deformation is taking place. The initial part of the curve is the consolidation curve and this is assumed to be where the specimen rutting due to densification beneath the loaded wheel. These two parts of the APA curve are shown in Figure 7.35.

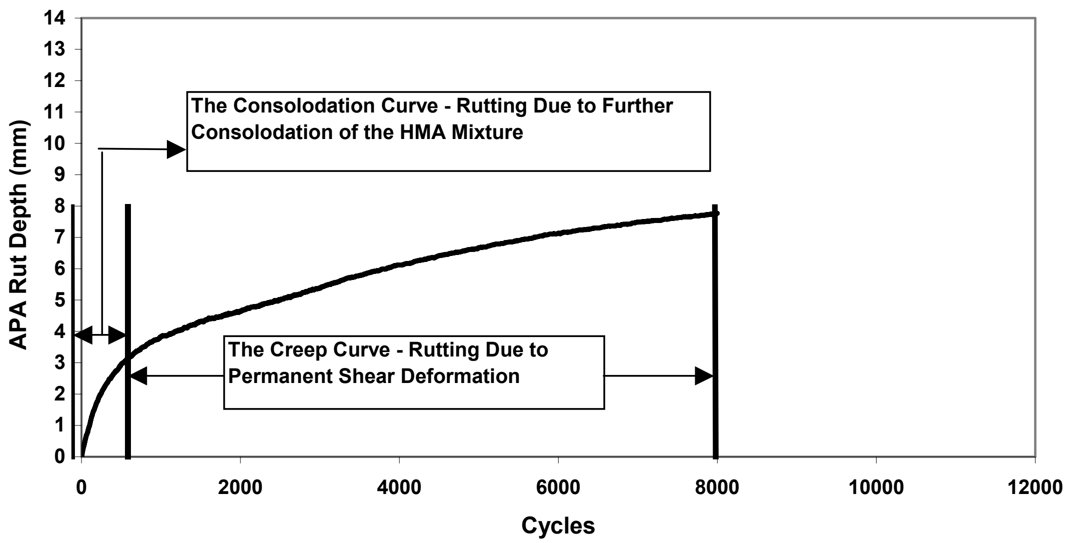


Figure 7.35 The Two Different Portions of the APA Rut Depth vs. Cycles Curve

The APA cycles until failure data was determined for each of the 10 projects and the data is summarized in Table 7.38 and

Table 7.47. The average APA cycles to failure of the three APA specimens and the standard deviation is presented. The average data from each traffic level was averaged together and is presented in Table 7.48 through Table 7.50.

Table 7.38 APA Cycles to Failure, Brimley M-28

Project Name:	Brimley M-28	Air Voids		
Traffic Level:	**5 E 1**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	3,136 (1,117)	1,796 (791)
	Opt. AC	2,639 (704)	2,265 (796)	882 (110)
	Opt. AC +0.5%	5,075 (2,817	1,608 (441)	**N/A**

Table 7.39 APA Cycles to Failure, Elk Rapids US-31

Project Name:	Elk Rapids US-31	Air Voids		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	7,997 (4,666)	2,584 (1,214)
	Opt. AC	12,033 (1,320)	7,404 (1,995)	2,513 (1,922)
	Opt. AC +0.5%	9,163 (6,836)	5,582 (1,668)	**N/A**

Table 7.40 APA Cycles to Failure, Monroe US-24

Project Name:	Monroe US-24	Air Voids		
Traffic Level:	**5 E 3**	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	**N/A**	14,500 (7,079)	24,067 (7,234)
	Opt. AC	24,333 (11,594)	21,433 (7,151)	6,770 (4,863)
	Opt.AC +0.5%	15,000 (5,587)	5,997 (599)	**N/A**

Table 7.41 APA Cycles to Failure, Brooklyn M-50

Project Name:	Brooklyn M-50	Air Voids		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	10,903 (6,704)	6,506 (1,782)
	Opt. AC	18,500 (9,224)	5,225 (1,203)	7,632 (2,063)
	Opt. AC +0.5%	9,630 (1,949)	5,574 (2,169)	**N/A**

Table 7.42 APA Cycles to Failure, Lansing, US-43

Project Name:	Lansing US-43	Air Voids		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	28,033 (6,626)	8,967 (404)
	Opt. AC	27,833 (5,014)	15,100 (9,700)	4,493 (1,240)
	Opt. AC +0.5%	24,667 (4,561)	16,769 (12,965)	**N/A**

Table 7.43 APA Cycles to Failure, Indian River

Project Name:	Indian River I-75	Air Voids		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	5,156 (2,208)	2,832 (833)
	Opt. AC	6,227 (1,563)	3,032 (1,701)	1,919 (1,744)
	Opt. AC +0.5%	2,457 (217)	861 (328)	**N/A**

Table 7.44 APA Cycles to Failure, Grayling US-27

Project Name:	Grayling US-27	Air Voids		
Traffic Level:	5 E 10	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	14,800 (2,163)	10,616 (7,188)
	Opt. AC	27,433 (6,757)	25,033 (16,459)	8,753 (3,185)
	Opt. AC +0.5%	21,133 (11,208)	12,500 (2,252)	N/A

Table 7.45 APA Cycles to Failure, Auburn Hills I-75

Project Name:	Auburn Hills I-75	Air Voids		
Traffic Level:	5 E 30	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	24,100 (25,210)	10,408 (3,077)
	Opt. AC	15,836 (5,016)	6,553 (3,259)	42,498 (747)
	Opt. AC +0.5%	11,900 (3,812)	6,734 (1,767)	N/A

Table 7.46 APA Cycles to Failure, Clarkston I-75

Project Name:	Clarkston I-75	Air Voids		
Traffic Level:	5 E 30	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	9,715 (6,410)	2,904 (941)
	Opt. AC	10,435 (4,996)	5,296 (1,863)	2,687 (422)
	Opt. AC +0.5%	12,656 (10,836)	2,901 (489)	N/A

Table 7.47 APA Cycles to Failure, Saginaw I-75

Project Name:	**Saginaw I-75**	**Air Voids**		
Traffic Level:	**5 E 30**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	12,033 (3,782)	2,313 (549)
	Opt. AC	21,933 (12,574)	9,305 (3,867)	2,238 (545)
	Opt. AC +0.5%	15,367 (6,269)	6,076 (3,528)	**N/A**

Table 7.48 Average APA Cycles to Failure, 5E3 Mixtures

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 3**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	11,133 (6,150)	11,052 (3,410)
	Opt. AC	18,289 (7,379)	11,354 (3,450)	5,638 (2,949)
	Opt. AC +0.5%	11,264 (4,791)	5,718 (1,479)	**N/A**

Table 7.49 Average APA Cycles to Failure, 5E10 Mixtures

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	15,999 (3,666)	7,472 (2,808)
	Opt. AC	20,498 (4,445)	14,388 (9,287)	5,055 (2,056)
	Opt. AC +0.5%	16,086 (5,329)	10,043 (5,182)	**N/A**

Table 7.50 Average APA Cycles to Failure, 5E30 Mixtures

Project Name:	Average of All Projects	Air Voids		
Traffic Level:	**5 E 30**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	15,283 (11,801)	5,208 (1,522)
	Opt. AC	16,068 (7,529)	7,051 (2,996)	2,725 (571)
	Opt. AC +0.5%	13,308 (6,972)	5,237 (1,928)	**N/A**

7.11 Predicted HMA Rutting Performance

In the previous section, an empirical rut prediction model was developed based on upon APA data. Based on the model, a performance based APA specification was developed for Michigan HMA pavements (Table 7.24). To help determined the usefulness of this PBS, the APA data from the ten Michigan projects was used to predict the amount of ESALs the in-service pavements could withstand before failure, which is taken as a 10 mm downward rut depth. The following equation was used to convert the APA cycles to failure data from Section 7.11.2 into ESALs to failure. The equation is simply Equation 7.11 solved for the total amount of ESALs.

$$ESALs_{Failure} = ((APA\,Cycles\,to\,7mm\,Rut) * (129.9\,Rutting\,ESALs / Cycle)) / (RS * RF / 365) \qquad (7.15)$$

where:

$ESALs_{Failure}$ = Amount of ESALs Until Rutting Failure
APA Cycles to 7mm Rut = Taken From the Data in Section 6.2.2
RS = Length of Rutting Season in Days
RF = The Fraction of the Total ESALs where Rutting Takes Place

The rutting factor and length of rutting season are the values as presented previously for the region where the pavement was constructed.

The data for each of the 10 projects follows in Table 7.51 through Table 7.60. Both the average number of ESALs and the standard deviation of the three test specimens are presented. The average predicted amount of ESALs to pavement rutting failure for each traffic level follows in Table 7.61 through Table 7.63.

These results can be compared to PBS specification in Table 7.24. Also, these results can be compared with the actual future pavement performance of the ten projects to access the accuracy of the rut prediction model presented.

Table 7.51 ESALs To Pavement Rutting Failure, Brimley M-28

Project Name:	**Brimley M-28**	**Air Voids**		
Traffic Level:	**5 E 1**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	11,127,850 (1,162,061)	4,917,973 (1,807,515)
	Opt. AC	7,717,098 (1,792,079)	6,286,850 (1,582,648)	2,772,081 (339,011)
	Opt. AC +0.5%	12,587,222 (3,726,886)	5,123,928 (1,351,201)	**N/A**

Table 7.52 ESALs To Pavement Rutting Failure, Elk Rapids US-31

Project Name:	Elk Rapids US-31	Air Voids		
Traffic Level:	5 E 3	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	2,081,988 (694,638)	739,520 (286,225)
	Opt. AC	4,137,190 (202,194	2,557,291 (603,311)	925,805 (591,695)
	Opt. AC +0.5%	3,692,640 (1,573,765)	1,724,518 (464,586)	N/A

Table 7.53 ESALs To Pavement Rutting Failure, Monroe US-24

Project Name:	Monroe US-24	Air Voids		
Traffic Level:	5 E 3	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	15,794,726 (7,753,705)	30,051,812 (7,212,404)
	Opt. AC	28,083,726 (13,552,466)	25,073,346 (8,386,249)	10,035,906 (2,487,114)
	Opt. AC +0.5%	20,095,270 (2,317,234)	6,932,868 (657,943)	N/A

Table 7.54 ESALs To Pavement Rutting Failure, Brooklyn M-50,

Project Name:	Brooklyn M-50	Air Voids		
Traffic Level:	5 E 3	4%	8%	12%
Asphalt Content	Opt. AC-0.5%	N/A	15,326,749 (4,812,627)	8,273,465 (1,375,025)
	Opt. AC	25,725,073 (4,445,080)	6,215,850 (1,394,268)	9,864,666 (919,598)
	Opt. AC +0.5%	11,110,651 (2,240,597)	5,925,368 (2,054,116)	N/A

Table 7.55 ESALs To Pavement Rutting Failure, Lansing US-43

Project Name:	Lansing US-43	Air Voids		
Traffic Level:	5 E 10	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	35,863,802 (2,638,057)	10,347,500 (229,225)
	Opt. AC	31,797,833 (5,502,958)	13,331,687 (3,793,910)	5,765,075 (803,736)
	Opt. AC +0.5%	27,901,540 (4,717,915)	13,947,447 (5,909,629)	N/A

Table 7.56 ESALs To Pavement Rutting Failure, Indian River I-75

Project Name:	Indian River I-75	Air Voids		
Traffic Level:	5 E 10	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	1,627,628 (703,015)	831,120 (95,991)
	Opt. AC	2,189,019 (407,359)	791,989 (224,849)	777,108 (458,872)
	Opt. AC +0.5%	808,633 (71,011)	251,210 (70,713)	N/A

Table 7.57 ESALs To Pavement Rutting Failure, Grayling US-27

Project Name:	Grayling US-27	Air Voids		
Traffic Level:	5 E 10	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	4,629,579 (303,077)	3,776,744 (2,285,606)
	Opt. AC	9,835,981 (1,220,251)	6,187,432 (1,843,494)	2,997,212 (1,022,715)
	Opt. AC +0.5%	7,433,009 (3,542,778)	4,276,849 (656,667)	N/A

Table 7.58 ESALs To Pavement Rutting Failure, Auburn Hills I-75

Project Name:	Auburn Hills I-75	Air Voids		
Traffic Level:	5 E 30	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	34,208,874 (25,736,628)	12,581,437 (3,490,887)
	Opt. AC	20,695,783 (2,755,076)	8,764,118 (2,763,959)	5,320,555 (294,112)
	Opt. AC +0.5%	14,778,234 (3,992,948)	7,403,582 (1,681,649)	N/A

Table 7.59 ESALs To Pavement Rutting Failure, Clarkston I-75

Project Name:	Clarkston I-75	Air Voids		
Traffic Level:	5 E 30	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	11,537,578 (7,509,77)	3,039,312 (641,280)
	Opt. AC	13,443,110 (5,063,938)	5,457,782 (1,167,227)	1,864,872 (409,061)
	Opt. AC +0.5%	16,836,240 (11,734,322)	3,539,348 (465,714)	N/A

Table 7.60 ESALs To Pavement Rutting Failure, Saginaw I-75

Project Name:	Saginaw I-75	Air Voids		
Traffic Level:	5 E 30	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	N/A	14,746,943 (3,824,018)	2,876,448 (378,652)
	Opt. AC	20,132,829 (7,039,281)	9,163,702 (2,242,891)	2,621,388 (621,809)
	Opt. AC +0.5%	14,952,376 (2,427,327)	7,048,202 (4,071,271)	N/A

Table 7.61 The Average Amount of ESALs to Pavement Rutting Failure, 5E3

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 3**	4%	8%	12%
Asphalt Content	Opt. AC - 0.5%	**N/A**	11,067,821 (4,420,323)	13,021,599 (2,947,885)
	Opt. AC	19,315,330 (6,066,580)	11,282,162 (3,461,276)	6,942,126 (1,332,802)
	Opt. AC +0.5%	11,632,854 (2,043,865)	4,860,918 (1,058,882)	**N/A**

Table 7.62 The Average Amount of ESALs to Pavement Rutting Failure, 5E10

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 10**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC – 0.5%	**N/A**	14,040,336 (1,214,716)	4,985,121 (870,274)
	Opt. AC	14,607,611 (2,376,856)	6,770,369 (1,954,084)	3,179,798 (761,774)
	Opt. AC +0.5%	12,047,727 (2,777,235)	6,158,502 (2,212,336)	**N/A**

Table 7.63 The Average Amount of ESALs to Pavement Rutting Failure, 5E30

Project Name:	**Average of All Projects**	**Air Voids**		
Traffic Level:	**5 E 30**	**4%**	**8%**	**12%**
Asphalt Content	Opt. AC - 0.5%	**N/A**	20,164,465 (12,356,806)	6,165,732 (1,503,606)
	Opt. AC	18,090,574 (4,952,765)	7,795,201 (2,058,026)	3,268,938 (441,661)
	Opt. AC +0.5%	15,522,283 (6,051,532)	5,997,044 (2,072,878)	**N/A**

7.12 Analysis of APA Test Results

The previous section outlined the results of APA testing of ten Michigan Hot-Mix Asphalt (HMA) paving projects. Three separate types of results were summarized:

1. The APA rut depth at 8000 cycles,
2. The APA cycles until failure (failure being a 7mm APA rut), and
3. The ESALs that the pavement is predicted to withstand until rutting failure is based upon the empirical rut prediction model described previously.

The results presented in the previous section are statistically analyzed in the following sections. Specifically, the following will be done:

1. The results will be analyzed to determine whether or not changes in asphalt content and air voids result in statistically different APA rut depths at 8000 cycles and APA cycles until failure. Past experience has shown that changing asphalt content and air void content does change rutting performance of in-service pavements. Because of this, it would be beneficial to know that the APA is sensitive to changes in these properties.
2. The average APA rut depths and standard deviations for each Superpave design level will be analyzed. It is of interest to know if HMA mixtures designed at different Superpave levels perform differently in the APA.
3. Lastly, regression models will be constructed to predict the APA rut depth using potentially ten different HMA material properties as predictor variables.

7.13 Statistical Analysis of the APA Rut Depth at 8000 Cycles Results

It is of interest to know whether or not the APA is sensitive to changes in asphalt content and air voids. To determine this, a test matrix was developed (Table 7.64) to analyze APA test results while varying HMA properties. These variations in HMA properties are similar to the variation that may occur in the field.

Table 7.64 Test Matrix Used for Testing Each HMA Project

X=Specimen	Air Voids (% of Total Volume)			
		4%	**8%**	**12%**
Asphalt Content (% of Total Mass)	**Low Asphalt Content (Opt. AC – 0.5%)**	**N/A**	**XXX**	**XXX**
	Optimum Asphalt Content	**XXX**	**XXX**	**XXX**
	Low Asphalt Content (Opt. AC – 0.5%)	**XXX**	**XXX**	**N/A**

This test matrix in Table 7.64 was used for each of the ten projects. To access whether or not the APA is sensitive to changes in air voids and asphalt content a statistical analysis was conducted. The goal of the statistical analysis was to determine if the changes in air voids and asphalt content resulted in statistically different APA performance. Two statistical methods were used to evaluate the effects of changes in the HMA properties to determine whether or not statistical differences exist. The two methods used were the Tukey's and Duncan's Multiple Range (DMR) Tests. These tests were used because they are effective when a factorial design is unbalanced. The test matrix in Table 7.1 is a 3^2 factorial design. It is an unbalanced design, however, because, the top left and bottom right cells of the test matrix contain no data. Both types of tests were conducted at the 95% (100-alpha) level of confidence.

