Burns Cooley Dennis, Inc.
Geotechnical, Pavements and Materials Consultants

IMPLEMENTATION OF SUPERPAVE MIX DESIGN FOR AIRFIELD PAVEMENTS

Volume I : Research Results

for

AAPTP PROJECT 04-03

Submitted to

Airfield Asphalt Pavement Technology Program

By

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Draft Final Report

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ABSTRACT

Airfield Asphalt Pavement Technology Program Project 04-03, Implementation of Superpave Mix Design for Airfield Pavements, was conducted to develop and recommend a method of designing hot mix asphalt (HMA) for airfield pavements utilizing the Superpave gyratory compactor (SGC). The research approach entailed three phases of work. During the first phase, background information was obtained through reviewing specifications, reports and literature and discussions with airfield pavement experts. This information was then utilized to develop an approach for developing a mix design method for airfield HMA using the Superpave gyratory compactor. The second phase of work entailed carrying out the research approach developed at the conclusion of the first phase or work. This second phase consisted of both field and laboratory work. Ten airfields from across the US were visited in order to evaluate performance. Materials from the different airfield pavements were obtained and included during a laboratory study. The final phase of work involved analyzing all data and preparing the final report.

Based upon the first phase of research, there are a lot of similarities between the Marshall and Superpave methods of designing HMA. Both methods have similar requirements for materials utilized within the mix and both methods rely heavily on selecting appropriate volumetric properties to define the optimum asphalt binder content. The biggest difference between the two methods is the design compactive effort. The Marshall mix design method utilizes the impact loading of the Marshall hammer while the Superpave method utilizes the kneading action of the SGC. Therefore, the bulk of the research was conducted to develop the appropriate design compactive effort using the SGC.

At the conclusion of the study, a mix design method was recommended for airfield HMA that utilizes the SGC. The method entails primarily four steps. The first step in the mix design method is to select appropriate materials. Materials needing selection include coarse aggregates, fine aggregates, asphalt binder, anti-stripping additives and mineral fillers. Recommended values for a guide specification were provided. The next step in the mix design procedure is to develop a design gradation utilizing the selected aggregates. This process involves blending the selected stockpiles to meet the recommended gradation bands and selecting a blend that will meet all requirements. The third step entails selecting optimum asphalt binder content. Optimum asphalt binder content is defined as the asphalt binder content that results in 4.0 percent air voids and meets all other volumetric properties. The design compactive effort is applied utilizing the SGC. The final step in the mix design method is to evaluate the designed mixture for moisture susceptibility.

SUMMARY OF FINDINGS

Airfield Asphalt Pavement Technology Program Project 04-03, Implementation of Superpave Mix Design for Airfield Pavements, was conducted to develop and recommend a method of designing hot mix asphalt (HMA) for airfield pavements utilizing the Superpave gyratory compactor (SGC). The research approach entailed three phases of work. During the first phase, background information was obtained through reviewing specification, reports and literature, and discussions with airfield pavement experts. This information was then used to develop an approach for developing a mix design method for airfield HMA using the SGC. The second phase of work entailed carrying out the research approach developed at the conclusion of the first phase of work. This second phase consisted of both field and laboratory work. Ten airfields from across the US were visited in order to evaluate performance. Materials from the different airfield pavements were obtained and included during a laboratory study. The final phase of work involved analyzing all data and preparing the final report.

During the first phase of research, the researchers contacted a number of individuals experienced in construction of flexible airfield pavements. These conversations were conducted to identify concerns about utilizing both the Marshall and Superpave mix design methods for designing airfield HMA. Additionally, the researchers wanted to identify the most common distress types encountered in airfield flexible pavements. Also conducted during the first phase of work was a critical comparison between the Marshall and Superpave mix design method.

Based upon the critical comparison between the two mix design methods, the two have many similarities. Both include four primary steps in the mix design process, which include: selection of materials, selecting the design gradation, selecting optimum asphalt binder content, and evaluation of moisture susceptibility. Both methods have criteria for the selection of materials that are similar in that the desired quality characteristics are similar. The test methods are not always the same; however, the desired characteristics are similar. There are more differences in how the aggregates are blended. The Superpave gradation requirements allow for the most gradation options (maximum aggregate sizes) and the most production shapes for a given maximum aggregate size. The Marshall gradation requirements tend to be more restrictive because of the use of gradation bands. The biggest difference in the two mix design methods is the method of applying the laboratory design compactive effort. The Marshall method utilizes the impact energy of the Marshall hammer and the Superpave method utilizes the kneading action of the SGC. The design compactive effort using the Marshall hammer is the number of impacts imparted onto the confined HMA sample, while the SGC kneads the confined HMA sample using a specified number of gyrations. With respect to moisture susceptibility, both methods utilize tensile strength ratios. Based upon the critical comparison, the primary issues that had to be addressed as part of this study were design laboratory effort, appropriate volumetric criteria for selection of optimum asphalt binder content, appropriate gradation requirements for airfields, and appropriate test methods and criteria for materials selection. Based upon the discussions with the airfield pavement experts, the major distress types that should be considered are related to environmental effects.

At the conclusion of the first phase of work, are experimental program was developed in order to address the issues described above. The experimental plan involved identifying and visiting ten airfield pavements from across the US. Included within the ten airfields were general aviation airfields, large commercial airfields, and military airfields. At each airfield, a performance evaluation of the selected pavement was conducted. Materials from the original sources that were used to fabricate the HMA were also obtained and utilized in a laboratory study. These original materials were used in two primary laboratory evaluations. The first evaluation was conducted to select an appropriate number of gyrations to design HMA. The second laboratory evaluation was conducted to evaluate appropriate gradation requirements. Based upon the results of the field and laboratory work, a mix design method was developed for airfield HMA that utilize the SGC.

The basic structure of the mix design method that was developed was identical to that described above as there are four primary steps which include: 1) selection of materials; 2) selection of design gradation; 3) selection of optimum asphalt binder content; and 4) evaluation of moisture susceptibility. Of particular interest, three different design compactive efforts were recommended with the SGC. The appropriate design compactive effort is selected based upon the tire pressures expected for the aircraft that will utilize the airfield pavement. Recommended material requirements, specifically aggregates, are also based upon the expected tire pressures. Gradation recommendations within the mix design method are a compromise between the two historical airfield mix design methods, Item P-401 and UFGS 32 12 15.

CHAPTER 1
Introduction and Research Approach

INTRODUCTION

Approximately ninety percent of America's paved runways are paved with hot mix asphalt (HMA). However, only a small percentage of the total HMA placed in the United States is used for airfields. Historically, HMA for airfield pavements has been designed using the Marshall mix design method. Conversely, the vast majority of non-airfield HMA pavements placed during the last 5 to 7 years have been designed using the Superpave mix design system. The percentage of HMA that is being designed using the Superpave mix design system is increasing every year. Therefore, mix design experience is being gained by HMA contractors, commercial labs, and industry personnel in the area of Superpave. Since the Marshall mix design procedure is becoming the exception to the rule, industry personnel are becoming increasingly unfamiliar with the Marshall mix design method. As such, the airfield industry needs to implement the Superpave mix design system in airfield pavements in order to benefit from the industry's experience with Superpave.

Background

Airfield Hot Mix Asphalt Design Specifications

Three specifications are typically used to design airfield HMA pavements. These include Item P-401 documented in the Federal Aviation Administration (FAA) Advisory Circular (AC) 150/5370-10B; the Department of Defense (DoD) Unified Facilities Guide Specification (UFGS)-32 12 15; and Engineering Brief (EB) 59A. Item P-401 and

UFGS-32 12 15 are Marshall mix design specifications. Item P-401 is utilized on most civilian airfields. The UFGS-32 12 15 is utilized to design HMA for military airfields.

EB-59A is the current Superpave mix design system allowed for airfield pavements. Using EB-59A requires approval at the FAA regional office level because it is considered a modification of standards. EB-59A was released in May 2006 and its predecessor EB-59 was released in December 2001. The relatively recent releases of the specifications and the extra approvals required in using these specifications have resulted in relatively few airfields utilizing either EB-59 or EB-59A specifications.

A Brief History of the Marshall Mix Design System

The basic concepts of the Marshall mix design method were initially developed by Bruce Marshall with the Mississippi State Highway Department around 1939. The Marshall mix design procedure evolved over the years from the period of World War II to the late 1950s. The motivation for developing the mix design procedure was a need for a method to proportion aggregates and asphalt binder that could sustain increasing wheel loads and tire pressures produced by military aircraft.

In order to develop the design procedure, the Army Corp of Engineers Waterways Experiment Station reviewed the Marshall mix design method along with several others and ultimately investigated the Marshall mix design method versus the more commonly used Hubbard-Field test method (1). The laboratory investigation of the two methods revealed that the Marshall mix design method compared favorably with the results of the Hubbard-Field method in measurement of stability, sensitivity to asphalt, and reproduction of test results. The Hubbard-field apparatus was large, heavy, and not easily portable. The Marshall method was eventually recommended for adoption by the Corps of Engineers because: 1) it was designed to stress the entire HMA sample rather than just a portion of it; 2) it facilitated rapid testing with minimal effort; 3) it was compact, light, and portable; and 4) it produced densities reasonably close to field densities. The Marshall stability test method could also be performed with minor adjustments to the existing California Bearing Ratio (CBR) equipment that was being used during pavement structural designs (1).

Sample preparation in the original Marshall mix design method was different than it is today. The original Marshall compaction procedure was 25 blows of the standard Proctor hammer followed by the application of a 5000-lb static load for two minutes. This static load was used to level the sample. Initially, during the Marshall Stability test, stability was the sole characteristic measured. The flow measurement was later added because of the desire to add a measurement of strain to the Marshall Stability test, and measured in units of 1/32 inches rather than the current 0.1 inches (2).

The Marshall Procedure continued to evolve during the mid 1940's. Some initial test sections indicated that the original Marshall mix design method selected an asphalt content that was too high. After review, a new compactive effort was selected of 55 blows on each side of the specimen followed by the 5000-lb static load. Another study concluded that the static load could be removed if the area of the hammer face were increased from the 1.95 sq. in. of the modified AASHO hammer used for Proctors to 3.875 sq. in. and the hammer weight increased from 10-lb to 12.5-lb. The density achieved by the 55 blow method (including static loading) was approximately equal to the density achieved by 50 blows per face of a 12.5 lb hammer.

A conference was held in Vicksburg, MS regarding the Marshall procedure in 1947. The participants recommended that a 10 pound hammer be used instead of the 12.5 lb hammer. The research using the 10 pound hammer at 50 blows subsequently returned a conclusion that stated the target field density was 98 percent of that determined by the 50-blow compactive effort. Thus, the standard compaction procedure became 50 blows per face with a 10 lb hammer (2).

As aircraft loadings and tire pressures increased, the Army Corp of Engineers developed a modification to the 50 blow compactive effort. For pavements expected to receive tire pressures from 100 psi to 250 psi, the compactive effort was raised from 50 to 75 blows with a Marshall hammer (3).