The statistical analysis was performed using SAS statistical software. Using SAS, an analysis of variance (ANOVA) table was developed. The two treatments used in this model were asphalt content and air voids (i.e. rut depth = f(asphalt content, air voids)) where the properties were entered as categorical data (i.e. low, optimum, and high asphalt contents were entered into the program as 1, 2, and 3, respectively, while 4, 8, and 12% air voids was entered 1, 2, and 3). The ANOVA table includes the mean square error (MSE), an estimator of the sample variance, which is needed for both Tukey and DMR testing. SAS was used to conduct the Tukey and DMR tests.

Carmer and Swanson (1973) reported that the DMR test is a very effective test at detecting true differences in means. Montgomery (2001) reports that the Duncan procedure is quite powerful and is very effective at detecting differences in between means when real differences exist. Tukey's test is a more conservative test. The DMR test will be emphasized in the following statistical analysis for these reasons.

The results of this statistical analysis are shown in Table 7.65 and Table 7.66. In the Tables, HMA mixture types with the same letter performed the same, while HMA mixtures with different letters performed statistically different.

7.13.1 Analysis of the Effect of Asphalt Content on APA Rut Depth at 8000 Cycles

Upon examining Table 7.65, it can be seen that the DMR test detected five projects exhibited sensitivity to changing asphalt content. Two of these projects, Lansing M-43 and Auburn Hills I-75, did not rank the specimens correctly (i.e. rut depth did not increase with increasing air voids) and this is probably the result of error. After considering this, only three projects were sensitive to asphalt content. These three projects did not occur within any particular Superpave design level, so the effects of asphalt content on APA performance does not increase nor decrease with an increase in the mixture design level. All three of these projects showed a statistically greater rut depth when the asphalt content was high (Optimum AC + 0.5 %). This does lend credibility to the APA since high asphalt contents decrease stability in HMA mixtures. But since it only occurred for three out of ten projects, it is concluded, in general, that the APA rut depth at 8000 cycles is not statistically affected by changing asphalt content in general. This conclusion is based on differing the asphalt content by ±0.5% from optimum asphalt content. On the other hand, it could be possible that the APA is sensitive to changes in asphalt content. In this case, the seven HMA mixtures did not demonstrate sensitivity to changes in asphalt content because they were in fact not sensitive to changes in asphalt content (i.e. they exhibit the same rut performance at all three of the asphalt contents tested.)

Table 7.65 Analysis of the Effect of Asphalt Content on APA Rut Depth at 8000 Cycles, (α=0.05)

Superpave HMA Mixture Design Level	Project Location	Asphalt Content (% by Mass)	Tukey 95% Grouping	Duncan 95% Grouping
E1	Brimley, M-28	Low	A	A
		Optimum	A	A
		High	A	A
E3	Elk Rapids, US-31	Low	A	A
		Optimum	A	A
		High	A	A
	Monroe, US-24	Low	A	A
		Optimum	A	A
		High	B	B
	Brooklyn, M-50	Low	A	A
		Optimum	A	A
		High	A	A
E10	Lansing, M-43	Low	A B	A B
		Optimum	A	A
		High	B	B
	Indian River, I-75	Low	A	A
		Optimum	A	A
		High	B	B
	Grayling, US-27	Low	A	A
		Optimum	A	A
		High	A	A
E30	Auburn Hills, I-75	Low	A	A
		Optimum	B	B
		High	A B	A B
	Clarkston, I-75	Low	A	A
		Optimum	A	A
		High	A	A
	Saginaw, I-75	Low	A	A
		Optimum	A	A B
		High	A	B

7.13.2 Analysis of the Effect of Air Voids on APA Rut Depth at 8000 Cycles

The APA rut depth at 8000 cycles showed a significant sensitivity to changes in air void content (Table 7.66). According to the DMR groupings, only one of the projects exhibited no statistical changes in APA rut depth due to changes in air void content. The HMA mixture that showed no sensitivity to changes in air voids was Monroe, a HMA mixture that performed well for all but one asphalt content/air void combination (Table 7.27). The other nine projects that did demonstrate sensitivity to changes air voids showed the following:

- In three of the projects, the APA rut depths from specimens with 8 and 12% air voids were statistically different then the 4% specimens;
- In three of the projects, the APA rut depth from 12% air void specimens were statistically different then specimens prepared to 4 and 8% air voids; and
- In three of the projects, the APA rut depth from specimen's at all three air void levels was statistically different.

Research conducted by Linden and Van der Heide (1987) stressed the importance of proper compaction and concluded that degree of compaction is one of the main quality parameters of placed mixtures. Proper compaction reduced the amount of rutting due to consolidation and also provides increased aggregate interlock. Normally, a HMA pavement is compacted to approximately 7-8% air voids during construction. From Table 7.66 and the APA results described previously it can be seen that for most of the HMA mixtures, the 12% air void mixture performed statistically worse then the 4% and/or 8% mixtures. This is in line with Linden and Van der Heide's findings.

Table 7.66 Analysis of the Effect of Air Void Content on APA Rut Depth at 8000 Cycles, (α=0.05)

Superpave HMA Mixture Design Level	Project Location	Air Voids (% by Volume)	Tukey 95% Grouping	Duncan 95% Grouping
E1	Brimley, M-28	4	A	A
		8	B	B
		12	B	B
E3	Elk Rapids, US-31	4	A	A
		8	A	A
		12	B	B
	Monroe, US-24	4	A	A
		8	A	A
		12	A	A
	Brooklyn, M-50	4	A	A
		8	B	A B
		12	A B	B
E10	Lansing, M-43	4	A	A
		8	A	B
		12	B	C
	Indian River, I-75	4	A	A
		8	A	A
		12	A	B
	Grayling, US-27	4	A	A
		8	A	A
		12	B	B
E30	Auburn Hills, I-75	4	A	A
		8	A B	B
		12	B	B
	Clarkston, I-75	4	A	A
		8	B	B
		12	C	C
	Saginaw, I-75	4	A	A
		8	B	B
		12	C	C

Based on these findings it can be concluded that the APA rut depth is sensitive to air voids and in particular shows decreased performance with poorly compacted mixtures (air voids greater then 8%). This lends credibility to the practice of taking field cores or beams from newly constructed pavements. If a pavement has been poorly compacted, the resulting decrease in pavement performance would be shown by decreased APA performance.

7.14 Statistical Analysis of the APA Cycles to Failure Results

Presently, most if not all state highway agencies that use the APA in HMA specifications use a pass/fail rut criterion to differentiate between rut resistant and rut prone HMA mixtures. In the previous section, an empirical rut prediction model based upon APA data was presented. This model converts the amount of APA cycles needed to reach a failure APA rut depth to the ESALs needed to cause a pavement rutting failure. A 7 mm rut was shown to correlate with pavement failure, and thus the APA cycles needed to cause a 7 mm rut corresponds to ESALs to failure. Consequently, in order to validate the model, it is useful to know whether the amount of APA cycles needed to induce failure is sensitive to changes in air voids and asphalt content. It has been shown in the literature that high asphalt contents decrease HMA pavement stability and high air voids increase consolidation rutting and decrease aggregate interlock. Both of these factors would decrease a pavement's life. Based on this it is thought that if a performance model is to based upon APA data, the APA cycles to failure property should be sensitive to air voids and asphalt content.

Tables 7.38 through 7.47 summarize the APA cycles to failure results for all 10 projects. These results were analyzed in the same manner as the APA rut depths at 8000 cycles, except the ANOVA table was based upon a different model, APA cycles to failure = f(air voids, asphalt content).

7.14.1 The Effect that Changing Asphalt Content has on the Number of APA to Failure

The effect of changing asphalt content on the number of APA cycles to failure is summarized in Table 7.67. Only one HMA mixture out of ten is shown to be sensitive to a change in asphalt content. Consequently, it can be concluded based on this data that the amount of APA cycles to cause a 7 mm rut depth is insensitive to asphalt content. Upon examination of the APA result mean cycles to failure results in Tables 7.38 through 7.44 it appears that the results follow the correct trends. Most of the data shows decreasing APA cycles to failure with increasing asphalt content. This is what was to be expected. The problem is the variability about the means. The standard deviations are consistently large throughout most of the APA results. Duncan's multiple range method of comparing means is sensitive to these large standard deviations and thus it is difficult to statistically prove that means are different. One way to decrease the variability is to increase the sample size by creating and testing more APA specimens. However, this may be uneconomical since procuring and testing APA specimens is both timely and costly. It is thought that the sample size used in this study, 3 specimens, is a good sample size to use in APA testing. In conclusion, it appears that since statistical differences in the APA cycles to failure between mixture variations do not exist, the APA is unable to predict changes in HMA pavement performance due to changes in asphalt content. Also, since there is a great amount of variability in the number of APA cycles to failure criterion, a PBS based upon APA data may be unrealistic.

Table 7.67 Analysis of the Effect of Asphalt Content on APA Cycles Until Failure, (α=0.05)

Superpave HMA Mixture Design Level	Project Location	Asphalt Content (% by Mass)	Tukey 95% Grouping	Duncan 95% Grouping
E1	Brimley, M-28	Low	A	A
		Optimum	A	A
		High	A	A
E3	Elk Rapids, US-31	Low	A	A
		Optimum	A	A
		High	A	A
	Monroe, US-24	Low	A	A
		Optimum	A	A
		High	A	A
	Brooklyn, M-50	Low	A	A
		Optimum	A	A
		High	A	A
E10	Lansing, M-43	Low	A	A
		Optimum	A	A
		High	A	A
	Indian River, I-75	Low	A	A
		Optimum	A	A
		High	B	B
	Grayling, US-27	Low	A	A
		Optimum	A	A
		High	A	A
E30	Auburn Hills, I-75	Low	A	A
		Optimum	A	A
		High	A	A
	Clarkston, I-75	Low	A	A
		Optimum	A	A
		High	A	A
	Saginaw, I-75	Low	A	A
		Optimum	A	A
		High	A	A

Table 7.68 Analysis of the Effect of Air Void Content on APA Cycles Until Failure, (α=0.05)

Superpave HMA Mixture Design Level	Project Location	Air Voids (%)	Tukey 95% Grouping	Duncan 95% Grouping
E1	Brimley, M-28	4	A	A
		8	A B	A B
		12	B	B
E3	Elk Rapids, US-31	4	A	A
		8	A	A
		12	B	B
	Monroe, US-24	4	A	A
		8	A	A
		12	A	A
	Brooklyn, M-50	4	A	A
		8	B	B
		12	A B	B
E10	Lansing, M-43	4	A	A
		8	A	A
		12	B	B
	Indian River, I-75	4	A	A
		8	A	A B
		12	A	B
	Grayling, US-27	4	A	A
		8	A B	A B
		12	B	B
E30	Auburn Hills, I-75	4	A	A
		8	A	A
		12	A	A
	Clarkston, I-75	4	A	A
		8	A B	A B
		12	B	B
	Saginaw, I-75	4	A	A
		8	B	B
		12	B	C

7.14.2 The Effect that Changing Air Voids has on the Number of APA Cycles to Failure

The sensitivity of the APA cycles to failure criterion to changes in air void content is summarized in Table 7.68. The statistical differences in the number of APA cycles to failure, shown in Table 7.68, are similar to the differences in APA rut depth at 8000 cycles shown in Table 7.66. The statistical differences are summarized as follows:

- In five of the projects, the number of APA cycles to failure from 8 and 12% specimens were statistically different then the 4% specimens;
- In three of the projects, the APA cycles to failure for 12% specimens was statistically different then specimens prepared to 4 and 8% air voids: and
- In one of the projects, the APA cycles to failure for specimens prepared to all three air void levels were statistically different.

In most cases, when a HMA mixture shows statistical differences in APA rut depths @ 8000 cycles due to changes in air voids it also shows the same or approximately the same statistical differences in the number of APA cycles to failure. This suggests a relationship between APA cycles to failure and the APA rut depth at 8000 cycles. This relationship is plotted in

Figure 7.36. It can be seen that the APA cycles to failure is related to APA rut depth at 8000 cycles. A decreased APA rut depth corresponds to increased APA cycles to failure.

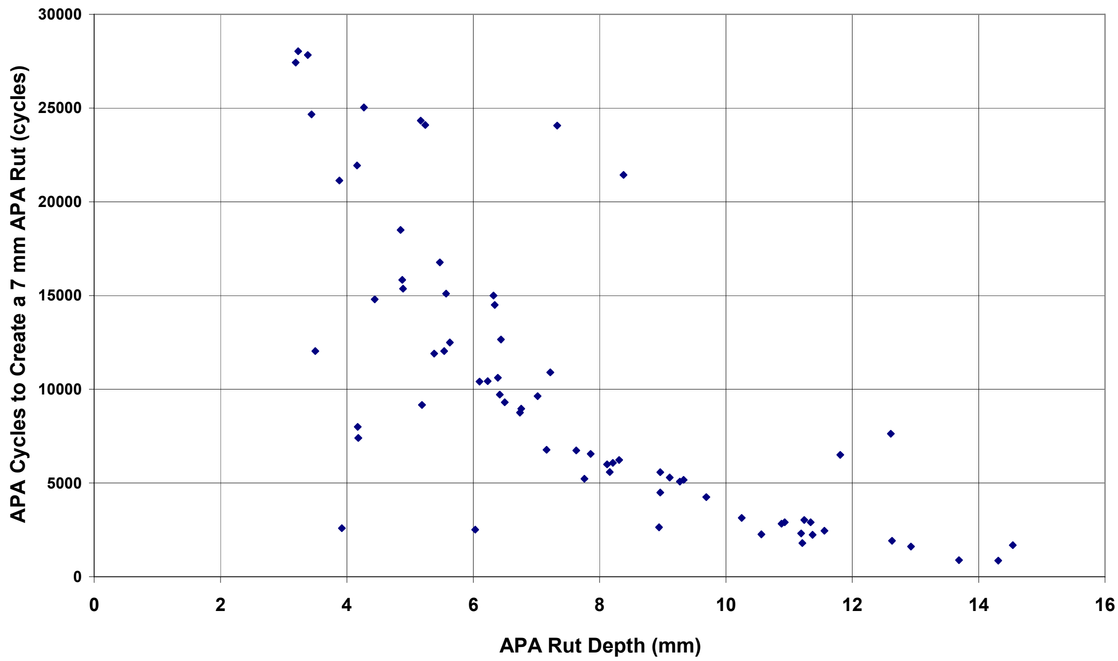


Figure 7.36 APA Cycles to Failure and APA Rut Depth at 8000 Cycles

Since the APA cycles to failure, and consequently the amount of ESALs to rutting failure predicted by the empirical rut prediction model, shows a great deal of variability it may not be feasible to base a rut prediction model based upon APA data. However,

Figure 7.36 lends credibility to the pass/fail criterion currently used by state highway agencies. In the figure it can be seen that low APA rut depths correspond to increased APA cycles to failure and increased pavement life.

7.15 Analysis of Differences in APA Performance at Different Superpave Design Levels

The Superpave HMA mixture design system uses different design criteria for different trafficking levels. Each design level is based on a different number of gyrations in the Superpave Gyratory Compactor (SGC). Higher design levels must meet the Superpave design criteria under greater compactive level mimicking the increased truck traffic the HMA mixture will encounter in service. To withstand a higher compactive effort in the SGC, higher quality aggregates are required.

It was expected the higher traffic level HMA mixtures would perform better in the APA then lower traffic mixtures. It was also thought that the standard deviation of identical specimens being compacted in the APA would increase with an increase in Superpave traffic level. This is because higher quality HMA mixtures are thought to be more sensitive to changes in HMA properties. Knowing whether or not the last two statements are true will be useful to HMA practitioners when using the APA in HMA paving specifications.

To assess the changes in mean and standard deviation of APA rut depth with changes in Superpave mixture design levels the following steps were taken. First, it was decided that only the 8% air voids, optimum asphalt content mixtures would be compared since this is the combination of HMA properties that a properly mixed and compacted HMA pavements should have (i.e. adequate compaction and design asphalt content). Secondly, all the 8% air void, optimum asphalt content mixes for each design level were averaged together to find a mean APA rut depth and standard deviation for each traffic level. The E1 traffic level contained only one project, Brimley M-28, so averaging this project was not necessary. These APA average rut depths and standard deviations for each project level are shown in Table 7.69.