Performance of Airfield and Highway Flexible Pavements

Airfield and highway flexible pavements have many similarities, but also have many differences. Both airfield and highway flexible pavements are designed to transfer loads to the underlying subgrade in a manner that does not overstress the subgrade or create large tensile stresses at the bottom of the asphalt layer. Also, highways and airfields typically utilize the highest quality materials near the pavement surface while material quality generally decreases with depth. The primary differences between highways and airfields, though, are the types of loads and number of loads that are experienced during the design life.

Airfield pavements tend to experience far fewer load repetitions over their design lives than do highway pavements. Table 1 presents the total number of aircraft operations at ten selected airports during 2007. This data illustrates that during an average day many of the busiest airports in the U.S. had less than 2,500 operations. These airports also have multiple runways and taxiways to distribute the traffic. In fact, there are many pavement areas within the nation's busiest airports that may not have a single load applied during the pavement's entire life. Conversely, for many interstate highways, the average daily traffic can be above 40,000 with heavy truck traffic being in the tens of thousands per day.

Table 1: Total Aircraft Operations at Selected U.S. Carrier Airports in 2007

	Aircraft Operations	
Airport	Annual	Average Day
Chicago O'Hare International	935,356	2,563
Los Angeles International	672,245	1,842
Hartsfield-Atlanta International	989,305	2,710
John F. Kennedy International	453,258	1,242
San Francisco International	371,291	1,017
Denver International	611,971	1,677
LaGuardia	401,410	1,099
Miami International	386,734	1,060
Washington National	279,939	767
Boston Logan International	410,295	1,124

The other primary difference between airfield and highway pavements is the types of loadings. For highways, heavy truck traffic is the primary characteristic used to specify pavement structure and materials. This spectrum of traffic has been quantified by the use of equivalent single axle loads (ESALs) as the controlling factor for both pavement design and selection of HMA materials. For airfields, the pavement is generally designed and specified based upon a design aircraft(s). Depending upon whether the airport is a small general aviation airport or large commercial airport, the design aircraft can be as small as a Cessna Skyhawk having a gross weight of approximately 3,000 lbs or an Airbus A380 having a maximum take off weight of 1,300,000 lbs. Another factor related to loads is tire pressure. Small aircraft can have tire pressures similar to automobiles, while some military fighter jets can have tire pressures over 300 psi.

Another difference between airfield and highway pavements is the traffic patterns. For highways, the traffic is generally channelized and falls within narrow wheelpaths along the roadway. Traffic patterns on airfields can vary from channelized – moving (taxiways) to channelized-stacked (runway-taxiway ends) to evenly distributed and random (aprons) to occasional (runway edges) to almost never (shoulders and blast pads).

The loading types and repetitions on airfield pavements require some areas of the pavement structure to be able to withstand the sudden impact of landing aircraft. Generally, most airfield pavements do not have load associated distresses unless the pavement structure was under-designed or there were construction related problems. As a matter of fact, a survey of fifteen airfield asphalt industry professionals conducted during this project indicated that the main distress experienced in airfield pavements is not structural, like is typically seen in highway pavements (e.g., rutting, fatigue cracking), but rather environmental. Runways, taxiways and aprons are more prone to raveling and block cracking, which are caused by environmental conditions such as oxidation and weathering. In colder climates, thermal cracking is also a serious problem with airfield HMA pavements.

Highway pavements experience a rejuvenating effect of the vehicular traffic that helps to mitigate cracking. Airfields do not experience much of the rejuvenating effect or extra compaction that helps combat cracking and oxidation. Instead, airfield pavements may see only a few loadings a day, especially when considering wheel wander and the various gear configurations that are present on the different types of aircraft loading at an airport.

The priority for maintenance in airfield pavements is even more critical for safety concerns than it is for the highway. The tolerance for severity in pavement distresses for aircraft is much smaller than the tolerance by highway vehicles. If a vehicle hits a pothole in an asphalt pavement, it typically causes rider discomfort and, in extreme conditions, minor vehicular damage. If a heavily loaded plane hits a pothole, it could break a gear resulting in very expensive equipment damage and potentially cause injuries or fatalities depending on the speed of the plane. Another concern for distresses in airfield pavements is foreign object debris (FOD), which causes foreign object damage. Loose aggregate on an airfield pavement can cause damage to propellers and jet engines. This FOD damage can be very expensive to repair, but could also lead to passenger casualty in a worst case scenario.

The Evolution of the Superpave Mix Design System

Beginning in October 1987, the Strategic Highway Research Program (SHRP) began research on developing a new system for specifying asphalt materials (4). This $150 million project ($50 million of which was spent on asphalt) funded by congress was originally tasked with developing an asphalt binder specification, mixture design and analysis system, and a computer software system with increasingly complex tests and specifications as traffic levels increased. Currently, the asphalt binder specification and mixture design system are used in common practice. The developed mix design system was called Superpave which is an acronym for Superior Performing Asphalt Pavements.

The signature piece of equipment within the Superpave mix design system is the Superpave gyratory compactor (SGC), but the mix design system is more than just the compactor. The Superpave mix design system provides specifications for choosing asphalt binder and aggregates as well as volumetric requirements for HMA compacted in the SGC. It is a performance based design system that measures physical properties of the binder and aggregate that are directly related to field performance. The performance or "proof" test of the HMA mixture under the Superpave design system is still under development. As of 2005, the Superpave mix design system was being used by forty six of the states in the U.S.

Problem Statement

The Marshall mix design procedure was originally developed in the 1940's for airfield pavements. While this mix design procedure has performed well for airfield and highway pavements for over 50 years there is a need to adopt the new Superpave mix design procedure for airfield pavements.

An issue with the Marshall mix design method is that the compaction process does not orient the aggregate in the laboratory compacted sample the same way that it is oriented in the field. This results in a problem when attempting to conduct performance tests since the particle orientation will affect the measured results. The gyratory compactor produces aggregate orientation that is more similar to what is seen in the field.

Another issue with the Marshall method of mix design is the higher variability of test results. The proficiency sample data from the AASHTO Materials and Reference Laboratories (AMRL) over the past three years shows that the SGC provides sample air void contents with lower overall variability (standard deviation = 0.995) than samples compacted using the Marshall pedestal and hammer (standard deviation = 1.059). This lower variability should result in a more consistent design and should allow QC testing to better compare with QA testing (5).

A third, and likely most important, issue with the Marshall mix design process is that most state DOTs have begun using the Superpave mix design procedures. Since most asphalt work is done by the DOTs, it is becoming more difficult to find contractors and commercial laboratories having the proper accreditations with the Marshall mix design method. This problem will become much worse in the future.

Given the issues with the Marshall mix design procedure, it is desirable to adopt the Superpave mix design system for airfield pavements. Superpave was developed for highway pavements, not for airfield pavements, so some modifications to the process are likely needed prior to adopting for airfields. The Superpave mix design process should not be adopted without some research to identify the specific procedures to be used for airfields. The compactive effort in the mix design procedure should be a function of traffic level, traffic loads, speed of traffic, and/or tire pressures, etc.

Objective

The objective of this study was to adapt Superpave gyratory compactor procedures to design airfield HMA mixes with properties comparable with P-401.

Scope

In order to accomplish the project objective, the researchers carried out a number of tasks. Initially, the mix design specifications typically used to construct HMA layers were critically reviewed. Comparisons between the Marshall and Superpave mix design systems were made with emphasis on identifying similarities and differences between the two systems. Next, the researchers contacted a number of experts in the area of HMA construction on airfields to discuss concerns with both the Marshall and Superpave systems. During these discussions, the researchers also identified ten airfields located throughout the US for execution of a field and laboratory study. For each of the identified airfields, the researchers visited and conducted a pavement performance evaluation. Additionally, cores were obtained in order to establish the in-place properties of the HMA. Materials as close to the original materials as possible were obtained and included within the laboratory study. The in-place mixes were replicated using the obtained materials. Specimens were compacted with both the Marshall hammer and Superpave gyratory compactor using various compactive efforts. Specimens were also prepared for performance testing. The performance test selected for this project was the confined repeated load permanent deformation test (or commonly called the Flow Number Test). At the conclusion of the study, the data was analyzed in order to adopt a Superpave mix design system for airfield pavements.

Report Format

This report is comprised of three separate volumes. Volume I provides results of all research along with conclusions and recommendations for implementing Superpave for airfields. This volume also includes an implementation plan that outlines how the results of the research may be quickly implemented by FAA and DoD. Volume II provides a guide specification for designing HMA for airfields using the Superpave concepts. Volume III is a stand-alone guidance document on the selection of appropriate HMA mixtures for airfield applications when using the Superpave mix design procedures. This guidance also provides the recommended Superpave mix design method for airfields along with discussions on construction, performance, quality control and quality assurance. The purpose of Volume III is to provide practical guidance for engineers practicing airfield HMA pavement construction.

CHAPTER 2:
Research Approach

INTRODUCTION

As stated previously, the objective of this project was to adapt the Superpave mix design procedures for designing HMA to be placed on airfield pavements. In order to accomplish this objective, three phases of work were required. During the first phase of research, background information was obtained through reviewing specifications, reports and literature and discussions with airfield pavement experts. This information was then utilized to develop an approach for adapting Superpave for airfields.

The second phase of work entailed carrying out the research approach developed at the conclusion of the first phase of work. This second phase consisted of both field and laboratory tasks. Ten airfields from all over the US were visited in order to evaluate performance. Materials from the different airfield pavements were also obtained and used in the laboratory.

The final phase of research was to prepare the final reports. The following sections provide details on the overall research approach.

Phase 1: Program Review

The objective of the first phase of research was to develop an approach for adapting the Superpave mix design procedures for use on airfields. In order to develop the approach, the researchers conducted a literature review, interviewed airfield pavement experts, and reviewed various specifications.

The literature was reviewed in order to identify any evaluations between the Marshall and Superpave mix design methods that had already been accomplished. Of particular interest were studies that compared the compactive efforts within the two mix design methods.

Background information on the current issues/concerns with HMA pavements was obtained by interviewing various airfield pavement experts. The FAA is divided into nine regions. Roughly, the regions represent different climatic zones. The researchers contacted a number of FAA Pavement Engineers to discuss HMA pavement performance in the various regions of the country. The US Corps of Engineers, Air Force Civil Engineer Support Agency and Naval Facilities Engineering Command also have engineers familiar with airfield pavement performance. Discussions with these various engineers were conducted to identify:

1) Concerns about existing HMA mix design specifications.
2) Pavements constructed using EB-59 or EB-59A.
3) Problematic areas for localized problems, e.g. taxiways, aprons, shoulders, blast pads, etc.
4) Candidate airfields to conduct field work during the second phase of research
5) Airports that have modified specifications to combat specific distresses.

The final task conducted to review the state-of-practice was to critically compare the various specifications used to design HMA for airfield pavements. As stated previously, there are three primary specifications used to design HMA for airfield pavements. Item P-401, UFGS-32 12 15, and EB-59A. A critical review of these specifications was conducted in order to identify similarities and differences between the requirements within the specifications. Identifying the similarities and differences assisted in allowing the researchers to adapt the Superpave method to airfield HMA.