As can be seen from Table 7.69, the APA rut depth did not decrease with a increasing traffic level. Instead the trend is for the APA rut depth to increase with increasing traffic level (with the exception of the on E1 HMA mixture). The reason for this trend may be the fact that APA test temperature varied from test to test. The APA test temperature for all six of the E10 and E30 mixtures were tested at 64 °C while all 5E3 mixtures were tested at 58 °C. This may have resulted in an increased rut depth even though the higher binder performance grades of the 5E3 and 5E10 HMA mixtures should have alleviated the increased rutting due to a higher APA test temperature. Perhaps the reason for the lack of any noticeable trends in Table 7.69 is due to the small amount of projects tested. Only three projects were tested at each traffic level and a excessively bad or good performing mixture could skew the average means away from the actual mean that would result if more projects were sampled. Also, there are no noticeable trends in the standard deviations about the mean rut depths. By examining the data it can be seen that no further statistical analysis is needed since there would be no statistical differences amongst the E3, E10, and E30 HMA mixtures. The E1 HMA mixture may be significantly different, but it the mean rut depth is based only one project, Brimley M-28. It is recommended that before any conclusions are made regarding changes in APA rut depth and standard deviation at different Superpave levels more projects should be tested. In conclusion, the effects of Superpave design level on APA rut depth and standard deviation of APA rut depth is not known.

Table 7.69 Average APA Rut Depth at 8000 Cycles and Standard Deviation at Different Superpave Design Levels

Superpave Traffic Level	E1	E3	E10	E30
Ave rage APA Rut Depth at 8000 Cycles (mm)	10.56	6.77	7.03	7.8
Standard Deviation (mm)	0.73	2.07	3.48	1.93

7.16 The Fitting of Regression Models to Predict APA Rut Depth

It would useful to HMA practitioners to have a regression model available to them that can predict the APA rut depth at 8000 cycles. A regression model, if able to predict APA rut depth adequately, could be used in the mix design process as a screening tool to predict the change of APA performance that will result from using different materials in an HMA mixture. The data lists used in the development of these regression models is found in Appendix D.

7.16.1 A Summary of the Ten HMA Mixture Properties Included in the Models

In this model, ten HMA properties were chosen as predictor variables to predict the dependant variable, which in this case is the APA rut depth at 8000 cycles. Two separate models were developed, one intended for use in research and one intended for practitioners using only HMA mixture properties typically found on a Job Mix Formula (JMF). A description of the ten HMA properties used as dependent variables as well as the properties effect on APA performance are as follows:

- Superpave Mixture Design Level: This property was included in the regression model as a classification variable. An increase in the Superpave mixture design level (i.e. from an E3 HMA mixture to a 5E10 mixture) would be expected to increase APA performance.
- Is the HMA mixture a coarse or a fine mixture (i.e. does the gradation curve pass above or below the Superpave Restricted Zone). This was a classification variable in the model; 0=Coarse, 1=Fine. Prior research has shown that coarse mixtures are more susceptible to changes in HMA mixture properties and thus can be more susceptible to rutting.
- Was the asphalt binder bumped?: In practice, asphalt binders are typically "bumped" a performance grade (PG) above the PG required for a project. For example, the M-43 HMA project in Lansing used a PG 70 binder when the climate required that only a PG 64 binder be used. Binder bumping is typically done on high stress pavements where the truck traffic is very high or traffic is moving slowly, such as at intersections. This practice is intended to increase the rut resistance of HMA pavements. The APA test settings presented in this paper require that APA testing be done at the high temperature of the PG grade. The APA test temperature does not include bumps in the binder grade. For example, the Lansing M-43 project was tested in the APA at 64 °C, not 70 °C. The binder bump was included in the model as a classification variable; 0= binder was not bumped, 1=binder is bumped. A binder bump is expected to result in better APA performance.
- Fine aggregate angularity (FAA): FAA is a measure of the angularity of the aggregate passing the No. 8 sieve. The FAA used in this model was taken from the JMF of the project. The FAA is determined by AASHTO TP33. It should be noted that the FAA of

the laboratory HMA mixtures may have varied slightly from the JMF FAA values. An increase in FAA is thought to increase APA performance.

- G*/sin δ: G*/sin δ, the complex modulus, is the asphalt binder property used in the Superpave Performance Grade Binder Specification to assess a binder's susceptibility to rutting (SHRP,1997). G*, the complex shear modulus, is a measure of the asphalt binder's resistance to deformation while the phase angle, δ, is a measure of the relative amounts of elastic and inelastic deformation. G*/sin δ was determined from rolling thin film oven (RTFO) aged binders sampled from each project. This was done in accordance with AASHTO TP5 at the Michigan Technological University asphalt binder lab. Higher values of G*/sin δ is thought to increase APA performance.
- Asphalt Film Thickness: This property is the measure of the thickness of the asphalt binder film surrounding the aggregate in an HMA mixture. The asphalt film thickness is dependant on the amount of asphalt content, the aggregate gradation, and the aggregate particle shape. The method of calculating the asphalt film thickness used in this study was developed by the National Stone Association Aggregate Handbook (1991). It assumes that all of the aggregate particles are round or cubical, thus it does not consider aggregate shape or texture in its estimation of asphalt film thickness. Based upon the literature, the relationship between asphalt film thickness and APA performance is unclear.
- Fines to Binder Ratio: The fines to binder ratio (F/B ratio) is simply the ratio of mass of the material passing the No. 200 sieve divided by the mass of total asphalt binder in an HMA mixture. The mass of the asphalt binder used in the ratio was the total asphalt

mass, not just the effective asphalt mass. The F/B ratio was calculated based upon the laboratory HMA mixture.   The F/B ratio in this study changed due to changes in the asphalt content, not by changing the amount of material passing the No. 200 sieve, which remained constant for each project. It is believed that fines can become embedded fully into the asphalt binder and act as an asphalt binder extender. As more fines are included into the asphalt binder the asphalt becomes stiffer and will improve APA performance.

- Asphalt Content: The asphalt content was varied as shown in the test matrix (Table 7.64). An increase in binder content above the optimum asphalt content normally results in a loss of mixture stability and a decrease in APA performance.
- Air Voids: Air voids were calculated as summarized in Chapter 5 and varied as shown in Table 7.64. An increase in air voids leads to an increase in consolidation rutting as well as lower shear resistance and consequently decreases APA performance.
- Voids in the Mineral Aggregate: Voids in the Mineral Aggregate (VMA) is "the volume of interangular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume of the sample"(SHRP, 1996). Based upon the literature, the effect of VMA on APA performance is unclear.

In addition to describing the ten HMA properties included in the regression models, it is necessary to establish the range of each property. The regression models presented in the following sections are meant to be used when the HMA properties are within the ranges presented in Table 7.70.

Table 7.70 Range of HMA Properties Included in Regression Models

HMA Mixture Property	Range
Superpave Mixture Design Level	E1, E3, E10, and E30
Gradation	Superpave Coarse and Fine Gradations
Asphalt Binder Bumping for High-Stress HMA	No Bump or a One Binder Grade Bump
Fine Aggregate Angularity (%)	40.7 – 48.2
Rolling Thin Film Oven aged G*/sin δ (kPa)	2.26 – 4.43
Asphalt film Thickness (microns)	4.55 – 13.25
Fines to Binder Ratio	0.61 – 1.35
Asphalt Content (%)	3.8 – 7.7
Air Voids (%)	4 – 12
Voids in the Mineral Aggregate (%)	13.8 – 23.9

7.16.2 A Summary of the Process Used to Build the Regression Models

Using the preceding HMA mixture properties as predictor variables, regression models to predict the APA rut depth at 8000 cycles were produced. The SAS System for Regression, a statistical software package, was used to develop these regression models. The models were developed using the following steps:

1. All main effects were included in the model. The stepwise regression procedure was used to find which main effects should be included in the model (Neter et al., 1996). This was done using the SAS statistical package. $\alpha = 0.1$ was used for both entering and taking main effects out of the model during the stepwise regression procedure.
2. After the main effects in the model were chosen, interaction effects between the continuous predictor variables were entered into the model. All effects were entered into the model and the effects were removed if the partial F-Value of the predictor was greater then 0.1. This, too, was done using the SAS statistical package.

3. Once a candidate model was identified, the SAS residual plot (Appendix G) was created to find whether or not a transformation of the dependant variable was necessary.

The resulting models are summarized in the following sub-sections. Appendix D presents the SAS data lists used to create these regression models. Appendix F presents a table of Pearson correlation coefficients, which demonstrates that there are significant interactions between the variables in these regression models. Appendix G presents the SAS output of each model. The SAS output includes the following:

- An Analysis of Variance (ANOVA) table,
- Partial F-Values and p-values of all predictor variables,
- Predictor variables and their standard deviation,
- Residual plots, and
- Normal probability plots.

As mentioned different models were developed: models for research purposes and models that are more practitioner oriented. These are summarized in the following sections.

7.16.3 Regression Model Developed for Use by Researchers

The regression model developed for researchers was developed using all ten of the HMA mixtures summarized in the preceding sections. Two separate models were developed. The first model was developed using data from all 210 specimens. The second research model was developed using the average data from each asphalt content/air void combination, or 70 data points in all. It is anticipated that if an APA specification is developed, the APA specifications will be based on the average results taken from a sample size of three APA specimens. By developing a model based on the average data, most of the error due to the APA itself was removed from the model and the R^2 increased accordingly.

The first model developed has a coefficient of correlation (R^2) of 0.749 and is used to predict the APA rut depth of a single specimen tested in the APA:

$$\begin{aligned} APA\,Rut\;Depth_{8000\,Cycles} = & -429.635 - 5.909\,\mathrm{E3} - 5.566\,\mathrm{E10} - 4.481\,\mathrm{E30} + 0.704\,VMA \\ & + 21.734\,AFT + 260.873\,FB + 8.959\,FAA - 2.066\,AFT\;FB \\ & - 0.425\,AFT\;FAA - 5.349\,FB\;FAA \end{aligned} \qquad (7.16)$$

where:

E3 = Is the HMA a Superpave E3 Mixture? Yes = 1, No = 0
E10 = Is the HMA a Superpave E10 Mixture? Yes = 1, No = 0
E30 = Is the HMA a Superpave E30 Mixture? Yes = 1, No = 0
VMA = Voids in the Mineral Aggregate (% by Mixture Volume)
AFT = Asphalt Film Thickness (microns)
FB = Fines/Total Binder Content
FAA = Fine Aggregate Angularity

The second model has an R^2 of 0.806 and is used to predict the average APA rut depth of three APA specimens:

$$APA\,Rut\,Depth_{8000\,Cycles} = -12.300 - 1.710\,\mathrm{E3} - 1.503\,\mathrm{E10} + 2.263\,Bump + 0.149\,VMA \\ +1.279\,AFT + 3.838\,RTFO + 0.066\,VMA\,AFT \\ -0.664\,AFT\;RTFO \tag{7.17}$$

where:

E3 = Is the HMA a Superpave E3 Mixture? Yes = 1, No = 0
E10 = Is the HMA a Superpave E10 Mixture? Yes = 1, No = 0
Bump = Was the Upper PG Bumped? 0=Yes, 1=No
VMA = Voids in the Mineral Aggregate (% by Mixture Volume)
AFT = Asphalt Film Thickness (microns)
FAA = Fine Aggregate Angularity
RTFO = (G*/sin (delta)) of RTFO Aged Asphalt Binder

7.16.4 Regression Model Developed for Use by Practitioners

The regression model developed for practitioners was created using eight of the predictor variables. Asphalt film thickness and G*/sin δ were left out of the model because they are not found on the JMF sheets for MDOT. This model can be used by practitioners to develop HMA mixtures that perform well in the APA. This would be useful if an APA specification is included in the Michigan HMA paving specification.

The first model has a R^2 of 0.523 and is used to predict the APA rut depth of a single APA specimen:

$$APA\,Rut\,Depth_{8000\,Cycles} = 2.095 - 3.392\,\mathrm{E3} - 2.529\,\mathrm{E10} - 1.535\,E30 + 0.644\,VMA - 5.080\,FB \qquad (7.18)$$

where :

E3	= Is the HMA a Superpave E3 Mixture? Yes = 1, No = 0
E10	= Is the HMA a Superpave E10 Mixture? Yes = 1, No = 0
E30	= Is the HMA a Superpave E30 Mixture? Yes = 1, No = 0
VMA	= Voids in the Mineral Aggregate (% by Mixture Volume)
FB	= Fines/Total Asphalt Binder Ratio

The second model has a R^2 of 0.727 and is used to predict the average APA rut depth of three APA specimens:

$$APA\,Rut\,Depth_{8000\,Cycles} = -14.357 - 5.900\,\mathrm{E3} - 3.153\,\mathrm{E10} - 3.095\,\mathrm{E30} + 2.598\,Grad + 1.591\,Bump + 0.760\,VMA + 1.365\,AC \qquad (7.19)$$

where :

E3	= Is the HMA a Superpave E3 Mixture? Yes = 1, No = 0
E10	= Is the HMA a Superpave E10 Mixture? Yes = 1, No = 0
E30	= Is the HMA a Superpave E30 Mixture? Yes = 1, No = 0
Bump	= Was the Upper PG Bumped? 0=Yes, 1=No
Grad	= What Kind of Aggregate Gradation? 0 = Fine, 1 = Coarse
AC	= Asphalt Content (% of Mass of Mixture)
VMA	= Voids in the Mineral Aggregate (% by Mixture Volume)

7.16.5 Commentary on the Use of the Regression Models

The models developed in the previous section can be used to predict the APA rut depth of HMA based on ten different HMA properties. These models should be useful in the HMA design process and the following sections contain comments about the relative effect that the predictor variables have on the predicted APA rut depth.

7.16.5.1 Commentary on the Regression Model Developed for Researchers to Predict the APA Performance of a Single APA Specimen

This model, presented as Equation 7.16, is used to predict the rut depth of one APA specimen and demonstrates a good coefficient of correlation, $R^2 = 0.749$. This model shows a decrease in the APA rut depth of 5.91, 5.57, and 4.48 mm for E3, E10, and E30 Superpave design levels, respectively. If zero is entered in all three of the design level terms, the model predicts the rut depth of an E1 mixture. This model used four HMA properties to predict the APA rut depth: VMA, asphalt film thickness (AFT), fines to total binder content ratio (FB), and fine aggregate angularity (FAA). The VMA is believed to include the effects of both the asphalt and air void contents. This is because VMA is, by definition, the volume of effective asphalt content and air voids combined. An increase in VMA from 18 to 19% corresponds to an increase in the APA rut depth of 0.70 mm. The other three properties interact with one another as shown by the interaction terms in the model. The effects that changes in these HMA properties have on APA rut depth in this model are shown as follows:

- Effects of change in AFT: To evaluate the effect that changing AFT has on the APA rut depth, Equation 7.16 is derived with respect to AFT as follows

$$d\,RD \big/ d\,AFT = 21.734 - 2.066\,FB - 0.425\,FAA \tag{7.20}$$

Using Equation 7.20 and setting FAA and FB at 44.3 and 0.98 (the midpoints of their respective ranges), an increase in the AFT of one micron corresponds to an increase in APA rut depth of 0.9 mm. The AFT is a function of asphalt content and gradation. In particular, the AFT increases with an increase in asphalt content. Based on this, the increase in APA rut depth with increasing AFT could be the result of increasing asphalt content.

- Effects of change in FB: To evaluate the effect that changing FB has on the APA rut depth, Equation 7.16 is derived with respect to FB as follows:

$$d\,RD \big/ d\,FB = 260.873 - 2.066\,AFT - 5.349\,FAA \qquad (7.21)$$

Using Equation 7.21 and setting AFT and FAA at 8.9 micron and 44.3 (the midpoints of their respective ranges), an increase in the FB of 0.1 results in an increase in the APA rut depth of 0.55mm. An increase in FB should stiffen the binder and lower the APA rut depth. The FB effect may be effected by interaction amongst different HMA properties.

- Effects of change in FAA: To evaluate the effect that changing FAA has on the APA rut depth, Equation 7.16 is derived with respect to FAA as follows

$$d\,RD \big/ d\,FAA = 8.959 - 0.425\,AFT - 5.349\,FB \qquad (7.22)$$

Using Equation 7.22 and setting FB and AFT at 0.98 and 8.9 microns, an increase in the FAA of one percent results in a decrease in the APA rut depth of 0.07 mm. An increase in the FAA should decrease the APA rut depth, but this decrease is shown to be minimal.

In conclusion, The APA rut depth in this model is very sensitive to whether or not a HMA mixture is a Superpave E1 mixture is and sensitive to changes inVMA, AFT and FB. The effect that FAA has on rut depth is minimal, but some of the effect of changes in this property may be included in VMA through interaction.

7.16.5.2 Commentary on the Regression Model Developed for Researchers to Predict the Average APA Performance of Three APA Specimens

Building a regression model to predict the average rut depth of three specimens should reduce the error due to the APA itself and should increase the correlation between the predictor variables and the APA rut depth. This was true in this case since the R^2 increased from 0.749 to 0.806 between the model used to predict the APA rut depth of a single specimen and the model used to find the average rut depth of 3 specimens. The regression model described in this commentary is Equation 7.17.