At the conclusion of the Phase I work, the researchers developed field and laboratory investigations plans. A three pronged approach was developed for the field and laboratory studies:

- Pavements were identified of varying maximum aggregate size, aggregate types, gradations, environmental conditions and loading, a condition assessment was performed and core samples were collected from the airfield pavements to verify the as-constructed mixtures.
- Materials that were as close as possible to the original materials were collected and the mixtures were replicated in the first field phase. Comparisons were performed with Superpave and Marshall compaction efforts. Performance tests were also performed to determine the rutting susceptibility of the mixtures.
- A draft specification was developed and on-going airport paving projects were sampled to evaluate the specification and collect data for the development of quality control and quality assurance specifications.

Pavements were identified for various types of loading. Two projects were identified as having rutting problems. Mix designs and quality control and quality assurance data for all pavements were obtained. The mix designs were used to select airfield pavements representing a range of materials and gradations. Original materials were identified and sampled for the candidate sites so that the mix could be replicated in the laboratory. Candidate sites were visited to perform a condition assessment and to obtain cores in order to verify the as-constructed mixture. Where possible, surface and underlying layers were obtained.   The preliminary experimental matrix is shown in Table 2. The final matrix is shown in a later section.

Table 2: Experimental Matrix

Loading	Number of Sites
Gross weight < 60,000 lbs or tire pressure < 100 psi	3
Gross weight > 60,000 lbs or tire pressure > 100 psi	4
Tire pressure > 200 psi	3

After cores were taken from each airfield visited, they were wrapped in plastic wrap to maintain their in-place moisture content and shipped back to the lab for density determination, indirect tensile strength, asphalt content and gradation. A visual moisture damage assessment was performed on the samples after the indirect tensile strength was determined. The intent was to use the in-place asphalt content and gradation to set target values for replicating the mix, one deviation from this intention is explained in a later section.

The ten mixes were replicated in the laboratory using original materials. Each mix was compacted at two Marshall compaction levels: 50- and 75-blows, and three gyratory N_{des} levels 50, 75 and either 35 or 100 gyrations. Either 35 or 100 gyrations were chosen dependent upon the bulk specific gravity (G_{mb}) of the 75 blow Marshall samples. The purpose of varying the gyratory levels was to create gyratory samples that had G_{mb}'s that bracketed the G_{mb} achieved by the 75 blow Marshall samples. If this could not be accomplished by the 50, 75 and 100 gyration level grouping; 35, 50 and 75 gyrations were used.

Based on this data developed by the variety of compaction levels and methods, the gyratory compaction effort to match each of the Marshall compactive efforts could be determined. In addition to the volumetric comparison of the three compactive efforts described above, the research team determined optimum asphalt binder content for each mix using N_{des} values of 50, 75 and 35 or 100. Depending upon the compactive effort of the original mix (50 or 75 blows), this provided a wide range of mix properties for each given material. These four mixes (3 N_{des} levels and original Marshall) were used for performance testing described in the following paragraphs.

There are two distress types that are directly related to HMA materials: permanent deformation and durability. Fatigue cracking is affected by HMA materials but is more related to pavement structure than HMA materials. The distress type that is easier to characterize in the laboratory is permanent deformation. Many tests have been used to characterize durability. However, these tests require a sophisticated model to relate results to field durability performance. If not calibrated for local conditions, these models are generally not accurate. Because of the lack of an adequate laboratory durability test, the research team adopted an experimental plan that utilized laboratory permanent deformation testing. Because the primary distress on airfield pavements are durability related, the researchers took the approach of increasing the binder content as high as possible without developing the potential for rutting. With this approach, sufficient binder would be added to the mixes without sacrificing rut resistance. The higher binder content would also provide the best durability. Also, the appropriate 50- and 75-blow properties were used to ensure that the recommendations are not excessively different from what is presently being done.

One of the Superpave simple performance tests recommended for permanent deformation is the confined repeated load permanent deformation test. This test was selected for this project to evaluate permanent deformation. Ahlrich (6) previously used this test with a 200 psi deviator stress utilizing a 40 psi confining stress to access the effect of aggregate properties on the rutting performance of heavy duty airfield HMA mixes. Samples were prepared at 7 percent air voids to simulate the approximate initial in-place density. Repeated load permanent deformation tests were conducted at three pressures: 100, 200 and 350 psi. The samples were tested with a 40 psi confinement pressure at the effective pavement temperature for rutting at the site. The samples were tested at the optimum asphalt content determined at two to three gyration levels and the original design asphalt content as shown in Table 3. Three replicates were tested at each pressure for a total of 276 tests. Three parameters can be assessed from the repeated load permanent deformation test: the secondary creep slope, total accumulated permanent strain, and number of cycles until tertiary flow occurs (flow number).

Table 3: Repeated Load Creep Tests

Loading	Optimum Asphalt Content at:				Number of Mixes
	50 gyrations	75 gyrations	100 gyration	Original	
Gross weight < 60000 lbs or tire pressure < 100 psi	100 psi 200 psi 350 psi	100 psi 200 psi 350 psi		100 psi	3
Gross weight > 60000 lbs or tire pressure > 100 psi	100 psi 200 psi 350 psi	100 psi 200 psi 350 psi	100 psi 200 psi 350 psi	200 psi	5
Tire pressure > 200 psi		100 psi 200 psi 350 psi	100 psi 200 psi 350 psi	350 psi	2

The results of these three parameters at the original JMF asphalt content were compared to the field performance to help establish performance criteria. Based upon the research team's experiences, the optimum asphalt content derived from 100 design gyrations is too low for a general aviation field (< 60000 lbs, < 100 psi); therefore, 100 gyration samples were not tested for the mixtures used on general aviation fields. Similarly, the 50 gyration mix would not provide acceptable performance where tire pressures are in excess of 200 psi. Therefore, 50 gyration mixes were not tested for pavements with tire pressures in excess of 200 psi. Thus, using the results from the repeated load permanent deformation test, it was possible to determine the laboratory compactive effort that provided adequate rut resistance to various tire pressures. By selecting the maximum binder content that provides acceptable rutting performance, durability performance would be maximized.

As stated previously, durability has been a difficult parameter to assess in the laboratory. Although techniques such the fracture energy ratio tests developed by Roque and Drakos (7) and Kim an Wen (8) could be used, it is expected that durability is highly affected by the selected binder grade and field construction. Inadequate density or segregation will likely lead to poor durability. These are not mix design problems. It is felt that durability is best addressed by maximizing the binder content while maintaining stability (rut resistance) and by binder grade selection. Comparisons with the 50- and 75-blow Marshall asphalt contents also provided a relative index of durability. Indirect tensile tests on cores from the field sites indicated whether or not moisture damage is a common problem on airfields. It is expected that the finer gradations and higher in-place density requirements on airfields may help prevent moisture damage. Also, the lower volume of traffic may reduce the possibility of moisture damage. Further, moisture damage, though affected by design binder content (film-thickness) is generally mitigated with liquid anti-stripping agents or the addition of lime, not mix design changes.

Even though the research team did not recommend any formalized durability testing, the research team did have concerns with coarser gradations, specifically gradations near the Superpave lower control point, for airfield pavements. Research has shown that these coarser gradations tend to be more permeable than fine-graded mixes. Therefore, for a number of mixes additional gradations were blended and designed. The intent of these additional gradations was to cover the wide range of potential gradations from the lower control point of Superpave to the fine side of UFGS- 32 12 15. For each of these additional gradations, mixtures were prepared for permeability testing. Cooley et al (9) presented a method for conducting permeability testing in the laboratory that correlated well with in-place permeability measurements. Also, Cooley et al (10) have recommended critical permeability values in which to compare results from this testing. High levels of permeability will increase the potential for oxidative aging and moisture damage. Kumar and Goetz (11) have shown a direct relationship between permeability and asphalt age hardening. Results of this permeability testing assisted the researchers in recommending gradation requirements for HMA to be used on airfields.

Once the laboratory data was compiled and the tentative framework developed, additional field testing was conducted. Quality assurance testing was conducted using the proposed specification as a shadow specification. Marshall samples were compacted for comparison. In addition to verifying the proposed volumetric properties and compaction levels, this data was used to adjust volumetric variability within the current P401 quality control/quality assurance specifications.

Phase II – Conduct Investigations

The field investigation was conducted in two phases. The first phase, described previously required the research team to visit ten airfields, visually assess performance and collect design and production data. This required careful coordination with airfield officials to obtain access to various airfield taxiways and runways. Cores were shipped to Burns Cooley Dennis, Inc. (BCD) for testing.

The second part of the field study was to sample two demonstration projects. This work provided a check of the mix design system, Quality control and quality assurance data and served as a step towards implementation by demonstrating the proposed design system to aviation officials.

The core testing was conducted by BCD. All aggregate processing, replication, and testing of Marshall mixes and gyratory mix designs was conducted by BCD. NCAT conducted all of the permanent deformation tests. This division of labor prevented confounding variability issues caused by performing tests on different sets of equipment.

Phase III - Reports

The final phase of work entailed compiling the draft final report according to the guidelines established by the AAPTP. Additionally, the researchers met with the project panel to present the research results. Following the review of the draft final report and the project panel meeting, the final report was submitted.

CHAPTER 3
Review of Existing Airfield Specifications

The primary hot mix asphalt (HMA) mix design specifications utilized for airfield pavements include Item P-401, documented in the FAA Advisory Circular (AC) 150/5370-10B, and the DoD's UFGS-32 12 15. Item P-401 is utilized on most civilian airfields. However, the HMA used on many general aviation airfields is designed using local specifications because of the relatively light aircraft and the relatively few operations. The UFGS-32 12 15 is utilized to design HMA for military airfields. EB59A does include the Superpave gyratory compactor in designing HMA for airfield pavements; however, it follows the general Superpave requirements for highways with the material requirements similar to the existing P-401 requirements.

Hot mix asphalt for highway pavements is most commonly designed in accordance with the Superpave mix design method as outlined in AASHTO M323, "Standard Specification for Superpave Volumetric Mix Design." Practically every State Department of Transportation has adopted the Superpave mix design method for designing HMA for highways. From a production standpoint, this means that most HMA produced in the U.S. is designed using the Superpave mix design method.

The following sections critically review the historical methods of designing HMA for airfields and the Superpave mix design method for highways. The review entailed evaluating the similarities and differences between the methods. EB59A was intentionally left out of this review because it is a hybrid containing parts of the historical methods and of the Superpave for highways methods.

General

All three of the HMA mix design specifications mentioned above have a similar goal: develop the right volumetric proportion of aggregates, asphalt binder, and air voids. By designing an HMA with the right volumetric proportions, the pavement structure the HMA is placed on should perform with respect to stability and durability. Each method includes basically the same four steps: 1) select acceptable materials (aggregates and asphalt binder); 2) blend the selected materials to meet specifications; 3) select an appropriate optimum asphalt binder content; and 4) evaluate the designed mixture for moisture susceptibility.

Selection of Materials

Materials used in the design of dense-graded HMA include coarse aggregates, fine aggregate, asphalt binder, and other materials that may be required to meet the mix design specifications. In some instances, mineral fillers are needed if local aggregates do not contain a sufficient amount of material passing the 0.075mm (No. 200) sieve. When local materials have a high potential for moisture susceptibility, anti-stripping additives are also commonly used within the HMA. The following sections discuss the material requirements for the different mix design methods.

Coarse Aggregates

All three mix design specifications provide recommendations for coarse aggregate angularity and shape. For coarse aggregate angularity, all three methods specify a minimum percentage of coarse aggregates with fractured faces; however, the requirements are slightly different. Item P-401 does not reference a specific standard for performing the fractured face count test; rather, it specifies that a fractured face "…shall be equal to at least 75 percent of the smallest mid-sectional area of the piece." This same terminology is included within the Corps of Engineers (COE) test method CRD-C 171-95, "Standard Test Method of Determining Percentage of Crushed Particles in Aggregate," which is referenced in UFGS-32 12 15. The primary difference between Item P-401 and UFGS-32 12 15 is that Item P-401 has specification requirements for both one and two or more fractured faces, while UFGS-32 12 15 only specifies a minimum percentage of coarse aggregates with two or more fractured faces.

The standard specification for Superpave (AASHTO M323) references ASTM D5821, "Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate," as the method of measuring fractured faces. Within this standard, a face is considered fractured "… only if it has a projected area at least as large as one quarter of the maximum projected area (maximum cross-sectional area) of the particle and the face has sharp or slightly blunt edges." Similar to Item P-401, the Superpave method has requirements for both one and two or more fractured faces.

Both Item P-401 and Superpave have varying fractured face requirements based upon anticipated pavement loadings. Item P-401 differentiates based upon an aircraft gross weight of 60,000 lbs. For pavements designed for aircraft larger than 60,000 lbs, the minimum requirement for one fractured face is 85 percent with a minimum of 70 percent two or more fractured faces. For pavements designed for aircraft less than 60,000 lbs, the minimum requirements are 65 percent one fractured face and 50 percent two or more. Similarly, the Superpave requirements for percent fractured faces are based upon anticipated loadings (Table 4), except the loadings are based upon equivalent single axle loads (ESALs). The UFGS-32 12 15 specification has a minimum requirement of 75 percent coarse aggregates with two or more fractured faces for all HMA.

Table 4: Superpave Aggregate Requirements

Design ESALs[a] (Million)	Fractured Faces, Coarse Aggregate[c], Percent Minimum — Depth from Surface ≤100mm	Fractured Faces, Coarse Aggregate[c], Percent Minimum — Depth from Surface >100mm	Uncompacted Void Content Of Fine Aggregate, Percent Minimum — Depth from Surface ≤100mm	Uncompacted Void Content Of Fine Aggregate, Percent Minimum — Depth from Surface >100mm	Sand Equivalent, Percent Minimum	Flat and Elongated[c], Percent Maximum
<0.3	55/-	-/-	-	-	40	-
0.3 to <3	75/-	50/-	40	40	40	10
3 to <10	85/80[b]	60/-	45	40	45	10
10 to <30	95/90	80/75	45	40	45	10
≥30	100/100	100/100	45	45	50	10

[a] The anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years.
[b] 85/80 denotes that 85 percent of the coarse aggregate has one fractured face and 80 percent has two or more fractured faces.
[c] This criterion does not apply to 4.75-mm nominal maximum size mixtures

All three specifications also have requirements for particle shape using ASTM D4791, "Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate." This test compares the dimensions of aggregate particles to evaluate particle shape. Item P-401 requires three measures of particle shape: flat particles, elongated particles, and flat and elongated particles. To conduct this test, aggregate particles are measured with a proportional caliper using the specified ratio. For Item P-401, the specified ratio is 5:1 though a ratio of 3:1 can be specified by the Engineer. To evaluate flat particles, the proportional caliper is used to compare the particle's thickness and width. Width is defined as the maximum dimension perpendicular to the aggregate's length; where, length is defined as the maximum dimension of the particle. Thickness is defined as the maximum dimension perpendicular to the length and width. Elongated particles are defined as those having a ratio of length to width greater than the specified value. For evaluating flat and elongated particles, the length of each particle is compared to its thickness. As can be seen, each of the measures, flat, elongated, and flat and elongated, provide a different evaluation of particle shape. Within Item P-401, a maximum percentage of flat, elongated, or flat and elongated particles is 8 percent.

The ratio by which aggregates are evaluated in the Superpave specification is identical to Item P-401, 5:1. However, instead of specifying requirements for all three measures of particle shape, the requirements for Superpave are for only flat and elongated particles. One other caveat is that flat and elongated particles are evaluated using the maximum and minimum dimensions, not necessarily the length and thickness. As shown in Table 4, a maximum of 10 percent flat and elongated particles are allowed in coarse aggregates.

The UFGS-32 12 15 method of comparing dimensions is similar to requirements in the Superpave method in that only flat and elongated is evaluated using the maximum and minimum dimensions of the particle. The only difference is that the specified ratio is 3:1 instead of 5:1. Because of the different ratio specified, the maximum percentage of flat and elongated particles is 20 percent within UFGS-32 12 15.

Both Item P-401 and UFGS-32 12 15 contain requirements for coarse aggregate toughness, soundness and cleanliness that are not contained within the Superpave method. Aggregate toughness is defined by ASTM C131, “Resistance to Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine,” in both airfield specifications. A criteria of 40 percent loss is specified in both; however, Item P-401 has a note indicating "… aggregates with a higher percentage loss of wear or soundness may be specified… provided a satisfactory service record… has been demonstrated."

Aggregate soundness is measured in accordance with ASTM C88, "Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate." Requirements within Item P-401 are a maximum of 10 percent loss when using sodium sulfate and 13 percent when using magnesium sulfate, while requirements within UFGS-32 12 15 are a maximum of 12 percent when using sodium sulfate and 18 percent when using magnesium sulfate.

The UFGS-32 12 15 specification is the only one of the three that provides requirements for the cleanliness (deleterious materials) of coarse aggregates. This specification requires a maximum of 0.3 percent clay lumps and friable particles when tested in accordance with ASTM C142, "Standard Test Method for Clay Lumps and Friable Particles in Aggregates."

Summary of Comparison between Coarse Aggregates

All three mix design specifications have requirements to ensure the desired coarse aggregate particle angularity and shape. All three methods also utilize similar test methods (Table 5), with only slight deviations. For coarse aggregate angularity, the percentage of fractured faces is used. The primary difference is that the historical airfield mix design specifications utilize a slightly different definition for fractured faces than does the Superpave specifications. The airfield specifications define a fractured face as an area equal to at least 75 percent of the smallest mid-sectional area of the particle. Using the Superpave specified ASTM D5821, a fractured face is at least 25 percent of the maximum projected area. In essence, these two definitions of a fractured face are practically the same because all three specifications minimize the percentage of flat and elongated particles.

Table 5: Coarse Aggregate Requirements Summary

Characteristic	Mix Design Specification		
	P-401	UFGS-32 12 15	Superpave
Angularity	Fractures Faces	Fractures Faces	Fractures Faces
Shape	Flat, Elongated & Flat and Elongated	Flat and Elongated	Flat and Elongated
Toughness	LA Abrasion	LA Abrasion	Individual Agency
Soundness	Sulfate	Sulfate	Individual Agency
Cleanliness	Deleterious Materials	Deleterious Materials	Individual Agency

As stated above, all three specifications control the percentage of flat and elongated particles, resulting in relatively cubicle coarse aggregates. The primary differences are the ratio at which the particles are compared and the characteristics being evaluated (flat, elongated, and flat and elongated).

The primary difference between the three mix design specifications related to coarse aggregate is that Item P-401 and UFGS-32 12 15 have requirements for toughness, soundness and deleterious materials. The Superpave specification does not have explicit requirements for these properties; however, they are recognized as important (12). Within the Superpave mix design system, toughness, soundness and deleterious materials are considered "source" properties and specified values are set by local agencies with knowledge of local materials.

Based upon this discussion, requirements for coarse aggregate within the three mix design specifications are very similar. This is especially true when considering that the source properties (toughness, soundness and cleanliness) are specified by most highway agencies when designing HMA using the Superpave system.

Fine Aggregates

All three mix design specifications contain fine aggregate requirements for angularity and cleanliness, though there are some differences in the methods of specifying these characteristics. Within the two historical airfield specifications, there is a maximum percentage of 15 percent natural sands (noncrushed material) to prevent an excessive amount of rounded fine aggregate particles. In addition to a maximum percentage of natural sands, UFGS-32 12 15 also has a requirement for uncompacted voids determined in accordance with ASTM C 1252, Method A, “Standard Test Method for Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading).” This test method has to be performed on the fine aggregate fraction of the total aggregate blend. A single requirement of 45 percent voids in the uncompacted aggregate is required. A note within UFGS-32 12 15 does, however, allow a minimum uncompacted void content as low as 43 if local experience indicates that a value lower than 45 can be used. The Superpave mix design specification also utilizes the uncompacted voids tests to ensure angular fine aggregates; however, AASHTO T304 Method A, “Uncompacted Void Content of Fine Aggregate,” is specified. The primary difference between UFGS-32 12 15 and Superpave specifications being that the Superpave mix design method has specification requirements based upon traffic (ESALs) (Table 4) instead of a single requirement.

All three mix design specifications also utilize the sand equivalency test (ASTM D2419 or AASHTO T176) to prevent fine aggregates with large percentages of clay-sized particles. The two historical airfield specifications require a minimum sand equivalent value of 45 while the Superpave specification has requirements based upon traffic (Table 4). In addition to the sand equivalency test, Item P-401 also has a requirement that natural sands used in the blend should not have a plasticity index greater than 6 or a liquid limit greater than 25 when tested in accordance with ASTM D4318, “Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

A maximum percentage of deleterious materials (ASTM C142) is specified in UFGS-32 12 15 (0.3 percent maximum) and is considered a source property (agency defined) under the Superpave mix design system. ”

The only other requirement for fine aggregates is contained in Item P-401. Within Item P-401, it states that fine aggregates shall “… be produced by crushing stone, slag or gravel that meets the requirements for wear and soundness specified for coarse aggregate.” The term “wear” in the preceding sentence indicating Los Angeles Abrasion.

Summary of Comparison for Fine Aggregate Requirements

Based upon the three mix design specifications, all provide requirements for fine aggregate angularity and cleanliness (Table 6). The primary differences between the three include the specified maximum allowable percentage of natural, uncrushed sand contained within Item P-401 and UFGS-32 12 15 and the requirements within Item P-401 that the parent aggregates used to create the fine aggregate meet toughness and soundness requirements presented earlier.