In this model, the effect that Superpave design level had on the APA rut depth was less pronounced then the model presented in Equation 7.16. Superpave E3 and E10 design levels decrease the APA rut depth 1.71 and 1.50 mm, respectively. The effect of the E30 design level is probably included in other HMA properties as the result of interaction.

The effect that "binder bump" has on the APA rut depth in this model is significant. A binder bump leads to a decrease in the APA rut depth of 2.26 mm. This is the most pronounce effect that any of the terms in this model have on the APA rut depth.

The remaining HMA properties in this model are VMA, AFT, and RTFO-aged complex modulus (RTFO). All of these properties interact with one another as shown by the interaction terms in this model. The effects that these properties have on APA rut depth are summarized as follows:

- The Effect of VMA: To evaluate the effect that changing VMA has on the APA rut depth, Equation 7.17 is derived with respect to VMA as follows

$$ {dRD}/{dVMA} = 0.149 + 0.066\,AFT \qquad (7.23) $$

Using Equation 7.23 and setting AFT at 8.9 microns, an increase in the VMA of one percent results in an increase in the APA rut depth of 0.73 mm. An increase in VMA could result from an increase in air void or asphalt content, and in this case the increase in APA rut depth would follow the correct trend.

- The Effect of AFT: To evaluate the effect that changing AFT has on the APA rut depth, Equation 7.17 is derived with respect to AFT as follows

$$ {dRD}/{dAFT} = 1.279 + 0.066\,VMA - 0.664\,RTFO \qquad (7.24) $$

Using Equation 7.24 and setting VMA and RTFO at 18.9% and 3.35 kPa (the midpoint of their respective ranges), an increase in the AFT of one micron results in an increase in the APA rut depth of 0.3 mm. The AFT is a function of asphalt content and gradation. In particular, the AFT increases with an increase in asphalt content. Consequently, the increase in APA rut depth with increasing AFT could be the result of increasing asphalt content.

- The Effect of RTFO: To evaluate the effect that changing RTFO has on the APA rut depth, Equation 7.17 is derived with respect to AFT as follows

$$ {dRD}/{dRTFO} = 3.838 - 0.664\,AFT \qquad (7.25) $$

Using Equation 7.25 and setting AFT at 8.9 microns, an increase in the RTFO of one kPa decreases the APA rut depth by 2.07 mm. Since RTFO is a measure of the binder stiffness and elasticity, an increase in the RTFO should decrease the APA rut depth as is shown.

In conclusion, bumping the binder grade has the most significant effect on the APA rut depth in this model. Changes in VMA and RTFO have a significant effect on the APA rut depth while changes in the AFT have a less significant effect. In this model, using the E3 and E10 Superpave design levels both lower the APA rut depth while the E30 design level did not have a significant enough effect to be included in this model. It is thought that the effect of the E30 Superpave design level is probably included in other predictor variables through interactions.

7.16.5.3 Commentary on the Regression Model Developed for Practitioners to Predict the APA Performance of a Single APA Specimen

The prediction of the APA rut depth of a single specimen based only on HMA properties found on the JMF may be unrealistic based on the model presented as Equation 7.18, which has a poor R^2 of 0.523. Due to this low correlation it is advised that HMA practitioners test APA specimens in sets of three and use Equation 7.19 to predict APA performance.

7.16.5.4 Commentary on the Regression Model Developed for Practitioners to Predict the Average APA Performance of Three APA Specimens

The model presented in Equation 7.19 provides an $R^2 = 0.727$, an improvement over the model used to predict the APA rut depth of a single specimen ($R^2 = 0.523$).

This model contains terms including all three of the Superpave design levels. The E3, E10, and E30 design levels decrease the APA rut depth by 5.90, 3.15, and 3.10 mm, respectively. If none of these terms are included in the model (i.e., zero is entered for all three terms) the model gives the predicted rut depth of an E1 mixture. Using the E3, E10, and E30 design levels results in a significantly lower APA rut depth than if the E1 design level is used.

If the asphalt binder is “bumped”, the APA rut depth decreases by 1.59 mm. Binder bumping has a significant effect on the APA rut depth in this model.

This model is sensitive to changes in the aggregate gradation. According to this model, a coarse-graded mixture will have a 2.60 mm deeper APA rut depth then a fine-graded mixture. Interestingly, this is the only model out of the four developed that included the gradation as a significant predictor variable. The FHWA at Westrack (1998) concluded that coarse-graded Superpave mixtures are significantly affected by in-place density, or air-voids. This may be the reason of the decreased APA performance of coarse-graded mixtures.

As in the other models, an increase in VMA increases the APA rut depth. In this case, an increase of the VMA from 18 to 19% results in a 0.76 mm increase in rut depth. The VMA is defined as the volume of the effective asphalt content and air voids combined. Since this model has asphalt content (AC) as a predictor variable, it is believed that the VMA may be effected by air void content. This conclusion is made since an increase in air void content leads to an increase in VMA and, as shown in this chapter, an increase in air voids also leads to an increased APA rut depth.

This model includes AC as a predictor variable. In the previous models, AC may have been interacting with AFT. AFT was not included while building this model, and as a result of this, AC may have became a significant predictor. An increase in asphalt content of 1% results in an increase in APA rut depth of 1.37 mm.

In conclusion, the APA rut depth is lowered significantly when developing HMA using the E3, E10, and E30 Superpave design levels as opposed to E1 mixtures. The APA rut depth predicted by this model is also significantly changed by aggregate gradation; a coarse-graded mixture ruts 2.60 mm more then fine-graded mixtures. VMA and AC also effect the APA rut depth, although less significantly then the design level or gradation.

7.17 Statistical Analysis of 4-Point Beam Fatigue Samples

The main purpose of the statistical analysis was to quantify the effects of certain variables and to model their effects on the fatigue life of hot mix asphalt. Each job listed in the experimental plan was tested in the four point beam fatigue apparatus. The results for each job were compiled and statistically analyzed using regression analysis. A regression equation allows for the relationship between a variable of interest (the dependent variable) to be explained as a function of other factors (the independent variables). The regression equation is commonly referred to as a prediction equation. Also, this type of analysis allows one to see what factors are statistically significant in explaining the relationship between the dependent and independent variables. The level of significance chosen for all regression modeling was 90%. This means that there is a 10% chance of making a Type I error. A Type I error occurs when the null hypothesis is rejected; when in fact it is true (McClave and Sincich, 2000). The dependent variable in the analysis was the natural log of the cycles to failure (Log Cycles). The independent variables were: microstrain (MS), air voids (AV), initial stiffness (IS), initial modulus (IM) and asphalt content (AC). These variables were the main effects in the model. Microstrain had no units, air voids was entered as a percentage (e.g., 4.00%), initial stiffness and modulus had the units of megapascals (MPa) and asphalt content was entered as a percentage by weight of the total mix (e.g., 5.0%). Nominal maximum aggregate size (NMAS 3, 4 and 5) and traffic level (E3, E10 and E30) were later added to the regression model as simple dummy variables.

7.17.1 Explanation of the Backward Elimination Regression Procedure

The particular type of regression analysis used was the backward elimination regression. This method systematically removes variables that were found statistically insignificant at a certain level of confidence. All of the main effects were regressed against the cycles to failure. It was found that by using the value of the natural log of the cycles to failure the model greatly improved. The relationship takes on the following form:

$$\text{Log Cycles to Failure} = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_4 X_4 + \beta_5 X_5 \qquad (7.26)$$

The β parameters in the first order model represent numerical coefficients that explain the relationship between the cycles to failure and the main effects. Next, any variable found insignificant at the α=0.10 level was removed. This process continued until all effects remaining in the model were significant. The remaining β parameters represent the slope of the line relating the cycles to a given independent variable, given that all of the other independent variables were held constant. This model was then checked for usefulness using a simple global F-test. The global F-test tested the following hypotheses using the two appropriate F-statistics (McClave and Sincich, 2000).

$$H_O : \beta_1 = \beta_2 = ... = \beta_n = 0$$
$$H_A = \text{At least one } \beta_{\text{i}} \neq 0$$

$$F_{calculated} = \frac{\text{Mean Square of the Model}}{\text{Mean Square Error}}$$
$$F_{critical} = F_{\alpha\text{, k, n-(k+1)}}$$
$$\text{Rejection Region: } F_{calculated} > F_{critical}$$

Where:

Mean square of the model is the variability of the independent variable that is explained in the regression model,

Mean square error is an estimate of the variance not explained by the regression model,

α is the confidence coefficient (0.10) for 90% confidence,

k is the number of β parameters not including the intercept (β_0), and

n is the sample size.

Next, it was important to see if it was possible to improve the model. Interaction and quadratic terms between the remaining significant main effect variables were entered into the model and then analyzed. The elimination procedure was then performed on a model containing any significant main effects, while removing any non-significant interaction or quadratic terms until all remaining variables were significant at the α=0.10 level. This type of model is no longer linear and takes on the following general form:

$$\text{Log Cycles} = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_1 \cdot X_2 + \beta_4 {X_1}^2 + \beta_5 {X_2}^2 \quad (7.27)$$

In this equation β_0 is the intercept, β_1 and β_2 cause the response surface to shift along their respective axes, β_3 controls the rotation of the function's surface (this is called an interaction term), and β_4 and β_5 (these two terms are called quadratic terms) control the type of the response function's surface and its rates of curvature (McClave and Sincich, 2000). The usefulness of this model was then checked using the same simple global F-test previously mentioned. The original regression data is provided in Appendix I.

7.17.2 Pairwise Comparison of Aggregate Size and Traffic Level

The next step was to add both the NMAS and the design traffic level into the regression model. First, the Bonferroni method was used to do pairwise comparisons of the mean value for the natural log of the cycles to failure. This method is conservative, meaning that it requires more evidence than some other pairwise comparison methods to reject the hypothesis that two means are equal. The Bonferroni method controls the Type I experimentwise error rate, which is the probability of making at least one Type I error when comparing the means of two groups. It was found that the mean cycles to failure for an NMAS=4 were statistically significantly different than the mean for an NMAS=5. Furthermore it was found that the mean log cycles to failure for an NMAS=5 was greater than that of NMAS=4. The same relationship was found between the mean cycles to failure for the traffic levels of E3 and E30. Also, it was found that the mean log cycles to failure for a traffic level of E3 was greater than that of E30. The Bonferroni test results are provided in Appendix I.

The model was then checked to ensure that the assumptions made when developing regression equations were, in fact, valid. It was assumed that the error in the regression model was distributed randomly. This was checked by making a plot of the residuals versus the predicted value. If no apparent pattern is seen, the model's error is randomly distributed. Also, it was assumed that the model's error follows a normal distribution. This was checked by making a normal quartile plot of the residuals. If this plot is a fairly straight line, then the assumption of normality is validated.

7.18 Discussion of the Regression Model for Beam Fatigue Samples

This section discusses the statistical regression analysis results of the fatigue life for the 18 paving projects from which aggregates and asphalt binder were collected. Preliminary graphs were made of the main effects versus the observed log cycles to failure. The graphs revealed that the fatigue life decreased with increasing rates of microstrain. The plots of air voids, initial stiffness and modulus illustrated a weak negative trend that meant fatigue life decreased as the magnitudes of these variables were increased. The plot of asphalt content versus fatigue life revealed that there was a slight positive trend. This means that the fatigue life increased slightly as the asphalt content increased.

All regression equations were derived using Statistical Analytical Software (SAS version 8.2, 2001). The SAS code is provided in Appendix I. When the gradations and traffic levels were added to the model, the β parameter for NMAS=4 was found to be statistically insignificant. To account for this and to effectively model all of the data, all observations that were originally labeled as having an NMAS=4 were combined with and changed labeled as 3 for the input data file. This seemed reasonable seeing that there was no statistically significant difference between the NMAS of 3 and 4 and that the NMAS was entered into the SAS data file as a simple dummy variable (0 if NMAS=5 and 1 if NMAS of 3 or 4). At this point there were only two (instead of the three original levels) levels for NMAS. Likewise, it was found that the β

parameter for the traffic level of E30 was not statistically significant. The traffic levels that were labeled as E10 were combined with and changed to those labeled E30 to account for this. This again seemed reasonable since there was no significant difference between the traffic levels of E10 and E30, and that traffic level was entered as another simple dummy variable into the SAS data file (0 if traffic=E3 and 1 if traffic=E10 or E30). The following plots and tables illustrate the mean log cycles to failure by traffic and gradation.

Table 7.71: Mean Log Cycles to Failure by Traffic and Gradation

NMAS Gradation	Traffic Level	Mean Log Cycles to Failure
3	3	11.07
3	30	11.28
4	3	11.18
4	30	10.47
5	3	11.25
5	10	11.06
5	30	11.07

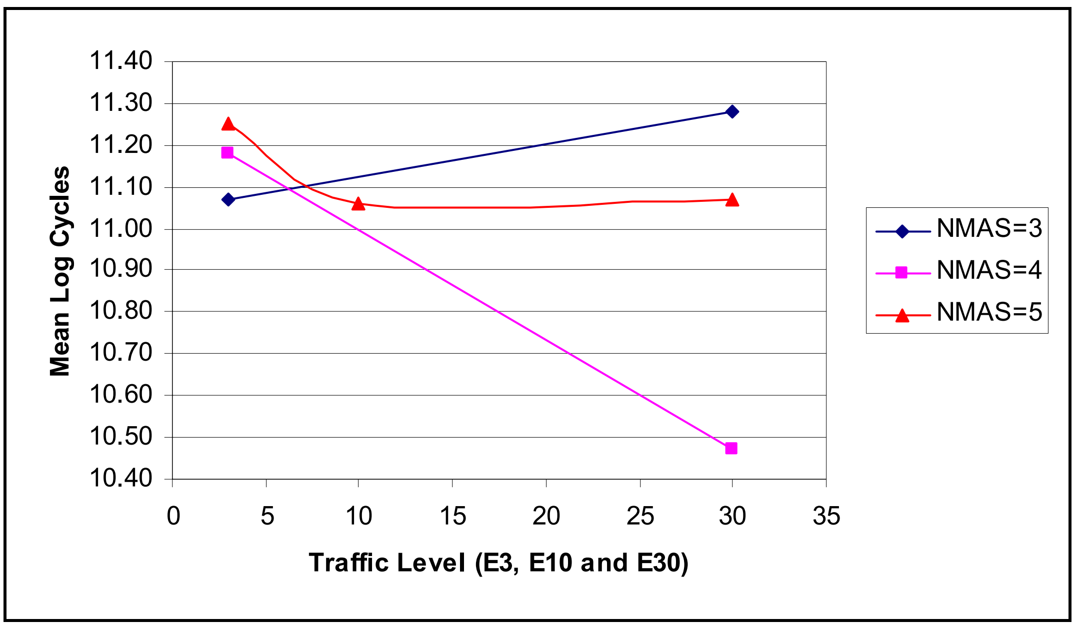


Figure 7.37: Mean Log Cycles to Failure vs. Traffic Level by Gradation

Figure 7.37 illustrates the interaction between all three traffic levels when plotted by NMAS. Figure 7.37 shows that the mean log cycles to failure for the E30 traffic levels was generally lower than those for E3. This means that an E30 pavement has a higher possibility of developing a fatigue crack earlier than does an E3 pavement. Also, an HMA mix with an NMAS=5 clearly had a longer fatigue life than did a mix with an NMAS of 4. This means that a mix with an NMAS=4 has a higher possibility of developing a fatigue crack earlier than does a mix that had an NMAS of 5. Figure 7.37 was made before combining any levels of traffic or NMAS. Table 4-2 and Figure 7.38 illustrate the mean log cycles to failure after the traffic and NMAS levels were combined. As seen in Figure 7.38 there is no longer any interaction between the traffic levels. It is also interesting to note that the lower traffic mixes performed better than do the higher traffic mixes, as illustrated in Figure 7.38. Also, the mixes with an NMAS=5 now clearly have longer fatigue lives than the combined NMAS=3/4.