Table 6: Fine Aggregate Requirements Summary

Characteristic	Mix Design Specification		
	P-401	UFGS-32 12 15	Superpave
Angularity	Max % Natural Sand	Max % Natural Sand Uncompacted Voids	Uncompacted Voids
Toughness	LA Abrasion (parent aggregate)	—	—
Soundness	Sulfate (parent aggregate)	—	—
Cleanliness	Sand Equivalency Plastic Limit Liquid Limit	Sand Equivalency Deleterious Materials	Sand Equivalency

Mineral Filler

Both Item P-401 and UFGS-32 12 15 have a requirement for mineral fillers; however, there are no requirements for mineral fillers in the Superpave specification. Item P-401 and UFGS-32 12 15 simply state that any mineral filler added to the coarse and fine aggregate must meet ASTM D242, "Standard Specification for Mineral Filler for Bituminous Paving Mixtures."

Asphalt Binder

All three mix design specifications allow the use of Performance Graded (PG) asphalt binders meeting the requirements of AASHTO M320, "Performance Graded Asphalt Binder." Item P-401 and UFGS-32 12 15 also allow viscosity and penetration graded binders. Research project AAPTP 04-02, "PG Binder Selection for Airfield Pavements," is addressing the PG asphalt binder selection issues for airfields.

Blending the Selected Materials

Once materials have been selected, the next step in all three mix design methods is to blend the materials. This step predominately entails blending the selected coarse and fine aggregate stockpiles to meet the respective gradation requirements. Tables 7 through 9 present the gradation requirements for Item P-401, UFGS-32 12 15, and Superpave, respectively.

Table 7: Item P-401 Gradation Requirements

Sieve Size	Percentage by Weight Passing Sieves			
U.S. (mm)	1½"max	1" max	3/4" max	½" max
1-1/2 (37.5)	100	---	---	---
1 (25.0)	86-98	100	---	---
¾ (19.0)	68-93	76-98	100	---
½ (12.5)	57-81	66-86	79-99	100
3/8 (9.5)	49-69	57-77	68-88	79-99
No. 4 (4.75)	34-54	40-60	48-68	58-78
No. 8 (2.36)	22-42	26-46	33-53	39-59
No. 16 (1.18)	13-33	17-37	20-40	26-46
No. 30 (0.600)	8-24	11-27	14-30	19-35
No. 50 (0.300)	6-18	7-19	9-21	12-24
No. 100 (0.150)	4-12	6-16	6-16	7-17
No. 200 (0.075)	3-6	3-6	3-6	3-6

Table 8: UFGS-32 12 15 Gradation Requirements

	Gradation 1	Gradation 2	Gradation 3
Sieve Size, inch (mm)	Percent Passing by Mass	Percent Passing by Mass	Percent Passing by Mass
1 (25.0)	100	---	---
¾ (19.0)	76-96	100	---
½ (12.5)	68-88	76-96	100
3/8 (9.5)	60-82	69-89	76-96
No. 4 (4.75)	45-67	53-73	58-78
No. 8 (2.36)	32-54	38-60	40-60
No. 16 (1.18)	22-44	26-48	28-48
No. 30 (0.6)	15-35	18-38	18-38
No. 50 (0.3)	9-25	11-27	11-27
No. 100 (0.15)	6-18	6-18	6-18
No. 200(0.075)	3-6	3-6	3-6

Table 9: Superpave Aggregate Gradation Control Points

Sieve Size, inch (mm)	Nominal Maximum Aggregate Size – Control Points (Percent Passing)											
	37.5 mm		25.0mm		19.0mm		12.5mm		9.5mm		4.75mm	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
2.0 (50.0)	100	---	---	---	---	---	---	---	---	---	---	---
1.5 (37.5)	90	100	100	---	---	---	---	---	---	---	---	---
1.0 (25.0)	---	90	90	100	100	---	---	---	---	---	---	---
¾ (19.0)	---	---	---	90	90	100	100	---	---	---	---	---
1/8 (12.5)	---	---	---	---	---	90	90	100	100	---	100	---
3/8 (9.5)	---	---	---	---	---	---	---	90	90	100	95	100
No. 4 (4.75)	---	---	---	---	---	---	---	---	---	90	90	100
No. 8 (2.36)	15	41	19	45	23	49	28	58	32	67	---	---
No. 16 (1.18)	---	---	---	---	---	---	---	---	---	---	30	60
No. 200 (0.075)	0	6	1	7	2	8	2	10	2	10	6	12

Item P-401 provides four gradation bands through which the blended aggregates must pass. The gradations are labeled based upon maximum aggregate size. For the purposes of Item P-401, the maximum aggregate size is the sieve one size larger than the first sieve to retain material. Gradation bands within Item P-401, are provided for 1 ½ in., 1 in., ¾ in, and ½ in. maximum aggregate sizes. The UFGS-32 12 15 specification provides three gradation bands that are simply labeled as Gradation 1, Gradation 2, and Gradation 3. Based upon the definition of maximum aggregate size utilized in Item P-401, these three gradations would have a maximum aggregate size of 1 in., ¾ in. and ½ in. A total of six gradation requirements are provided within the Superpave mix design specification. Within the Superpave specification, gradation requirements are based upon control points instead of gradation bands. The control points are less restrictive than full gradation bands as generally the gradation is only limited on four sieve sizes. Another difference is how gradations are defined. Within the Superpave mix design system, gradations are identified based upon nominal maximum aggregate size. Nominal maximum aggregate size (NMAS) is defined as one sieve size larger than the first sieve that retains (total) more than 10 percent aggregate (9). Put another way, the NMAS is one sieve size larger than the first sieve that has less than 90 percent of the blend passing. The maximum aggregate size is simply one sieve size larger than the NMAS.

As shown in Tables 7 through 9, all three mix design specifications utilize the same sieve sizes to characterize a gradation. Table 10 presents the standard sieve sizes utilized in all three specifications.

Table 10: Standard Sieve Sizes

Sieve Size, mm	Sieve Size, US Standard
50.0	2 in.
37.5	1 ½ in.
25.0	1 in.
19.0	¾ in.
12.5	½ in.
9.5	3/8 in.
4.75	No. 4
2.36	No. 8
1.18	No. 16
0.60	No. 30
0.30	No. 50
0.150	No. 100
0.075	No. 200

Figures 1 through 6 compare the gradation requirements included within the three mix design specifications. These figures present the gradations on a 0.45 power chart. The Superpave specification is the only one to have gradation requirements for a 2 in. (50 mm) maximum aggregate size (37.5 mm NMAS) (Figure 1). Both Item P-401 and Superpave have requirements for a 1 ½ in. (37.5 mm) maximum aggregate size gradation (25.0mm NMAS) (Figure 2). As shown on Figure 2, the gradation requirements are very similar on the No.8 (2.36 mm) sieve. However, because the Superpave requirements do not have control points between the 1 in. (25.0 mm) and No. 8 (2.36 mm) sieves, the Superpave gradation requirements are much less restrictive than Item P-401.