Table 7.72: Mean Log Cycles to Failure of Traffic and Aggregate Size after Level Combinations

NMAS Gradation	Traffic Level	Mean Log Cycles to Failure
3 & 4	3	11.08
3 & 4	10 & 30	10.67
5	3	11.25
5	10 & 30	11.07

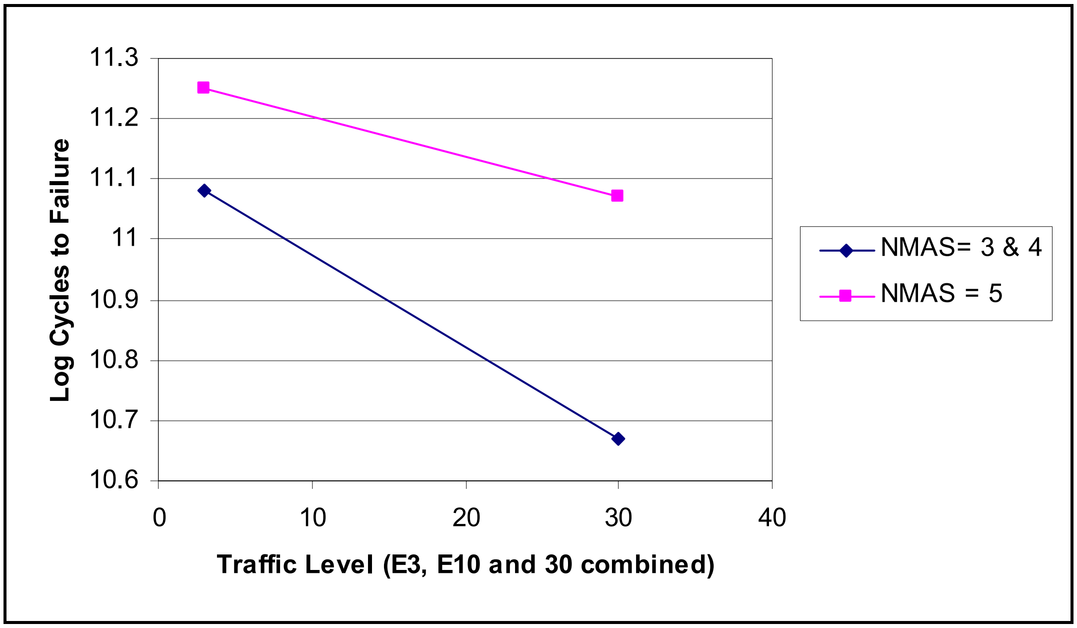


Figure 7.38: Mean Log Cycles to Failure vs. Traffic Level by Gradation (after level combination)

It should be noted that the initial experimental plan could be simplified for further analysis. By combining some levels among traffic and gradation an evolved experimental matrix was developed, as shown in Table 7.73.

Table 7.73: Analysis Plan with Level Combinations

		Traffic Level		
		E3	**E10 and E30**	
Nominal Maximum Aggregate Size (NMAS)	**(3) 19.0mm, 3/4" and (4) 12.5mm, 1/2"**	Old M-14 M-32 Alpena M-45 Grand Rapids M-35 Escanaba M-90 Lexington	I-94 Ann Arbor (WLD) 8 Mile Rd., Warren I-94 Ann Arbor (WLN) I-94 Ann Arbor, SMA	
	(5) 9.5mm, 3/8"	M-50 Brooklyn US-31 Elk Rapids US-24 Monroe	I-75 Indian River I-75 Grayling M-43 Saginaw St.	I-75 Auburn Hills I-75 Flint I-75 Saginaw

It is important to note that a few interaction terms were omitted from the final model. The first term was the interaction term with regards to initial stiffness and asphalt content (designated as ISxAC). It was found that when this term was included in the model, the traffic level term was insignificant. As a result, the ISxAC interaction term was discarded and the traffic level was then significant. It was felt that the inclusion of the main effects were more important than the interaction term. The second term that was omitted was the interaction between microstrain and asphalt content (MSxAC). This term caused the coefficient in front of AC (asphalt content) to be negative. The preliminary plot of asphalt content versus the natural log of failure cycles showed a weak positive trend. The MSxAC term was taken out of the model and the resulting coefficient for AC was then a positive number, which is rational for this coefficient. Finally, the terms for the interaction between air voids and asphalt content (AVxAC) and the quadratic term for air voids (AV^2) were discarded to keep the regression model as simple as possible for the future development of the pay factor system, which would use the overall regression function. The overall quality of the regression model as a measure of the R^2_{adj} was slightly lower without the interactions (0.6686) than with them (0.6878). The R^2_{adj} is a ratio of the measure of the variability of the dependent variable that is explained in the regression model divided by the error within the model while taking into account the number of prediction variables used as well as the sample size (McClave and Sincich, 2000). However, the overall F-test quality improved from 53.37 to 63.33 after discarding the interaction variables. The R^2_{adj} and the $F_{calculated}$ values are presented in Appendix I. This was due to the fact that the model now contained only the essential terms and thus, the mean square error of the model (which is a measure of the variability among the terms used in the regression) was higher.

7.18.1 Presentation of the Regression Model

The significant main effect variables were microstrain, air voids, initial stiffness and asphalt content. Initial modulus was dropped out of the regression early on due to the fact that these observations were highly numerically correlated when compared to initial stiffness. The final regression equation for all data combined is presented below.

$$\begin{aligned}
&\text{Log Cycles} = 17.07054 - 0.00853MS - 0.19451AV \\
&-7.7309\times10^{-4}IS + 0.21630AC + 3.26\times10^{-6}MS^2 \\
&+9.407489\times10^{-8}IS^2 - 7.62028\times10^{-7}MSxIS \\
&+6.631\times10^{-5}AVxIS - 0.18715(\mathit{Traffic} = \text{E10 or E30}) \\
&-0.28706(\text{NMAS=3 or 4})
\end{aligned} \qquad (7.28)$$

Where:

Log Cycles is the natural log of the cycles to failure for fatigue life,

MS and MS^2 are the microstrain level and the microstrain level squared, respectively,

AV is the air voids content, expressed as a percentage,

IS and IS^2 are the initial stiffness and the initial stiffness squared, respectively, in MPa,

AC is the asphalt content, expressed as a percentage,

MSxIS is the interaction term for microstrain and initial stiffness,

AVxIS is the interaction term for air voids and initial stiffness,

Traffic = E10 or E30 is equal to 1 if the traffic level is E10 or E30 and is equal to 0 if the traffic level is E3, and

NMAS = 3 or 4 is equal to 1 if the NMAS is 3 or 4 and is equal to 0 if the NMAS is 5.

In the above equation a traffic level of E3 was set as the base value. When using this equation, if the input traffic level of E3, then the traffic level term in the regression equation is dropped because it only applies for the E10 and E30 levels. However, if the traffic level is either E10 or E30, the equation is reduced by 0.18715. Similarly, an NMAS of 5 was set as the base value for NMAS. If the NMAS is 5, then the NMAS term in the above equation is not used. If the NMAS is either a 3 or 4, then the equation is reduced by 0.28706 (the NMAS coefficient).

The coefficients for microstrain and air voids appear reasonable, because as the magnitude of these variables is increased, the resulting fatigue life should decrease. Notice that the coefficient in front of initial stiffness is very small and negative. This is due to the high degree of correlation between initial stiffness and IS^2 (-0.9017), MSxIS (-0.6573), and AVxIS (-0.7141). The correlation coefficients are presented in Appendix I. The interaction and quadratic terms help to explain more variability in the model, but at the same time can have a diminishing effect on the coefficient for the main effect variables. This was especially true with respect to the asphalt content (AC) variable. In the main effects model it was clearly a dominant variable that controlled the fatigue life (see Appendix I). Now that the interaction, quadratic, traffic and NMAS terms had been added, the overall magnitude of the asphalt content coefficient was nearly equal but opposite to that of the air voids coefficient.

It is important to remember that this equation gives an output value of the natural log of cycles and that that value must be converted to an integer by exponentiation.

The regression model had an adjusted r-squared (R^2_{adj}) value of 0.6686. This means that the model explains about 67% of the variability in the original data. The R^2_{adj} is a good estimate of the model variability in that it takes into consideration the number of β parameters and the sample size. It is in essence, a more conservative estimate of the variability than the R^2 value (which is the ratio of the measure of the variability of the dependent variable that is explained in the regression model divided by the unexplained error within the model, but does not take into account the number of predictor variables used or the sample size,) and is often reported when more than one predictor variable is present in the model (McClave and Sincich, 2000). The reported $F_{calculated}$ value is 63.33, which is much larger than the $F_{critical}$ value of about 1.60 (α=0.10, $\nu1$=10, $\nu2=\infty$). $\nu1$ and $\nu2$ represent the degrees of freedom based on the number of prediction parameters and the overall sample size, respectively. The residuals (natural log observed cycles - natural log predicted cycles) of the regression model had a random distribution with no apparent pattern, and indicated that there was no strong dependence on any one variable. This is shown in the plot of the residuals versus the predicted log cycles to failure in Figure 7.39. The check for normality was done by simply creating a normal quantile plot of the residuals for the model, as shown in Figure 7.40. The plot reveals a fairly normal distribution as it follows a fairly straight line. The SAS code used, the regression analysis outputs, the plots of the main effects and the analysis of variance (ANOVA) table for the model is provided in Appendix I.

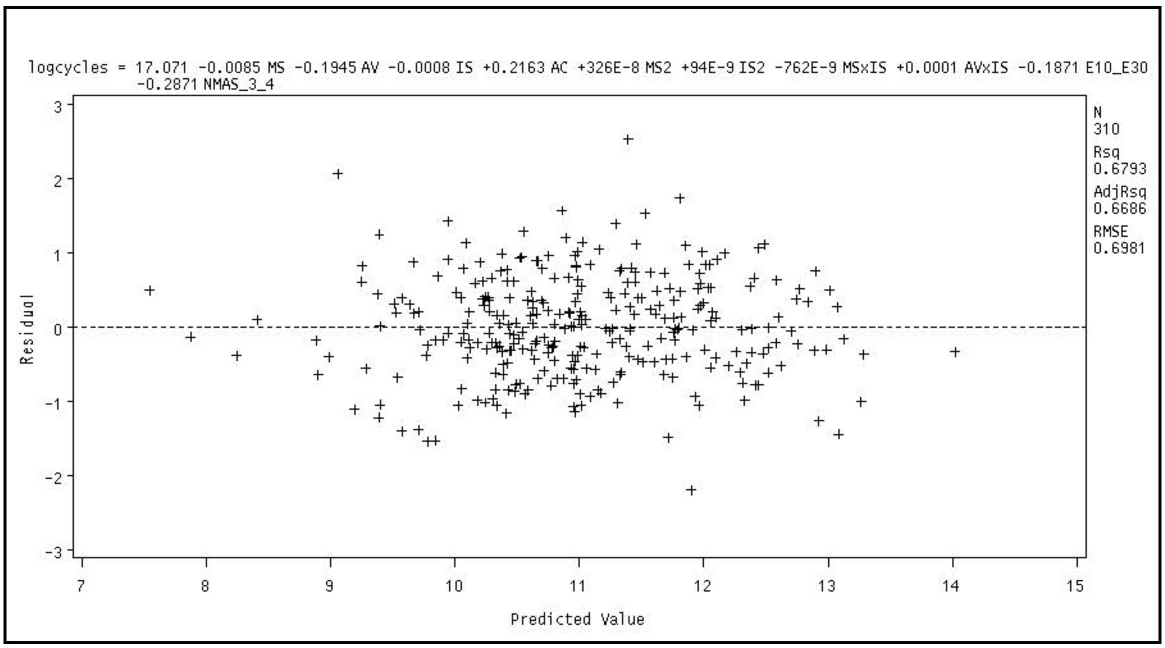


Figure 7.39: Plot of Residuals vs. Predicted Value of Log Cycles to Failure

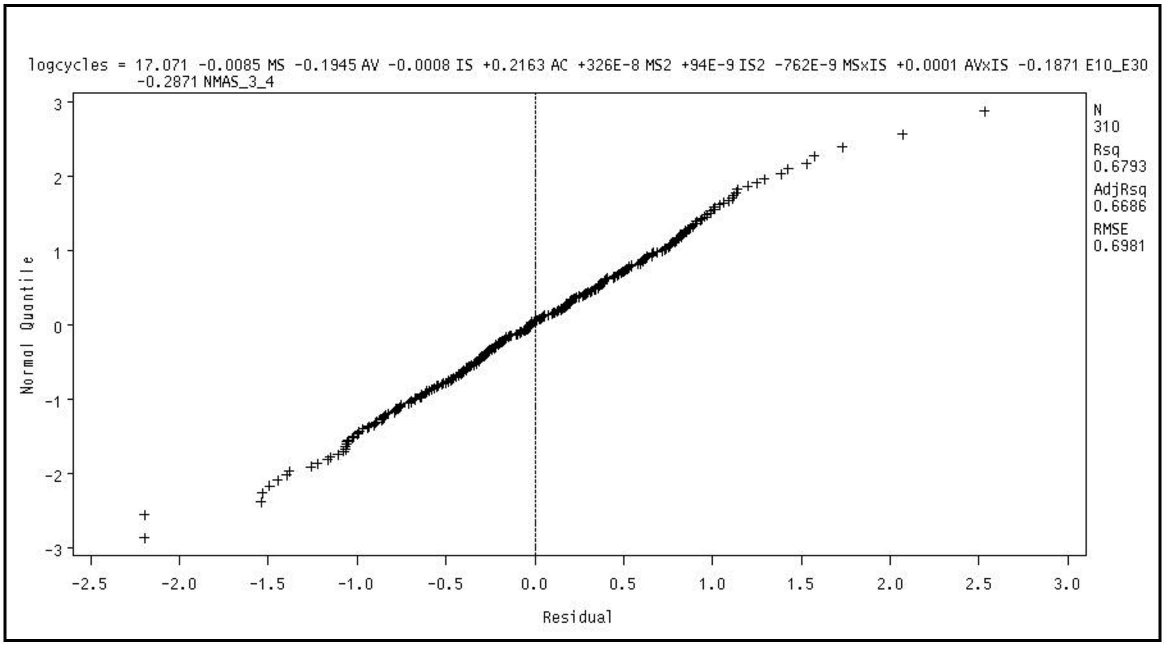


Figure 7.40: Normal Quantile Plot of the Residuals

7.19 Confidence Intervals for the Regression Coefficients

The overall regression equation was constructed using 90% confidence. Confidence intervals were constructed at 90% for the regression coefficients to illustrate the range in which the expected value of the coefficient can be found. A confidence interval is a range in which the true expected value of a certain parameter can be found based on the sample population. Confidence intervals are calculated with the following equation.

$$b_k \ \pm \ t\left(1-\frac{\alpha}{2};\ n-p\right) s\left\{b_k\right\}$$

Where:

b_k is the original regression coefficient for the parameter of interest (asphalt content or air voids),

t (1-α/2; n-p) is the t-statistic at a certain level of confidence, α is the confidence coefficient; either 0.25 or 0.10 for 75 and 90% confidence, respectively,

n, the sample size, is 310,

p, the number of predictor parameters being used, is 11 and

$s(b_k)$ is the standard deviation of the prediction parameter (available in Appendix I).

As previously mentioned, the magnitude of the asphalt coefficient was decreased when the interaction, quadratic, NMAS and traffic level variables were introduced. In Table 7.74 the 90% confidence interval range for the variable AC is 0.11511 to 0.31749. This may mean that the asphalt content has more of a positive effect on the fatigue life than equation 7.28 reveals, but is simply reduced by the other parameters. In conclusion these ranges represent with 90% confidence the true domain of the estimated coefficients for the parameters used in the regression analysis. These 90% confidence limit values for the parameters are summarized in Table 7.74. The 75% confidence interval limits provide a relative comparison of confidence interval width to the 90% confidence interval limits.

Table 7.74: 75 and 90% Confidence Intervals for Regression Coefficients

Variable	t, (1-alpha/2; n-p), 75%	75% Lower CL	75% Upper CL	t, (1-alpha/2; n-p), 90%	90% Lower CL	90% Upper CL
Intercept	1.159	15.82398	18.31710	1.645	15.29593	18.84515
MS		-0.01069	-0.00637		-0.0116	-0.00545
AV		-0.23598	-0.15304		-0.25355	-0.13547
IS		-0.001135301	-0.000410879		-0.00129	-0.00025745
AC		0.14522	0.28738		0.11511	0.31749
MS^2		1.96192E-06	4.55808E-06		1.42000E-06	5.11000E-06
IS^2		6.35128E-08	1.24637E-07		5.05663E-08	1.37584E-07
MSxIS		-9.76018E-07	-5.48038E-07		-1.07000E-06	-4.57389E-07
AVxIS		4.97015E-05	8.29185E-05		4.26600E-05	8.99600E-05
E10_E30		-0.30334	-0.07096		-0.35256	-0.2174
NMAS_3_4		-0.40745	-0.16667		-0.45845	-0.11567

7.20 Useful Information Concerning the Regression Model

Important information concerning the parameters involved in regression models can be made by using partial differentiation. This allows for the relative influence that a single variable imposes on the model while holding other variables constant to be explored. The regression equation was differentiated with respect to initial stiffness. This was done to investigate the effect of changing the air void level had on the resulting stiffness.

7.20.1 Differentiating with Respect to Initial Stiffness

The resulting equation that was obtained from partial differentiation with respect to initial stiffness is presented in equation 7.29:

$$\frac{d\ \text{Log Cycles}}{d\ \text{IS}} = -7.7309\times10^{-4} + 1.8814978\times10^{-7}\,IS - 7.62028\times10^{-7}\,MS + 6.631\times10^{-5}\,AV \qquad (7.29)$$

Where:

$\frac{d\ \text{Log Cycles}}{d\ \text{IS}}$ is the change in the slope of equation 7.28 with respect to initial stiffness,

IS is the initial stiffness (MPa),

MS is the microstrain, and

AV is the air void level, expressed as a percentage.