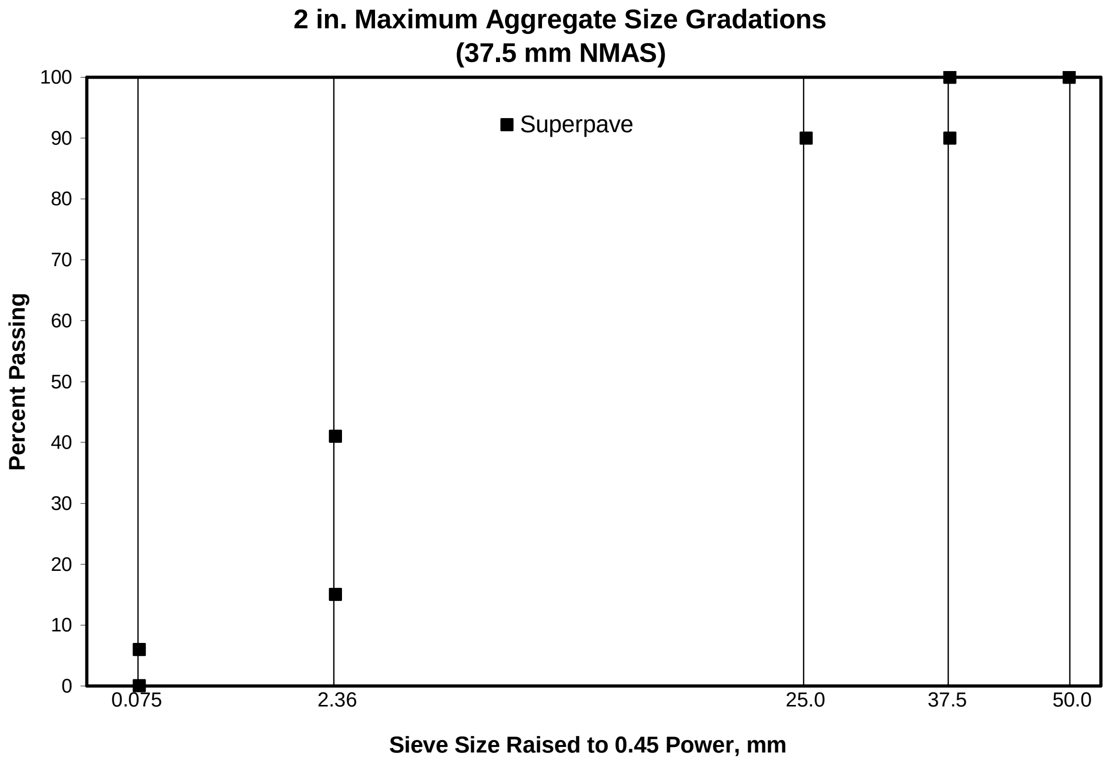

Figure 1: Gradation Requirements for 2 in. Maximum Aggregate Size Gradations

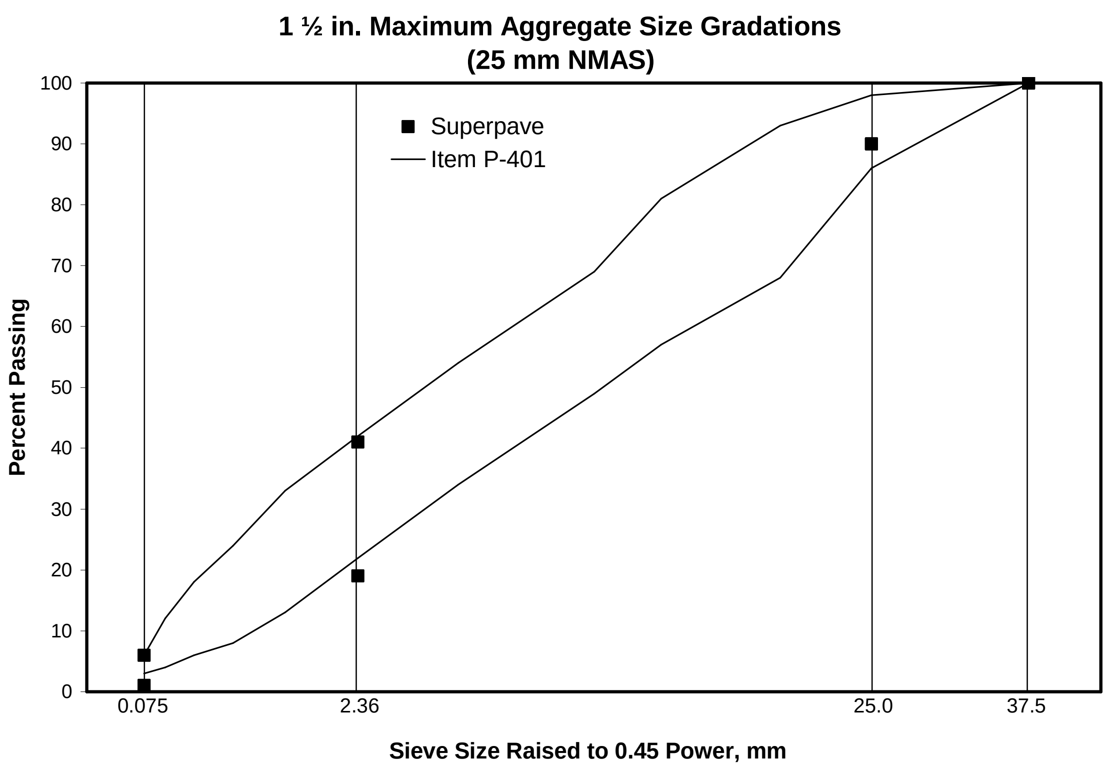

Figure 2: Gradation Requirements for 1 ½ in. Maximum Aggregate Size Gradations

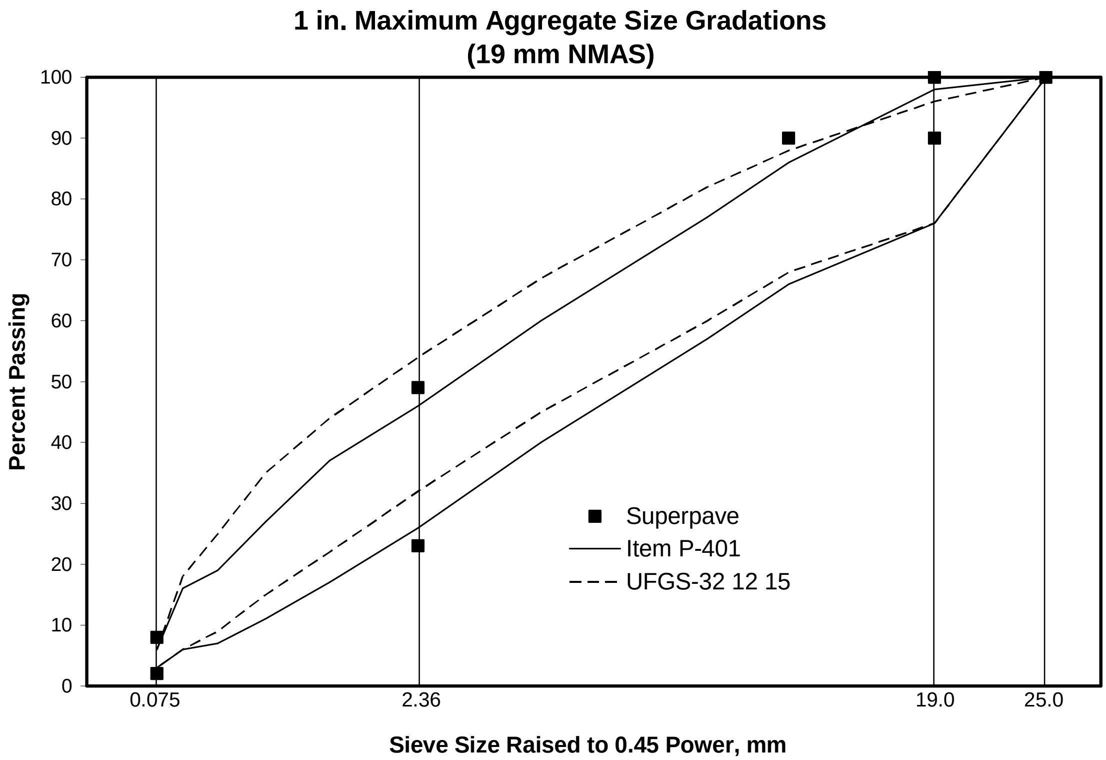

Figure 3: Gradation Requirements for 1 in. Maximum Aggregate Size Gradations

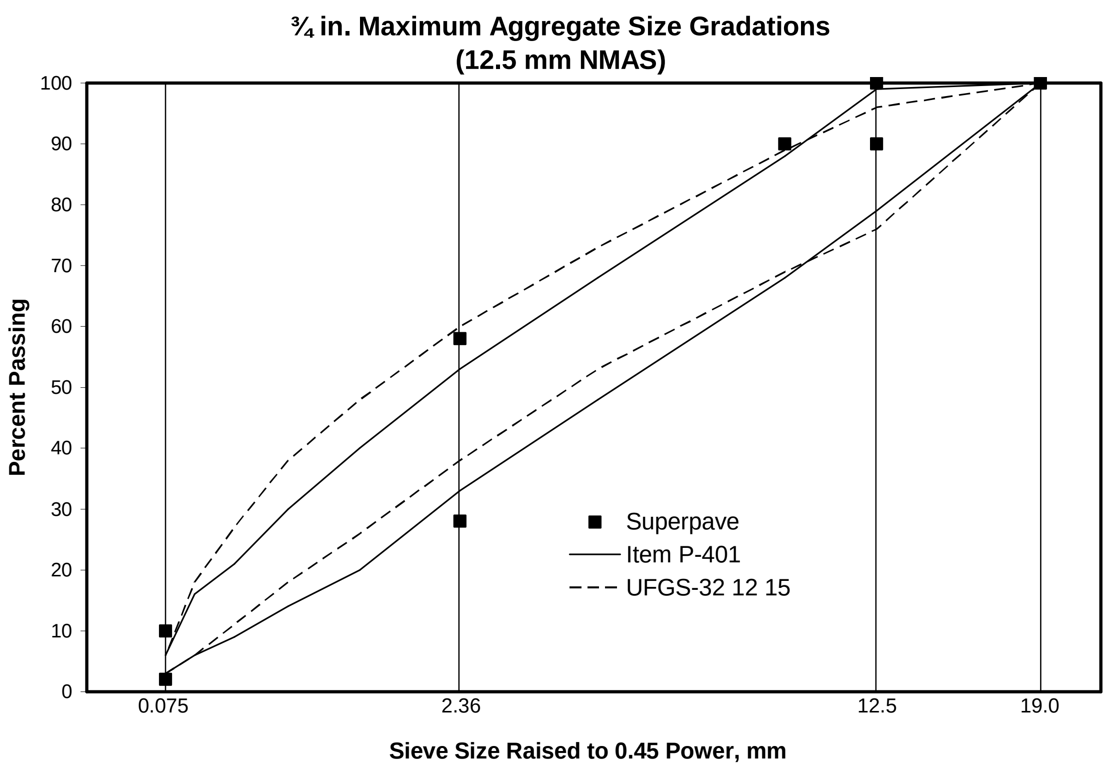

Figure 4: Gradation Requirements for ¾ in. Maximum Aggregate Size Gradations

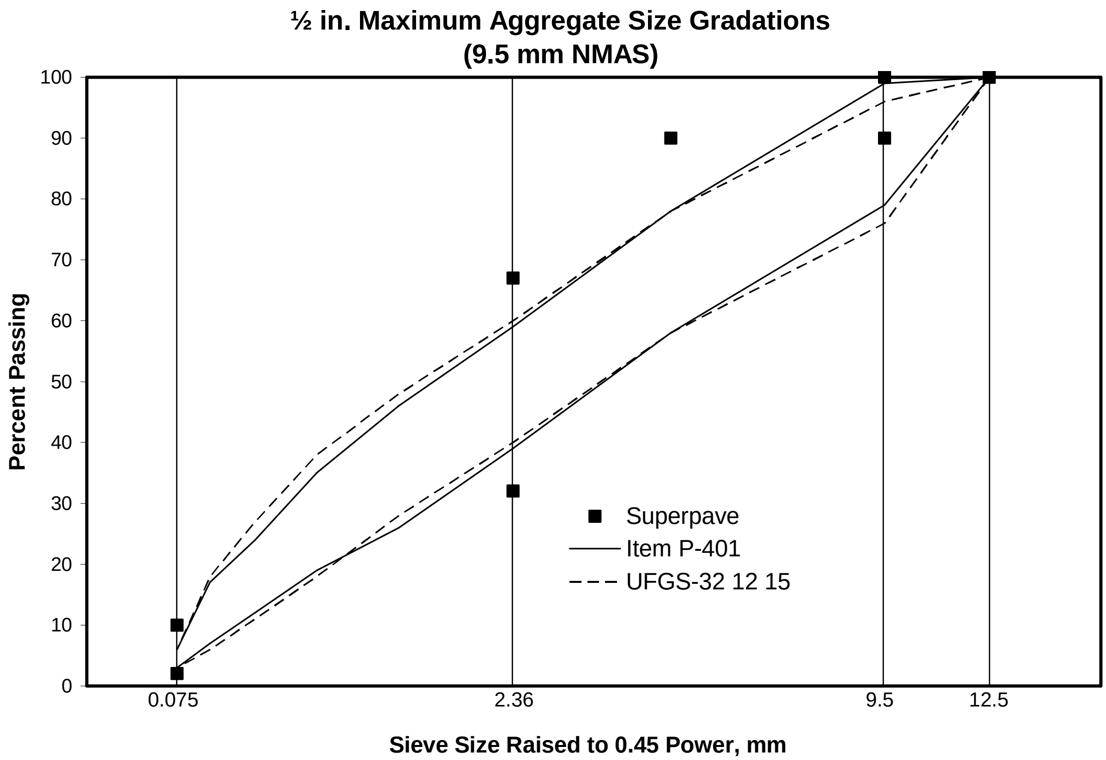

Figure 5: Gradation Requirements for ½ in. Maximum Aggregate Size Gradations

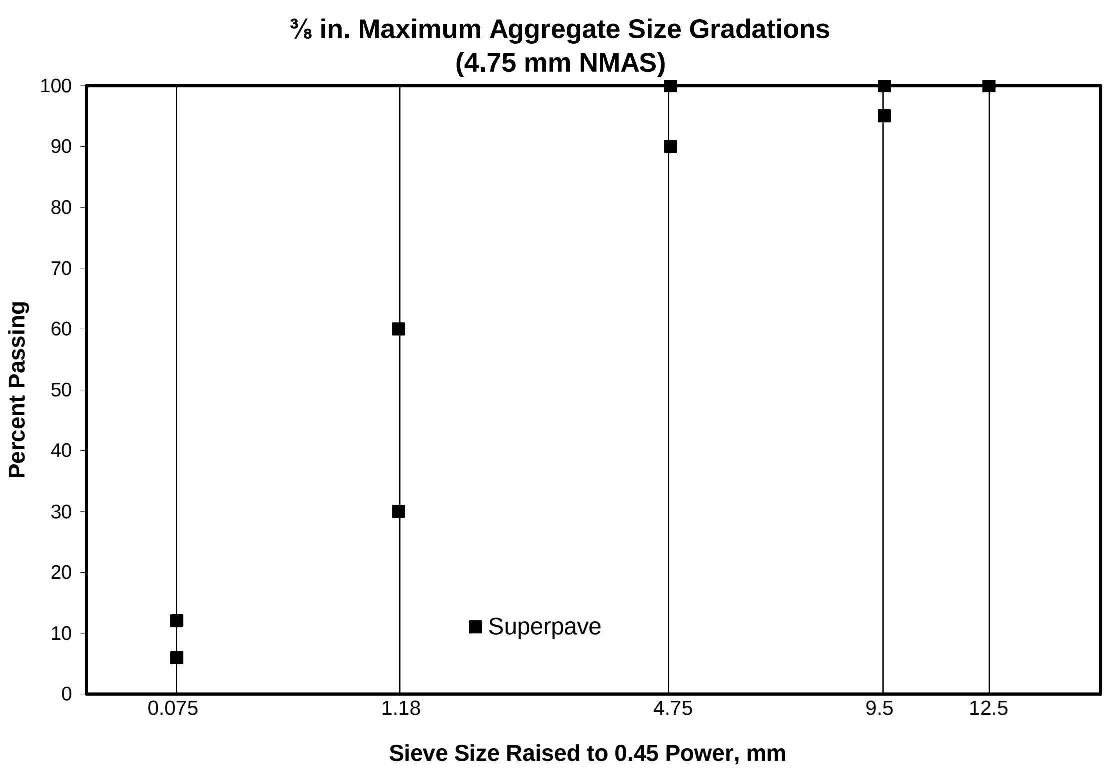

Figure 6: Gradation Requirements for ⅜ in. Maximum Aggregate Size Gradations

Figure 3 presents a comparison of gradation requirements between all three mix design specifications for a 1 in. maximum aggregate size gradation (19.0 mm NMAS). Of the three specifications, the UFGS-32 12 15 specification allows the finest gradation, while the Superpave specification allows the coarsest. The Item P-401 gradation band resides totally within the Superpave control points on the No. 8 (2.36 mm) sieve. The UFGS-32 12 15 gradation band is above the Superpave upper control point on the No. 8 (2.36 mm) sieve signifying the allowance of finer gradations. As for the lower Superpave control point, the UFGS-32 12 15 gradation band is 9 percent finer than the Superpave specification and 5 percent finer than the Item P-401 specification. Both the Item P-401 and UFGS-32 12 15 have a range of 3 to 6 percent passing the No. 200 (0.075 mm) sieve, while the range within the Superpave specification is 2 to 8 percent.

Gradation requirements for ¾ in. maximum aggregate gradations (12.5 mm NMAS) are illustrated in Figure 4. Similar to the 1 in. maximum aggregate size gradation requirements, UFGS-32 12 15 allows the finest gradation and Superpave allows the coarsest. The Item P-401 gradation band is totally included within the Superpave control points on the No. 8 sieve. The UFGS-32 12 15 lower limit on the No. 8 sieve is 10 percent finer than the Superpave lower control point and Item P-401 is 5 percent finer than the Superpave lower control point. Again, both Item P-401 and UFGS-32 12 15 have an allowable range of 3 to 6 percent passing on the No. 200 sieve while the Superpave control points allow 2 to 10 percent.

Figure 5 presents the gradation requirements from the three mix design specifications for ½ in. maximum aggregate size (9.5mm NMAS) gradations. Unlike the two previous gradations, Item P-401 and UFGS-32 12 15 reside totally within the

Superpave control points on the No. 8 sieve. Therefore, HMA designed in accordance with the Superpave specifications for a ½ in. maximum aggregate size can be either finer or coarser than the two historical airfield specifications. Similar to other gradation sizes, Item P-401 and UFGS-32 12 15 are more restrictive on the percentage of material passing the No. 200 sieve with a range of 3 to 6 percent. The Superpave specification for ½ in. maximum aggregate size gradation on the No. 200 sieve is a range of 2 to 10 percent.

The Superpave specification is the only one of the three that includes gradation requirements for a 3/8 in. maximum aggregate size (4.75mm NMAS). Figure 6 illustrates the control points for a 3/8 in. maximum aggregate size gradation.

Summary of Gradation Requirements

The Superpave mix design specification provides the most potential gradation sizes with six. Item P-401 provides requirements for four gradation sizes, while UFGS-32 12 15 provides three gradation bands. Where comparisons can be made, the UFGS-32 12 15 gradation requirements generally allow the finest gradations on the No. 8 sieve, while the Superpave specifications always allow the coarsest gradations. The two historical airfield specifications are much more restrictive in the potential gradations that can be blended than Superpave. By using relatively few control points, the Superpave gradation requirements are much less restrictive, especially for larger maximum aggregate size gradations.

Select Appropriate Optimum Asphalt Binder Content