This equation represents a change in the slope of the overall regression equation 7.28 with respect to initial stiffness. If values for AV, MS and IS are inserted into equation 7.28, the sensitivity of the overall regression equation 7.28 with respect to air voids and initial stiffness can be evaluated. To evaluate the impact on the cycles to failure that results from increasing the air voids by 1%, the above equation was solved using the average values of air voids, microstrain and initial stiffness. The average value of the air voids (8.05) is increased to 9.05 and the average values of initial stiffness (2626.15 MPa) and microstrain (694.19) are used. These values can be calculated from the original regression data that is presented in Appendix I. When these values were used as inputs, the resulting output was -0.00021, which is nearly equal to an increase of fatigue life of 1 cycle when exponentiated. This means that for every 1% increase in air voids, the value of the fatigue life increases by 1 cycle. This increase is hardly significant in magnitude and should be taken with a word of caution. The overall regression equation 4.3 is based on a natural logarithmic scale, which distorts the range of the tested fatigue samples. The maximum length of all of the fatigue tests was 1,115,900 cycles (as can be seen in Appendix I). If the intercept of equation 7.28 is exponentiated, the result is 25,920,377 cycles. This tells us that the model is capable of distorting results that are outside of the tested range of results. As mentioned previously, increasing the air voids too much would result in the pavement to fail prematurely because air voids cannot transfer load.

To examine the true impact that changing the air voids by 1% from the target value has on initial stiffness, equation 7.29 was simply set to zero and solved for initial stiffness. The resulting equation from transforming equation 7.29 is:

$$IS = 4108.907 + 4.050MS - 352.432AV \qquad (7.30)$$

Where:

IS is the intial stiffness of the pavement (MPa),

MS is the microstrain level, and

AV is the air voids, expressed as a percentage.

This equation represents the maximum value that the initial stiffness can have, given that all other variables in equation 7.28 are being held constant with exception to air voids. This illustrates that the initial stiffness was dependent on the air voids, which means that an increase in air voids lowers the initial stiffness and led to a decrease in the fatigue life. Table 7.75 shows the effect that changing the air void content by ± 1% from the observed average value (8.05%) had on initial stiffness when using equation 7.30. Thus, the true effect of changing the air voids by -1% and +1% can lead to an increase in fatigue life of 8,347 cycles and a decrease in fatigue life of 4,371 cycles, respectively. Air voids should be entered into equation 4.5 as a percentage (e.g. 8.05) and the average microstrain (694.19) and asphalt content (5.75%) values was used in the calculations. These values are reasonable ranges for the stiffness of HMA (WSDOT Pavement Design Guide, 1995).

Table 7.75: Resulting Laboratory Fatigue Life from Varying the Average Air Void Content (3 or 4 E 10 or 30 Pavement)

AV, % Change from Average	Resulting Air Voids, %	Initial Stiffness (MPa)	Initial Stiffness (psi)	Laboratory Fatigue Life (Cycles)
1	9.05	3,730.95	541,128	33,516
0	8.05	4,083.38	592,244	37,886
-1	7.05	4,435.81	643,360	46,234

The material characteristics have been quantified through the use of a multiple regression procedure. The main focus of this part of the research was to identify which parameters influence the laboratory fatigue life of hot mix asphalt. The analysis revealed that air voids and asphalt content are the dominating material characteristics, while the nominal maximum aggregate size and the design traffic level were also significant variables.

7.21 Pay Factor Development

For the purposes of this research it was decided to base the fatigue cracking and rutting pay factors on parameters that can be readily measured either in the field or shortly thereafter in a lab. Two main parameters that can be directly related to fatigue life are the air void content and the asphalt content. Two main parameters that can be directly measured or calculated for rutting are asphalt content and voids in mineral aggregate (VMA). As mentioned previously, air voids cannot transfer a load. The relative compaction is a measure of the air void level in a pavement. The higher the target air voids level, the lower the fatigue life. The target air void level in the field is usually 7%, with a standard deviation of about 1.2% (Deacon et. al., 1997). As asphalt content is increased, the fatigue life is also increased.

7.22 Pay Factor Development Process

The steps taken to develop pay factors for the State of Michigan with respect to fatigue cracking will be discussed herein. The procedure for developing pay factors for rutting are similar to that for fatigue cracking. Another statistical procedure called Monte Carlo simulation will be discussed. The Monte Carlo simulations were performed using a Microsoft Excel® based software package called @Risk© prepared by Palisade in February, 2001. The pay factors for MDOT were developed using many of the same techniques and equations that were presented by Deacon et al. in 1997. The work herein diverges from the aforementioned research in that aggregate size and traffic level design were taken into consideration when developing the proposed pay factors.

7.22.1 Monte Carlo Simulation of Laboratory Fatigue Life and Rutting

Monte Carlo simulation is a technique that can be used to test the accuracy of a given model. Monte Carlo simulation can also be used to analyze the risk involved given certain characteristics with respect to well-defined variables. Risk is a measure of uncertainty about an event. Therefore, past events have zero risk and future events can have varying degrees of risk. For the purposes of this research project the overall regression equation (Eqn. 7.19 and Eqn. 7.28) was used as a means of generating the laboratory rutting and fatigue life. The type of Monte Carlo simulation used is called Classic Monte Carlo sampling. This method takes an entire defined distribution and generates random samples from that distribution. This method is not as time efficient as some other methods, but with the software package available, this time differential was negligible. This was discovered by trying different sampling methods available in the software package.

In their 1997 research report Deacon et al. used the values shown in Table 7.76 as construction expectations in the calculation of their fatigue life. The air void and asphalt content standard deviations were based on previous in-situ data from California. Since VMA was not calculated in the study by Deacon, the mean and standard deviation from this research project were used in the Monte Carlo simulation.

Table 7.76: Deacon et al. Targeted Air Voids and Asphalt Content

	Air Voids, %	Asphalt Content, %
Target	7	5
Std. Dev.	1.2	0.19

For the purposes of this research the average asphalt content and standard deviation was calculated for each of the four groups mentioned in the revised analysis plan. This information was taken from the targeted asphalt contents used to mix and compact the beams in the laboratory. The data is presented below in Table 7.77. The numbers in parentheses are the average and the standard deviation, respectively, of the asphalt content. For the input value of the target air voids content the average value was used (8.05%). The targeted air void content throughout all simulations was 8.05%±1.2% (target value and expected standard deviation during construction, respectively). The reason that the standard deviation of 1.2% was used is that actual data taken from field cores was not available for the Michigan paving projects, and thus Deacon's values were used. It must be explained that the he Monte Carlo simulation generates random samples that go beyond the specified standard deviation and generates values that represent extreme observations in the distribution. This should account for any differences between the value of 1.2% used and the true value of the standard deviation for air voids of in-situ HMA pavements in Michigan. However, if the true standard deviation can be established, it should be used.

Table 7.77: Asphalt Content QC/QA Data (Hofmann, 2002)

		Traffic Level	
		E3	**E10 and E30**
Nominal Maximum Aggregate Size (NMAS)	**(3) 19.0mm, 3/4" and (4) 12.5mm, 1/2"**	(5.4, 0.211)	(5.1, 0.178)
	(5) 9.5mm, 3/8"	(6.4, 0.265)	(6.0, 0.144)
Weighted Average of Asphalt Content Standard Deviations in Michigan, %			0.1905

As indicated in Table 7.77, the standard deviations of the asphalt content for construction in Michigan were similar to that of Deacon's. For the purposes of this research the standard deviation for all modeled asphalt contents were set to 0.19% in the Monte Carlo simulation, to mimic actual field conditions. The average percent difference was calculated by dividing the standard deviation by the mean of asphalt content for each pavement type in Table 7.77. Then, the overall average asphalt content was found as well as the overall percent difference. The overall percent difference was multiplied by the overall average and then divided by 100 to get a representative standard deviation for the asphalt content in the State of Michigan. The actual standard deviation of asphalt content from Hofmann's QC/QA data was 0.1905%. Therefore, Hofmann's QC/QA data is identical to the 0.19% standard deviation for asphalt content value used by Deacon et al. in 1997. To simulate the estimated laboratory fatigue life, the values of the microstrain and stiffness and either the air voids or the asphalt content were held constant in regression equation 7.28. The reason for doing this was to calculate the impact that either the air voids or asphalt content would have on the fatigue life and ultimately, the resulting pay factors. The software then randomly generated a value from this distribution for either the asphalt content or air voids and then calculated the resulting cycles to failure. The models ranged anywhere from 1,800 to 10,000 simulations to determine the estimates of fatigue life using the Monte Carlo method.

7.22.2 ESAL Calculation Based on Simulated Fatigue Life

The next step was to convert the simulated laboratory off-target fatigue lives (the estimate of the fatigue life that was calculated by the regression equation due to a randomly generated observation of air voids or asphalt content) to ESALs. This was done using a shift factor (SF) and a temperature conversion factor (TCF) that was based on work that was first done by Deacon et al. in 1997. Both equations for the SF and the TCF are dependent upon pavement thickness. Therefore, it was necessary to determine an appropriate thickness using elements of the AASHTO pavement design method and a software package called Win-Pas that was developed by the American Concrete Paving Association.

Asphalt layer thicknesses were determined for each traffic level loading (E3, E10 and E30). This was done primarily because traffic loading is a key consideration in a pavement design problem. The software then calculated a required structural number (SN) based on the inputs. Then a pavement system was designed and the layer thicknesses needed that provided an SN equal to or greater than that required by the traffic loading was calculated. It should be noted that for the purposes of these pavement designs, a level of reliability of 95% was chosen from the possible range of 85-99.9% for an urban freeway. This refers to the probability that the pavement will perform to the design expectations over a given amount of time. The 1993 AASHTO design guide suggests that an overall standard deviation of 0.45 be used for flexible pavements. However, an overall deviation of 0.50 was chosen due to the fact that no project data was available; this is suggested by the 1995 WSDOT Pavement Guide. The soil resilient modulus was an assumed value of 5,000 psi. Serviceability levels are also required inputs represent the ability of a pavement to accommodate the design traffic levels. This is measured by a present serviceability index (PSI). The PSI ranges from 0 (a road that cannot be traversed) to 5 (excellent roadway conditions). The WSDOT Pavement Guide suggests the use of 4.5 and 3.0 for the initial and terminal serviceability's for an urban freeway. Table 7.78 shows the pavement design inputs for the structural number determination as calculated by the Win-Pas software.

Table 7.78: Win-Pas Software Input Parameters for Structural Number Determination

Design ESALs	3, 10 or 30 million
Reliability	95
Overall Deviation	0.50
Soil Resilient Modulus (psi)	5,000
Initial Serviceability, Po	4,5
Terminal Serviceability, Pt	3

To calculate the thicknesses of the layers that support the pavement, some assumptions were made. First, all supporting layers such as the aggregate base and subbase had equal thicknesses, regardless of the traffic loading. The supporting layer thicknesses are shown in Figure 7.41. Second, the elastic moduli of the base and subbase materials were estimated using reasonable judgment. The drainage coefficients for the base and subbase were taken from the 1993 AASHTO design guide. For the subbase it was assumed that the drainage quality would be poor, meaning that 5-25% of the time this layer is approaching saturation. This was done to err on the side of caution concerning the subbase. For the base it was assumed that the drainage quality would be fair, meaning that 1-5% of the time that this layer approaching saturation. This assumption was used because the base material should not be quite as exposed higher moisture levels like the subbase would. The calculations for the layer coefficients were also taken from the AASHTO 1993 pavement design guide. Table 7.79 summarizes the assumed values used in the analysis.

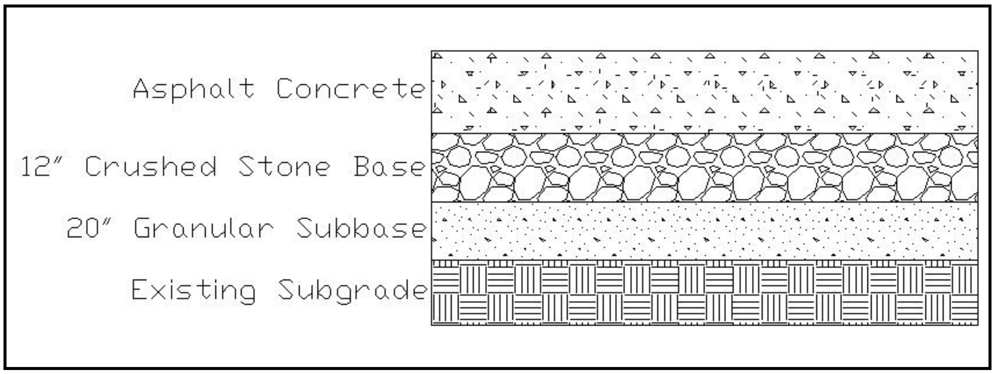


Figure 7.41: Pavement Design Layers

Table 7.79: Layer and Drainage Coefficient Determination

Material Description	Elastic Modulus	Calculation Equations for Layer Coefficients	Layer Coefficient	Drainage Coefficient
Hot Mix AC Surface	(E = 378 ksi)	$0.4*LOG(E_{base}/450)+0.44$	0.410	N/A
Untreated Permeable Crushed Base	(E = 35,000 psi)	$0.249*LOG(E_{@optimum MC})-0.977$	0.154	1.2
Dense Graded Untreated Subbase	(E = 20,000 psi)	$0.227*LOG(E_{@optimum MC})-0.839$	0.137	0.6

Finally, to determine the asphalt layer thickness, the layer coefficients, drainage coefficients and assumed layer thicknesses for the base and subbase materials were entered. The resulting asphalt layer thickness and the structural number of the pavement system was calculated and then verified that it was equal to or greater than that calculated by the values in Table 7.78. It was determined that E3 pavements needed 4 inches of asphalt, E10 pavements needed 6.5 inches of asphalt and E30 needed 8.5 inches of asphalt. The results of the pavement design for the development of the pay factors are presented in Appendix J.

After the resulting asphalt layer thicknesses were calculated, the simulated laboratory fatigue lives could be converted to ESALs. The equations proposed by Deacon et al. in 1997 were:

$$SF = 0.3771(t^2) - 2.6109(t) + 7.5121 \quad \text{for } 3.6 \le t \le 12 \tag{7.31}$$

$$TCF = 1.754\ln(\text{t}) - 1.256 \quad \text{for } t \ge 4 \tag{7.32}$$

In the above equations the units for t are inches. The designed thickness produced the following shift and temperature conversion factors for the different traffic loading levels shown in Table 7.80. To calculate the ESALs at each traffic level, the laboratory fatigue life was multiplied by the SF and then divided by the TCF (see equation 2.4).

Table 7.80: SF and TCF Calculations Based on Asphalt Layer Thickness

Traffic Level	Thickness (inches)	SF	TCF
E3	4	3.1021	1.1756
E10	6.5	6.4737	2.0271
E30	8.5	12.5649	2.4977

7.22.3 Change in Present Worth of First Rehabilitation Cycle

After the fatigue life and the resulting ESALs were calculated, a means of determining how long this "as-constructed" pavement would last and what effect that life would have on the owner agency costs were necessary. Deacon et al. proposed the following three equations in 1997.

$$RP = \frac{\text{Off-Target ESALs}}{\text{On-Target ESALs}} \tag{7.33}$$

$$OTY = \frac{\ln\left(1 + RP\left[\left(1+g\right)^{TY} - 1\right]\right)}{\ln\left(1+g\right)} \quad (7.34)$$

$$\Delta PW = 100\left(\frac{1+d}{1+r}\right)^{TY}\left(\frac{1+r}{1+d}\right)^{OTY} - 100 \quad (7.35)$$

Relative performance (RP) is simply a ratio of the simulated cycles to failure, which was converted to ESALs divided by the corresponding design ESALs (3, 10 or 30 million. The off-target year (OTY) takes into consideration the relative performance of the constructed pavement, the traffic growth (g) and the target year (TY). In all of the simulations for each level of traffic loading (e.g. E3, E10 and E30) the target year was 20. The change in present worth (ΔPW) is the percentage change of the cost of the first rehabilitation cycle. R is the rate of construction cost inflation and d is the discount rate. G, r and d are percentages, but are entered into the equations as decimals. The following values in Table 7.81 were used in the above formulas for all simulations.

Table 7.81: Simulation Constants for Change in Present Worth Calculations

TY	Traffic growth	Discount Rate	Rate of Inflation
20	0.025	0.05	0.02

Thus the pay factor system was based on the change in present worth of rehabilitation costs that the owner agency will incur that result from the as-constructed pavement. Please note that in all following sections a positive change in present worth represents a payment penalty. This is because a positive value signifies an increase in the cost to the owner for the first rehabilitation cycle. A present worth of zero represents a full payment situation, or the owner not having to pay for any rehabilitation cost prior to the end of the 20-year design life. A negative present worth represents a payment bonus situation. This signifies that the contractor has built a pavement that should save the owner money on the first rehabilitation cycle, because the performance parameters indicate that the pavement will last longer than the intended design life of 20 years for a given traffic loading level.