Once appropriate materials have been selected and the aggregates blended to meet the desired gradation, all three mix design specifications involve adding asphalt binder to the aggregates and performing laboratory compaction in order to evaluate the mixture's volumetric properties. The primary difference between the mix design specifications is the method of laboratory compaction. Item P-401 and UFGS-32 12 15 both specify the use of the Marshall hammer in accordance with Chapter 5 of the Asphalt Institute's MS-2, "Mix Design Methods For Asphalt Concrete and Other Hot Mix Types" (13). The Superpave mix design specifications require the use of the Superpave gyratory compactor (SGC) for the laboratory compaction of HMA during design.

The compactive effort utilized in all three mix design specifications is controlled by the anticipated loadings on the pavement. Within Item P-401, the design compactive effort is based upon the design aircraft gross weight and/or tire pressure. For design aircraft over 60,000 lbs or landing gear tire pressures of more than 100 psi, a design laboratory compactive effort of 75 blows per face of the Marshall hammer is used. Pavements designed for aircraft less than 60,000 lbs or tire pressures less than 100 psi are designed using 50 blows per face. Within UFGS-32 12 15, the design laboratory compactive effort is based upon landing gear tire pressure and location on the airfield. Similar to Item P-401, the HMA used on pavements designed for aircraft having tire pressures greater than 100 psi are to be designed using 75 blows per face of the Marshall hammer. Pavements designed for aircraft having tire pressures less than 100 psi are to be designed using 50 blows per face. UFGS-32 12 15 provides a stipulation that HMA used on shoulders should be designed using 50 blows per face.

The design compactive effort within the Superpave mix design specification is also based upon expected loadings. Within Superpave, the design compactive effort is defined as the design number of gyrations (N_{design}) in the SGC. Pavements designed for heavier or more loadings are designed using more gyrations. Currently, there are four design gyrations levels within Superpave 50, 75, 100 and 125 gyrations. The lowest, 50 gyrations, is generally specified for low volume pavements, while the highest, 125 gyrations, is generally specified for pavements with a very high volume of heavy loadings.

As stated previously, all three specifications include volumetrics in the selection of the optimum asphalt binder content. Volumetric properties, such as voids in total mix (VTM, generally called air voids), voids in mineral aggregate (VMA), and voids filled with asphalt (VFA) are all included within the three specifications. Samples of HMA are compacted using the design compactive effort at varying asphalt binder contents. The volumetric properties of each sample are then determined and compared to specification limits. This is true for all three mix design specifications.

An added feature to the mix design process, when using the Marshall hammer, is the measurement of Marshall stability and flow. These two tests are utilized as proof tests on the designed mix. Both Item P-401 and UFGS-32 12 15 include a minimum value of Marshall stability and a range of allowable flow values.

Marshall Mix Design Method

In order to select optimum asphalt binder content using the Marshall method of mix design, the relationships between asphalt binder content and air voids, VMA, VFA and stability are developed. The next step is to select an asphalt binder content that meets all requirements, similar to Figure 7. Though Item P-401 and UFGS-32 12 15 both utilize the Marshall mix design method, the specified criteria within these two specifications are slightly different. Tables 11 and 12 present the mix design criteria for Item P-401 and UFGS-32 12 15, respectively. The primary differences between these two specifications are the design air void range and the allowable range in flow values. Item P-401 allows optimum asphalt binder contents to be selected between 2.8 and 4.2 percent air voids, while the UFGS-32 12 15 specification has a range of 3 to 5 percent air voids for selection of optimum asphalt binder content. Requirements for flow values within Item P-401 range from 10 to 14 while within UFGS-32 12 15 the allowable range is 8 to 16.

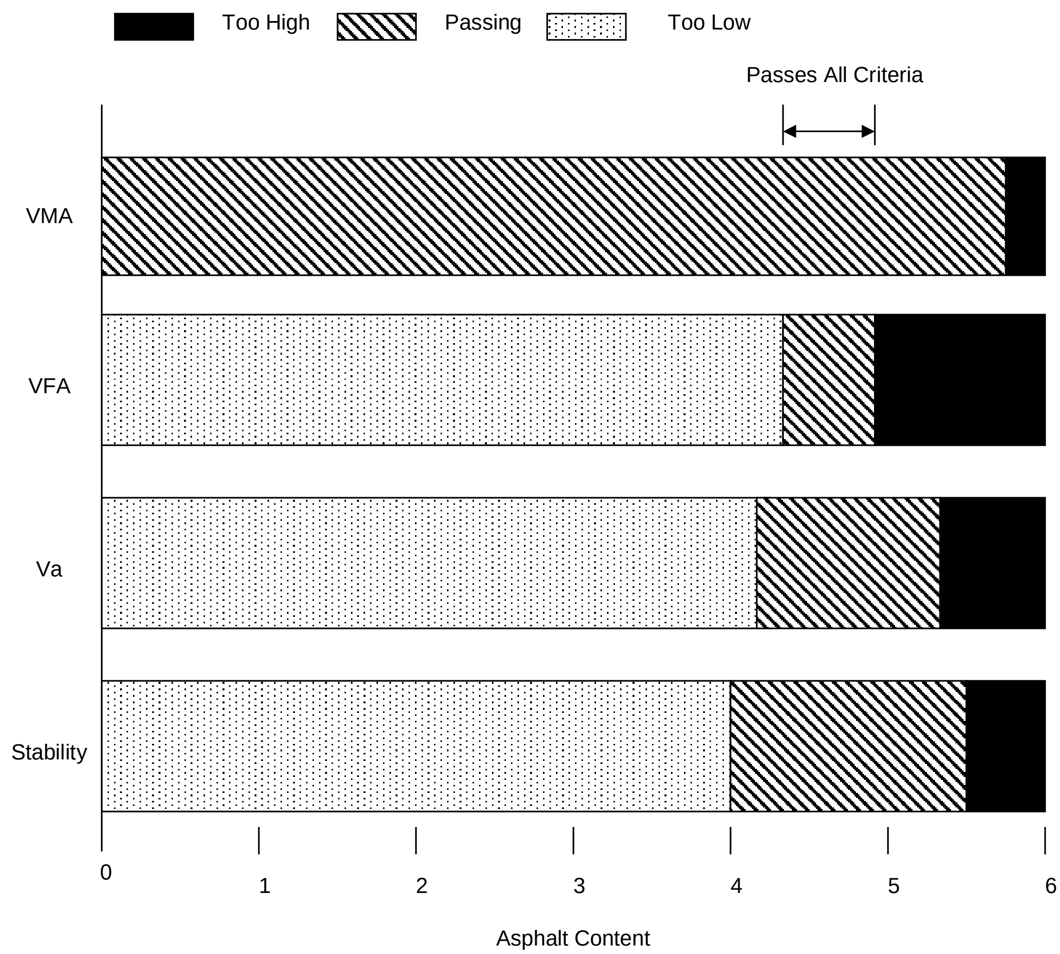

Figure 7: Selection of Optimum Asphalt Binder Content (13)