7.23 Presentation of the Developed Pay Factors

The following sections present and discuss the pay factors developed for MDOT with respect to the various design traffic and NMAS levels. The pay factors for each type of paving project were created separately for both air voids and asphalt content and VMA and asphalt content. The final pay factor system that utilizes both parameters will be explained herein.

7.23.1 Working-Hotelling Confidence Bands for Pay Factor Development

Confidence bands can be used for simple linear regression functions. Confidence bands are similar to confidence intervals in that they encompass a numeric region around a certain point of interest (Neter et al., 1996). However, confidence bands are more inclusive than confidence intervals in that the former generates an interval that also encompasses the entire regression line. In this case the percent change in present worth was calculated from either randomly generated air voids or asphalt contents. A simple linear regression relationship can be derived by either regressing asphalt content or air voids against the percent change in present worth of the first rehabilitation cycle. The reason that the confidence bands were used was to quantify a level of certainty about the payment interval with respect to the performance parameters (asphalt content and air voids). This was especially important when considering the award of a payment bonus or penalty. The confidence band line allows the owner to say with a certain level of confidence that the as-constructed pavement should last a certain amount of time and is therefore, deserving of a certain pay factor (bonus or penalty).

The particular method used was the Working-Hotelling 1-α confidence band. It utilizes a "W" statistic that is based on the F distribution. This again provides a larger interval than a simple confidence interval that uses a simple t-statistic (Neter et al., 1996). A Working-Hotelling confidence band is calculated with the following equations.

$$W^2 = 2F\left(1-\alpha;\ 2,\ n-2\right) \tag{7.36}$$

$$b_0 + b_1 X \ \pm\ W\sqrt{MSE}\left[\frac{1}{n}+\frac{\left(X-\bar{X}\right)^2}{\sum\left(X_i-\bar{X}\right)^2}\right]^{\left(\frac{1}{2}\right)} \tag{7.37}$$

Where:

W is the test statistic used,

F is the tabular F-value used (Neter et al., 1996),

α is the confidence coefficient equal to 1-confidence/100,

n is the sample size,

X is observed level of air voids or asphalt content,

$\bar{X}$ is the mean of the predictor variable (either asphalt content or air voids)

MSE is the mean square error of the regression between the change in present worth and either asphalt content or air voids, and

b_0 and b_1 are the intercept and slope, respectively, of the regression relationship between the percent change in present worth and either the asphalt content or air voids.

Equation 7.37 generates the confidence band. The first half of the equation is simply the regression relation between the change in present worth (Y) and the performance parameter (X;

air void or asphalt content). The second half of the equation shifts the value of the change in present worth in either a positive or negative direction along the y-axis, which in turn, generates the confidence band for the pay factor about a certain level of X. Three levels of confidence were used to generate confidence bands. The levels were: 90, 95 and 99.9% confidence. The bands widen as the level of confidence increases. Therefore, the 99.9% confidence band represents the most conservative pay factor in terms of awarding a payment bonus. An example of what the pay factor confidence bands look like is shown below in Figure 7.42. Figure 7.42 merely illustrates how the bands increase in size as the level of confidence increases. The numeric spread for the band may not be as large as shown in Figure 7.42.

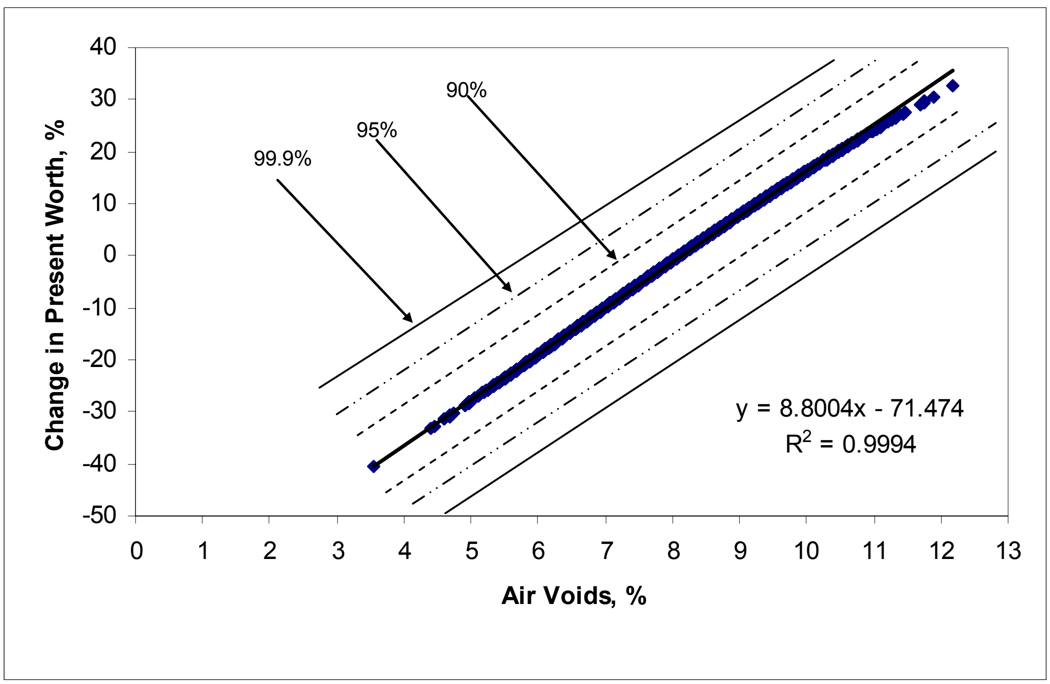


Figure 7.42: Example Plot of Confidence Bands for Pay Factor Development

7.23.2 An Example: 3 and 4 E3 Pavement Pay Factor Development

This section discusses the 3/4E3 pavement type and the resulting pay factors. This section entails a pay factor table that represents the entire range of asphalt and air void contents modeled and the resulting changes in present worth of the first rehabilitation cycle at each confidence band. The table shows the performance parameter of interest as well as the expected value of the present worth for the first rehabilitation, and finally the three confidence bands. Also presented is a graph that illustrates the general trend of each performance parameter regressed against the present worth variable. On the graphs there are two important features to notice. The first is a simple linear regression equation that can be used to calculate a pay factor, or change in present worth, at any point on the regression line. It is not recommended that the regression lines be used to extrapolate the data to generate a pay factor. This would be using the regression for a point that was not in the original data set and could result in significant error. The second key feature is the R^2 value, which is an indication of how well the dependent variable (change in present worth) is explained by an independent variable (either asphalt content or air voids). This first pavement type is illustrated here as an example of how the pay factors were developed. All remaining pay factors for every other pavement type are listed in Appendix J. This was done to avoid the redundancy of explaining similar results. Where the remaining pavement type pay factor information differs is in the regression equations and the range of pay factors developed. Also, the complete details of the calculation of the confidence bands and the regression equations used for the development of the pay factors can be found in Appendix J.

The 3/4E3 pavements had ranges of 3.5-12% and 4.7-6.0% for the air voids and asphalt contents, respectively. This resulted in pay factors that ranged from -28% to +34% for air voids and -6.5% to +6.4% for asphalt content. The figures below represent the ranges of the pay factors when regressed against the Monte Carlo simulated asphalt and air void contents. It is important to note that, as expected, an increase in asphalt content led to a decrease in the present worth of the first rehabilitation and that an increase in the air voids leads to an increase in the present worth of the first rehabilitation.

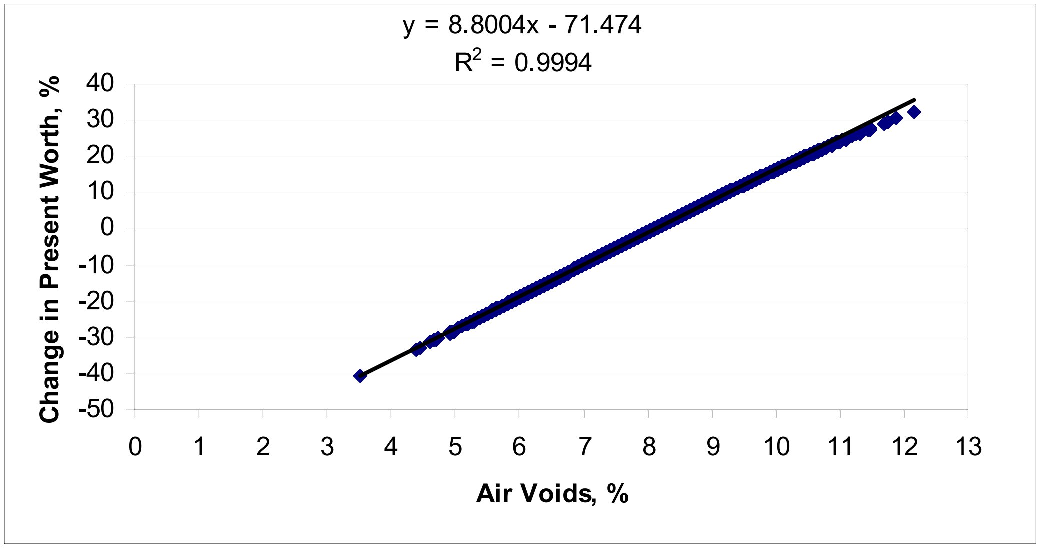


Figure 7.43: Change in Present Worth with Respect to Air Voids for 3/4E3 Pavements

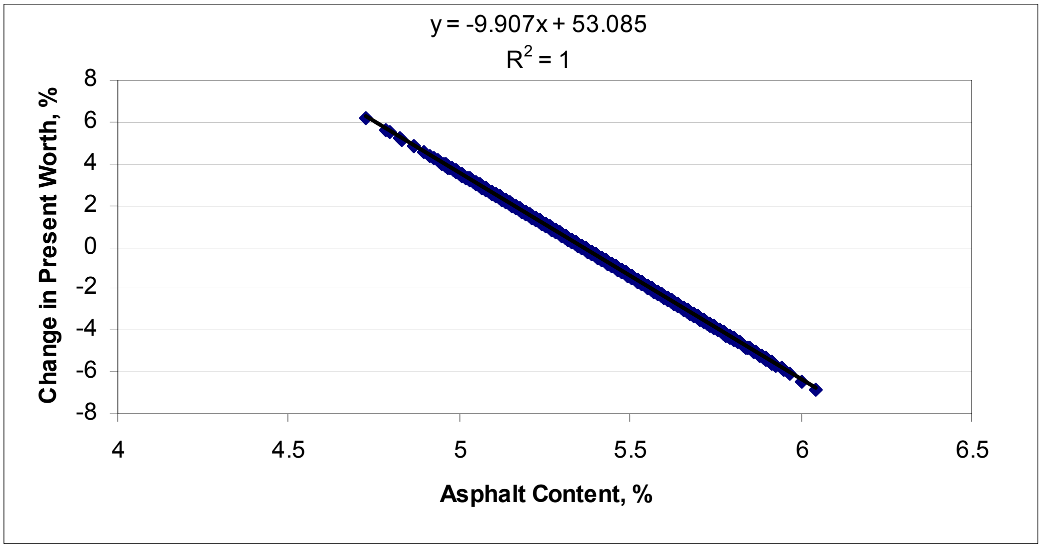


Figure 7.44: Change in Present Worth with Respect to Asphalt Content for 3/4E3 Pavements

Table 7.82 illustrates that the confidence bands are really quite narrow in magnitude, despite that they are in fact wider than confidence intervals. When making a plot of all of the generated confidence bands for any type of pavement, the lines were nearly indistinguishable, so a representative table was developed that captured the entire distribution of the simulated performance parameters and the resulting pay factors. The confidence bands do not take into account the confidence in the developed regression model equation 7.28 (e.g., the evaluation of the variable coefficients at 75 and 90% confidence), but only evaluate the precision of the simulation. The air voids were listed in increments that increased by 0.5% and the asphalt content was listed as increasing by 0.1%, as opposed to listing all of the simulated results that generated independent performance parameters. After the asphalt and air void contents are simply calculated from the HMA placed in the field, the appropriate pay factor is read from the table at the appropriate level of confidence the owner desires to use. The pay factor simulation process was shown to be very accurate and repeatable.

Table 7.82: Pay Factors for 3/4E3 Pavements Resulting from Various Air Void and Asphalt Contents

Air Voids, %	Change PW, %	99.9% Lower Confidence Band	99.9% Upper Confidence Band	95% Lower Confidence Band	95% Lower Confidence Band	90% Lower Confidence Band	90% Lower Confidence Band
5.0	-27.4726	-27.5176	-27.4277	-27.5023	-27.4430	-27.4986	-27.4467
5.5	-23.0725	-23.1111	-23.0338	-23.0979	-23.0470	-23.0948	-23.0501
6.0	-18.6723	-18.7049	-18.6397	-18.6938	-18.6508	-18.6911	-18.6535
6.5	-14.2721	-14.2990	-14.2452	-14.2898	-14.2544	-14.2876	-14.2566
7.0	-9.8719	-9.8938	-9.8500	-9.8863	-9.8575	-9.8845	-9.8593
7.5	-5.4717	-5.4899	-5.4536	-5.4837	-5.4598	-5.4822	-5.4613
8.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
8.5	3.3286	3.3110	3.3463	3.3170	3.3403	3.3184	3.3388
9.0	7.7288	7.7077	7.7499	7.7149	7.7427	7.7166	7.7410
9.5	12.1290	12.1031	12.1550	12.1119	12.1461	12.1140	12.1440
10.0	16.5292	16.4976	16.5607	16.5084	16.5500	16.5110	16.5474
10.5	20.9294	20.8918	20.9669	20.9046	20.9541	20.9077	20.9510
11.0	25.3295	25.2857	25.3734	25.3007	25.3584	25.3043	25.3548
11.5	29.7297	29.6795	29.7800	29.6966	29.7629	29.7007	29.7587
12.0	34.1299	34.0731	34.1867	34.0925	34.1673	34.0971	34.1627

Asphalt Content, %	Change PW, %	99.9% Lower Confidence Band	99.9% Upper Confidence Band	95% Lower Confidence Band	95% Upper Confidence Band	90% Lower Confidence Band	90% Upper Confidence Band
4.7	6.5224	6.5204	6.5245	6.5211	6.5238	6.5212	6.5236
4.8	5.5317	5.5299	5.5335	5.5306	5.5329	5.5307	5.5328
4.9	4.5410	4.5395	4.5425	4.5400	4.5420	4.5402	4.5419
5.0	3.5503	3.5491	3.5516	3.5495	3.5512	3.5496	3.5511
5.1	2.5596	2.5586	2.5606	2.5590	2.5603	2.5590	2.5602
5.2	1.5689	1.5681	1.5697	1.5684	1.5694	1.5685	1.5694
5.3	0.5782	0.5776	0.5788	0.5778	0.5786	0.5779	0.5786
5.4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5.5	-1.4032	-1.4038	-1.4026	-1.4036	-1.4028	-1.4035	-1.4028
5.6	-2.3939	-2.3947	-2.3931	-2.3944	-2.3934	-2.3943	-2.3934
5.7	-3.3846	-3.3856	-3.3836	-3.3852	-3.3839	-3.3852	-3.3840
5.8	-4.3753	-4.3765	-4.3740	-4.3761	-4.3745	-4.3760	-4.3746
5.9	-5.3660	-5.3675	-5.3645	-5.3670	-5.3650	-5.3669	-5.3651
6.0	-6.3567	-6.3585	-6.3549	-6.3579	-6.3555	-6.3577	-6.3557

7.24 Commentary on the Proposed Pay Factors

Since the air voids consistently yielded a broader range of pay factors, and hence, a more revealing prediction of the fatigue life (for the ranges of air voids experimented with in this research project), an owner/agency may consider using this as the more important factor for determining the pay factor. Another possibility would be to use a weighted average of the pay factors developed with respect to air voids and asphalt content. In terms of rutting the range of pay factors is the approximately the same, therefore an owner/agency may consider using a weighted average of the pay factors developed.

It has been shown that the statistically based pay factors developed herein can be generated with target values for air voids and asphalt content and VMA. In addition, a certain level of reliability can be used to predict the pavement performance and to assign a pay factor accordingly. If an owner agency wants to be conservative with respect to air voids or asphalt content when awarding a bonus or a penalty, the 99.9% confidence band pay factor can be used. The confidence bands merely serve as a boundary for the pay factors as opposed to capping the pay factors at some percentage. Of course, this decision is always up to the owner/agency paying for the work to be built. As always, the owner has the right to stipulate that if the performance parameters are gravely out of specification, the HMA material can be removed and replaced.

It should be noted that the developed pay factors do not increase in numerically equal increments when deviating away from the target value. This is due to the way the change in present worth is calculated. Equation 7.35 uses an exponential function to calculate the change in present worth. Therefore, the change resulted in a linear relationship when regressed against either asphalt content or air voids, but the incremental change away from the target value is not numerically equal.

7.24.1 Confidence Levels Concerning Pay Factors

The sensitivity of the level of confidence with respect to pay factor development was analyzed by utilizing different coefficients in equation 7.28. This was accomplished by developing confidence coefficients for asphalt content and air voids. As was previously mentioned in chapter 4, 75 and 90% confidence intervals concerning the regression coefficients were developed. For the purposes of sensitivity testing, both 75 and 90% confidence interval coefficients were used.

The sensitivity analysis was accomplished by using regression equation 4.3 and substituting the appropriate confidence coefficients into the regression equation and then simulating the effect that these coefficients had on the development of the change in the cost of the present worth of the first rehabilitation cycle. An example of the results for a 3/4E3 pavement type is shown below. As can be seen from figures 7.44 and 7.45, using different confidence coefficients has a much more profound effect on the resulting change in the present worth of the first rehabilitation cycle than does the use of the Working-Hotelling confidence bands, and therefore, may reveal what the true pay factors should be. Table 7.83 provides the regression equations for the trend lines shown in the figures. Therefore, an owner may truly access with an even more conservative level of confidence the possible resulting worth of the first rehabilitation cycle.

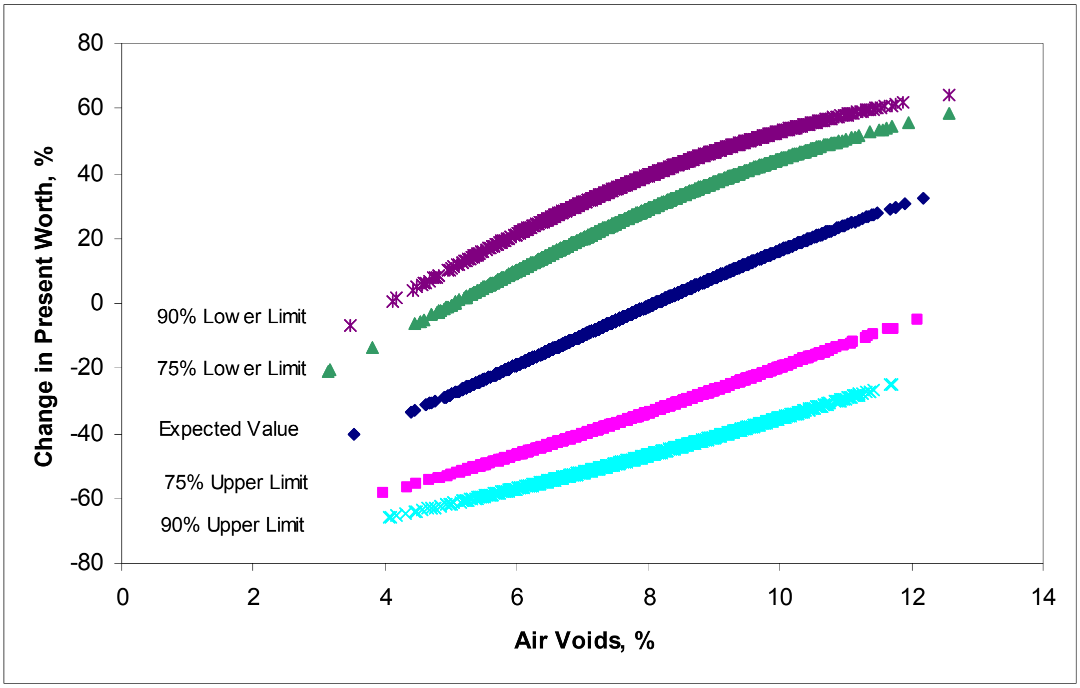


Figure 7.45: Confidence Sensitivity of Air Void Levels for a 3/4E3 Pavement

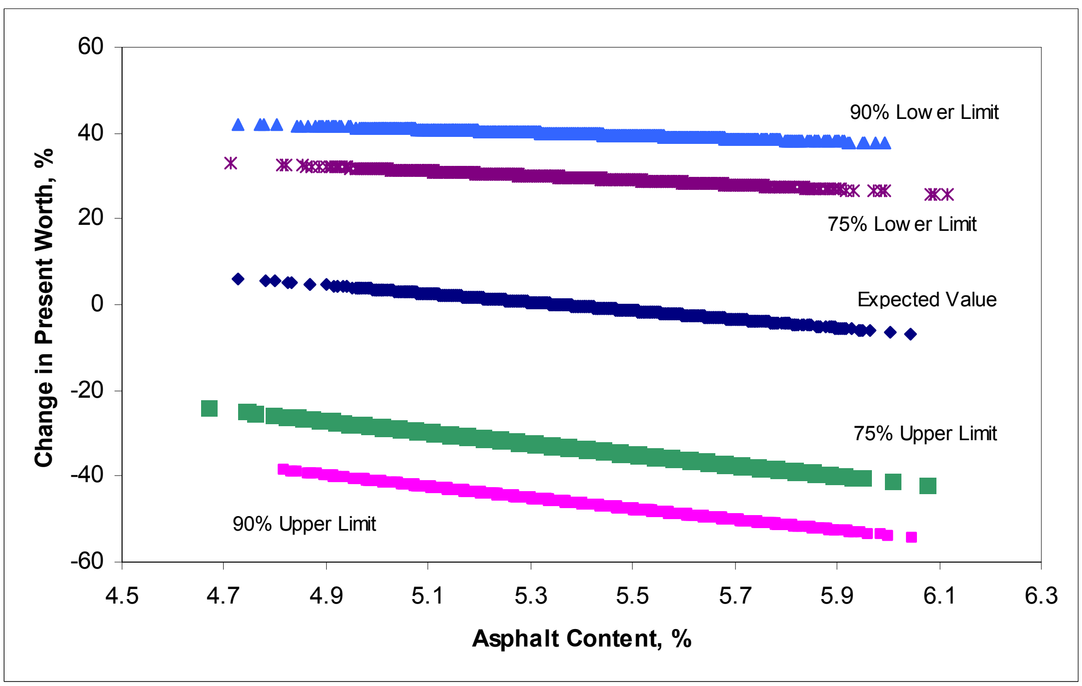


Figure 7.46: Confidence Sensitivity of Asphalt Content for a 3/4E3 Pavement

Table 7.83: Regression Equations for 75 and 90% Confidence Level Sensitivity

	Confidence Limit	Regression Equation for ΔPW
Asphalt Content (X)	90% Lower	58.985 - 3.5563(X)
	75% Lower	58.071 - 5.2824(X)
	Expected Value	53.085 - 9.907(X)
	75% Upper	35.327 - 12.751(X)
	90% Upper	22.632 - 12.791(X)
Air Void Content (X)	**Confidence Limit**	**Regression Equation for ΔPW**
	90% Lower	7.8247(X) - 24.045
	75% Lower	8.5877(X) - 40.322
	Expected Value	8.8004(X) - 71.474
	75% Upper	6.7346(X) - 87.622
	90% Upper	5.4437(X) - 90.11

CHAPTER 8 SUMMARY

8.1 Introduction

This project has successfully completed the first three phases of the research project encompassing a literature review, development of APA specifications and its use to determine rut potential of MDOT mixes, development of experimental plans and field sampling of materials on projects throughout Michigan, and the use of the four point beam fatigue to determine the fatigue life of MDOT mixes. As the project and other technologies evolve, the research team will continue to provide updates to the literature review portion of the project and examine statistical hypotheses for testing and update the experiments based on a continuing dialogue with the technical advisory group.

8.2 Performance Testing for Design

Identification of a performance test or set of tests to accompany the Superpave volumetric mix design procedures is needed. Originally, Superpave was developed for three trafficking levels: low, medium and high. Performance tests were to accompany the volumetric design criteria for medium and high trafficking levels. For numerous reasons, which include test time and cost, these tests were not implemented. Six years after the completion of the Strategic Highway Research Program and initial implementation of Superpave, there has not been a set of performance tests on the national level for implementation. This need still exists today.

The approach of placing as much asphalt in an aggregate structure without creating a rutting problem is a valid approach for use with the Michigan Department of Transportation HMA specifications for medium and high trafficking roadways. This approach is also reasonable for use in quality control/quality assurance specification and percent within limits.

The future specifications of HMA are not well known, but there are three general approaches which could be used: performance-related specifications, performance-based specification, and warranties. These advanced specification methods should rely on a rutting and fatigue cracking performance tests for HMA as often there are performance tradeoffs between the two distress modes. The more advanced specifications will link construction specifications to mixture and pavement design specifications through laboratory performance testing.

8.3 Preliminary Asphalt Pavement Analyzer Specifications

The research team has identified a preliminary Asphalt Pavement Analyzer specification to accompany the Superpave mix design procedures used in Michigan. The basis for this preliminary specification is based primarily on other agencies' specifications. The test conditions and specifications are summarized in Table 8.1.

Table 8.1 Asphalt Pavement Analyzer Test Conditions and Mixture Design Specifications

Parameter	Specification
Test Temperature	98% Level of Reliability for Pavement Environment
Environmental Condition	Dry
Specimen Size, mm	150Dx75H (SGC) or 300Lx125Wx75H (AVC)
Load, lbs	100
Hose Pressure, psi	100
Wheel Speed, ft/sec	2.0
Number of Conditioning Wheel Load Cycles	50
Number of Test Wheel Load Cycles	8000
Air Voids, %	7+/-1
Permanent Strain in Sample	<3mm (machine measurement)

The test temperature corresponding with the 98% level of reliability essentially is the high temperature of the binder grade but does not include the effect of grade bumping for slow moving or stopped trafficked areas nor the very high number of design ESAL levels. The dry test environment is specified as the test is being used strictly as an indicator of rutting potential. Testing in a wet environment would be testing designs for rutting and moisture damage potential. AASHTO T-283, "Resistance of Compacted Bituminous Mixture to Moisture Induced Damage," is a currently specified test used to evaluate a mixture's moisture susceptibility. The specimens that can be evaluated are ones compacted in a Superpave Gyratory Compactor (SGC) or an Asphalt Vibratory Compactor (AVC) with 7+/-1% air voids. The APA configuration is a 100lb load, 100psi hose pressure and 2.0fps rate of loading, which are the standards nearly all agencies are specifying. The number of conditioning wheel load cycles are 50 and the testing number of wheel load cycles of an additional 8000. The permanent strain in the HMA specimen must be less than 3mm as measured by the APA. The APA used to test specimens should be calibrated monthly according to the procedures outlined in the APA User's Manual (74).

8.4 Sampling

Analysis of the QC/QA test data indicates a distinct difference between truck and paver characteristics. This is very important for a variety of reasons. Sampling from the paver is the most representative of what an owner/agency purchases, however it is more time consuming and in most cases more work. Sampling from the trucks is more convenient. Certain mix designs exhibit a greater distinction between properties in the truck samples and the ending mix properties. For example, the mixes with a nominal maximum aggregate size (NMAS) of 3 resulted in a greater difference between sampling compared to the mixes with a NMAS of 5. Perhaps through further research correction factors can be developed to be applied to the change in mix properties from truck to paver. This would allow contractors and highway agencies a better idea of a mixes performance.

Sampling is time consuming for both parties. The amount and frequency should be debated based upon the scale of the project and the mix design. There is a definite need for faster and more advanced pavement performing testing.

8.5 Verification of Mix Designs

Chapter three outlines the mix design verifications following the Michigan Department of Transportation mix design criteria. The intent of the verification process is for the for future use in performance testing of the laboratory designs.

The research team was unable to obtain a mixture design for only one of the 20 projects (US-31, Elk Rapids). This mix because of its aggregate characteristics (gradation) coupled with its insensitivity to changes in binder content is indicating that a target air void content of 4% could not be achieved with a reasonable amount of asphalt binder. Ensuing performance tests of this mix design coupled with field performance will determine the quality of this mix.

Four other mixes had optimum asphalt binder contents outside of the +/-0.3% tolerance for MDOT's mix design verification, but in all instances were within +/-0.5%. Three mixes had VMA values below their respective MDOT minimum design thresholds. However, two of the three mixes did have JMF values relatively close to the minimum and thus the mix designers may be designer too close to the design criteria considering

VMA typically drops during field production. Only one mix was outside of the MDOT mix design VFA criteria.

8.6 Characterization of Asphalt Binders

Superpave binder grading and penetration testing of tank binder was done in accordance with AASHTO Standards as were the recovered asphalt binders. The abson method was used to recover HMA binders. The recovered binder samples were not aged in the rolling thin film oven as this test simulates the production and construction aging of the asphalt binder, which the recovered samples had already been subjected to. Thus, it was not possible to test the "virgin" state of the recovered samples nor reasonable to age the samples in the rolling thin film oven.

The statistical analysis demonstrates penetration grading of recovered binder samples has a weak relationship to tank binders for both truck and plate samples when grouping all of the binders, unmodified and polymer modified, together. Statistical comparison of unmodified binder penetration values between tank and truck as well as tank and plate samples appear to be more reasonable. The comparison of modified binder penetration values between tank and truck, and tank and paver are very poor.

The properties of recovered plate samples compare well with tank binders as measure in a dynamic shear rheometer (DSR) and the stiffness using the bending beam rheometer (BBR). Comparison of the m-values between sample types was generally poor. There was added statistical benefit in separating the polymer modified binders and the non-modified binders into two groups for the DSR and BBR test results for comparison of paver properties to tank properties. The truck sample properties were found to be overall weak in their relationship to tank sample properties. Closer examination of the truck and paver binder properties indicate that their may be some non-uniform aging of binders during the haul and placement time of mixes.

8.7 Hot Mix Asphalt and Aggregate Characterization of Truck and Paver Samples

Overall the comparison of truck and paver samples shows the two are close to each other as measured by volumetric properties and aggregate characteristics. The research team projects that a statistical analysis would demonstrate no statistical difference based on a 95% confidence limit.

Specifically only the gap-graded Superpave mix, or SMA, was outside of the construction tolerance of the #4 sieve and two mixes were outside of the tolerance of the #200 sieve. When the gradations of the mixes are compared to the mix design tolerances for the #8 and #30 sieve, only the gap-graded Superpave mix is outside the tolerance for the #8 sieve.

Characterization of the recovered fine aggregate via the fine aggregate angularity (FAA) found six of the 20 projects which were below the specified minimum. Closer examination finds that the FAA criteria is not being met for 6 of the 10 E10 and E30 mixes, while all of the E1 and E3 FAA criteria is being met. The lower FAA values could also lead to lower air voids at N_{design}.

In terms of volumetric properties, 16 mixes did meet the asphalt binder content tolerance band for either the paver or truck samples based on the average of four samples. The asphalt binder contents in production appear to be targeted towards the lower bound of the Michigan Department of Transportation tolerance band. Based on a QC/QA approach for payment, this tendency towards the lower tolerance band for asphalt binder would be somewhat expected as the asphalt binder is the most expensive bulk material component in the hot mix asphalt.

The research team did see many mixes fall outside of the 4+/-0.5% air void criteria at N_{design} with most of these mixes being low. This could be in part due to normal multi-laboratory variability or the intrinsic error in comparing back-calculated air voids at N_{design} to air voids obtained by compacting the specimens to N_{design}. Similar to the asphalt binder content, 16 mixes did obtain the minimum VMA specified by the Michigan Department of Transportation as well. It is important to understand that VMA is influenced by both asphalt binder content and air voids as VMA is simply a calculation of the non-absorbed asphalt binder and the air voids.

8.8 Pay Factor System for Mix Design

@Risk proved to be a useful tool in determining a mix designs performance. By visualizing which factors dominate in pay deduction, changes in the mix designs can be made to minimize these deductions. It also could be a future tool in setting up new tolerances for different mix designs.

In the @RISK simulations, many mix designs appeared to contain a dramatic jump in pay deduction from 0 to 10 for binder content. By incorporating pay deductions between 0 and 10, more mixes might fall closer to specifications due to added incentive to have less pay deduction. This same philosophy of added incentive could be used for all the pay factors that jump from 10 to 25.

Another consideration would be to use a cumulative pay deduction rather than the maximum. This will require a greater focus on all of the criteria. In order to keep this rounded focus an establishment of incentives may raise the quality of pavements. For example, noise and pavement smoothness may be a possible pay incentive.

8.9 Statistical Quantification of Material Properties for Fatigue

The regression analysis herein has statistically quantified the material properties relating to fatigue cracking in laboratory mixed and compacted HMA pavements that were sampled from around the State of Michigan. The analysis revealed that asphalt content and air voids have nearly an equal, but opposite impact on the fatigue life of HMA. This work has also shown the impact that the traffic level and aggregate size of HMA can be quantified in terms of dummy variables and does have an impact on the fatigue life of a pavement.

8.10 Statistically Based Pay Factor System for MDOT

This report has demonstrated the possibility of the development of a statistically based pay factor system with regards to fatigue cracking in the State of Michigan. The work herein can serve as a basis for developing and implementing such a system. The confidence bands developed herein provide a way of quantifying the expected performance of an HMA pavement, which negates the need for a cap value with respect to payment bonuses, if the performance parameters indicate that the pavement is of the utmost superior quality. This research will help to quantify an HMA's expected performance based on two parameters that can be easily tested in a short amount of time.

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