Table 11: Marshall Design Criteria - Item P-401

Test Property	Pavements Designed for Aircraft Gross Weight of 60,000 lbs or More or Tire Pressures of 100 psi or More	Pavements Designed for Aircraft Gross Weights Less than 60,000 lbs. or Tire Pressures Less Than 100 psi
Number of Blows	75	50
Stability, pounds (newtons)	2150(9564)	1350(6005)
Flow, 0.01 in. (0.25 mm)	10-14	10-18
Air Voids (percent)	2.8-4.2	2.8-4.2
Percent VMA (minimum)	See Table 13	See Table 13

Table 12: Marshall Design Criteria - UFGS-32 12 15

Test Property	75 Blow Mix	50 Blow Mix
Stability, Newtons minimum	9560	6000
Flow, 0.25mm	8-16	8-18
Air voids, percent	3-5	3-5
Percent VMA (minimum)	See Table 13	See Table 13
Dust Proportion	0.8-1.2	0.8-1.2
TSR, Minimum Percent	75	75

Also included within Tables 11 and 12 is a reference to Table 13. Table 13 presents the minimum VMA requirements within both Item P-401 and UFGS-32 12 15. As shown in Table 13, there are differences in the minimum VMA requirements. Item P-401 requires 1.0 percent more VMA than UFGS-32 12 15 for a given maximum aggregate size gradation.

Table 13: Minimum Percent Voids in Mineral Aggregate

Maximum Particle Size		Minimum VMA	Minimum VMA
in.	mm	P-401	UFGS-32 12 15
½	12.5	16.0	15.0
¾	19.0	15.0	14.0
1	25.0	14.0	13.0
1-1/2	37.5	13.0	---

UFGS-32 12 15 includes a specification range for dust proportion, 0.8 to 1.2, that is not included in Item P-401. Dust proportion is calculated as the percent aggregate mass passing the No. 200 sieve divided by the effective asphalt binder content.

The optimum binder content is selected as an asphalt binder content that meets all volumetric criteria as well as stability and flow. If any of the volumetric properties, stability or flow are not met, modifications to the materials and/or blend must be made.

Superpave Mix Design Method

Similar to the Marshall method, samples must be compacted at varying asphalt binder contents to the design compactive effort (N_{design}). Unlike the Marshall mix design method, the Superpave method involves selection of optimum asphalt binder content based solely on volumetric properties (i.e., no proof test). Some state agencies do utilize performance tests, such as the Asphalt Pavement Analyzer, Hamburg Wheel Tracking

Device, etc., as a proof test; however, there is currently no national standard method of test utilized as a proof test.

Volumetric properties included within the evaluation include air voids, VMA and VFA just as in the Marshall method (Table 14). Dust proportion is also included, similar to UFGS-32 12 15. However, there are two volumetric properties included in the Superpave mix design method that are not included in the Marshall method: percent theoretical maximum density at the initial number of gyrations ($\%G_{mm}@N_{initial}$) and percent theoretical maximum density at the maximum number of gyrations ($\%G_{mm}@N_{maximum}$). Unlike the impact of the Marshall hammer, the SGC kneads the HMA during compaction. During this kneading compaction, the SGC records the height of HMA after every gyration. This allows for the evaluation of the HMA at various gyration levels. Requirements for $N_{initial}$ are included within Superpave in an effort to prevent tender HMA mixes during construction. High values of $\%G_{mm}@N_{initial}$ indicate a mixture that compacts readily. Requirements for $N_{maximum}$ are provided to identify HMA mixes that may continue to compact over time resulting in a rut prone mixture.

Table 14: Superpave HMA Design Criteria

Design ESALs[a] (Million)	Required Relative Density, Percent of Theoretical Maximum Specific Gravity			Voids in the Mineral Aggregate (VMA), Percent Minimum Maximum Aggregate Size, mm						Voids Filled with Asphalt (VFA) Range, [b] Percent	Dust-to-Binder Ratio Range[c]
	$N_{initial}$	N_{design}	N_{max}	2	1.5	1	¾	½	3/8		
<0.3	≤91.5	96.0	≤98.0	11.0	12.0	13.0	14.0	15.0	16.0	70-80[d]	0.6-1.2
0.3 to <3	≤90.5	96.0	≤98.0	11.0	12.0	13.0	14.0	15.0	16.0	65-78	0.6-1.2
3 to <10	≤89.0	96.0	≤98.0	11.0	12.0	13.0	14.0	15.0	16.0	65-75[e]	0.6-1.2
10 to <30	≤89.0	96.0	≤98.0	11.0	12.0	13.0	14.0	15.0	16.0	65-75[e]	0.6-1.2
≥30	≤89.0	96.0	≤98.0	11.0	12.0	13.0	14.0	15.0	16.0	65-75[e]	0.6-1.2

[a] Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs or 20 years.

[b] For 37.5-mm nominal maximum size mixtures, the specified lower limit of the VFA range shall be 64 percent for all design traffic levels.

[c] For 4.75-mm nominal maximum size mixtures, the dust-to-binder ratio shall be 0.9 to 2.0.

[d] For 25.0-mm nominal maximum size mixtures, the specified lower limit of the VFA range shall be 67 percent for design traffic levels <0.3 million ESALs.

[e] For design traffic levels >3 million ESALs, 9.5-mm nominal maximum size mixtures, the specified VFA range shall be 73 to 76 percent and for 4.75-mm nominal maximum size mixtures shall be 75 to 78 percent.

Similar to the Marshall method, the relationships between asphalt binder content and the various volumetric properties are developed. Optimum asphalt binder content is defined as the asphalt binder content that results in 4.0 percent air voids and meets all other requirements shown in Table 14. If any volumetric properties do not meet requirements, the materials and/or gradation must be altered.

Summary of Comparison for Selection of Optimum Asphalt Binder

Both the Marshall mix design method utilized in Item P-401 and UFGS-32 12 15 and the Superpave mix design method rely on volumetric properties to select the optimum asphalt binder content for an HMA. The volumetric properties of air voids, VMA and VFA are included in all three. VFA is not directly included within Item P-401; however, VFA is indirectly specified because of the requirements on air voids and VMA.

Item P-401 and UFGS-32 12 15 both allow the mix designer to select the optimum asphalt binder content based upon a range of air voids, while the Superpave mix design system requires selection of optimum binder content at 4.0 percent voids. The biggest difference in selecting optimum binder content probably is the method of compaction. Item P-401 and UFGS-32 12 15 specify the Marshall hammer which compacts the HMA through impact. The Superpave mix design system specifies a Superpave gyratory compactor which compacts the HMA through kneading. An added benefit of the SGC is that the compaction characteristics of the HMA can be evaluated. This has resulted in two additional volumetric properties that are evaluated during selection of optimum asphalt: $\%G_{mm}@N_{initial}$ and $\%G_{mm}@N_{maximum}$. Another major difference is that Item P-401 and UFGS-32 12 15 both utilize Marshall stability and flow as a proof test. Currently, there is no proof test within Superpave.

Comparison of Moisture Susceptibility Requirements

The final step in all three mix design methods is to evaluate the designed mix for moisture susceptibility. All three methods utilize tensile strength ratios to define moisture susceptibility. Both Item P-401 and UFGS-32 12 15 specify ASTM D 4867, "Effect of Moisture on Asphalt Concrete Paving Mixtures," to indicate the potential for moisture damage. Both also require a minimum tensile strength ratio of 75 percent. The Superpave mix design specification requires the use of AASHTO T 283, "Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage," for measuring moisture susceptibility. A minimum tensile strength ratio of 80 percent is required for Superpave designs.

Summary of Critical Comparison

All three mix design specifications have many similarities. All include four primary steps: selection of materials, blending of selected materials, selection of optimum asphalt binder content and evaluation of moisture susceptibility. Each method has aggregate property criteria to ensure angular and clean aggregates that are properly shaped. All three specifications also ensure tough and durable aggregates; though, local agencies specify appropriate toughness and durability criteria within Superpave source properties. With respect to asphalt binders, all three allow the use of Performance Graded asphalt binders.

There are minor differences in how the aggregates can be blended. The Superpave gradation requirements allow for the most gradation options (maximum aggregate sizes). For a given maximum aggregate size gradation, use of the Superpave control points also allows for the most gradation shapes. The two historical airfield specifications are more restrictive because of the use of gradation bands. The UFGS-32 12 15 specification generally allows the finest gradations, while the Superpave specification allows the coarsest.

The biggest difference in designing HMA is that the two historical airfield specifications require laboratory compaction with the Marshall hammer, while the Superpave specification requires the Superpave gyratory compactor. These two methods of laboratory compaction are very different. Another difference is that the two airfield specifications utilize Marshall stability and flow as a proof test during mix design. Superpave does not currently include a proof test. When selecting the optimum binder content all three methods are similar in that volumetrics are used. Air voids, VMA and

VFA are all directly or indirectly specified. There are slight differences in the specified volumetric requirements; the biggest of which is the use of a range in design air voids within the Marshall methods.

With respect to moisture susceptibility, all three methods utilize tensile strength ratios to provide a measure of moisture damage potential. The methods specified have slight differences, but the underlying test method is the same. Specification values only differ slightly.

In summary, the three mix design specifications have many similarities. Without question, the goal of each mix design method is to produce an HMA that is stable and durable for its intended purpose. The primary issues that must be addressed as part of AAPTP 04-03 are the laboratory compactive effort, appropriate volumetric criteria for selection of optimum binder content, appropriate gradation sizes and shapes for airfields, method and criteria for evaluating moisture susceptibility and appropriate test method and criteria for materials selection.

CHAPTER 4
Field Visits

During the course of the study ten different airfields were visited across the United States. The ten selected airfields represent a range of climates, traffic levels and FAA regions. Figure 8 shows the distribution of the airfields across the country as well as the Long Term Pavement Performance (LTPP) climatic zone designations. These climatic zones are shown on Figure 8 because the research team made a concerted effort to identify airfields that had been exposed to different climates. Table 15 shows the breakdown of the airfields by traffic classification.

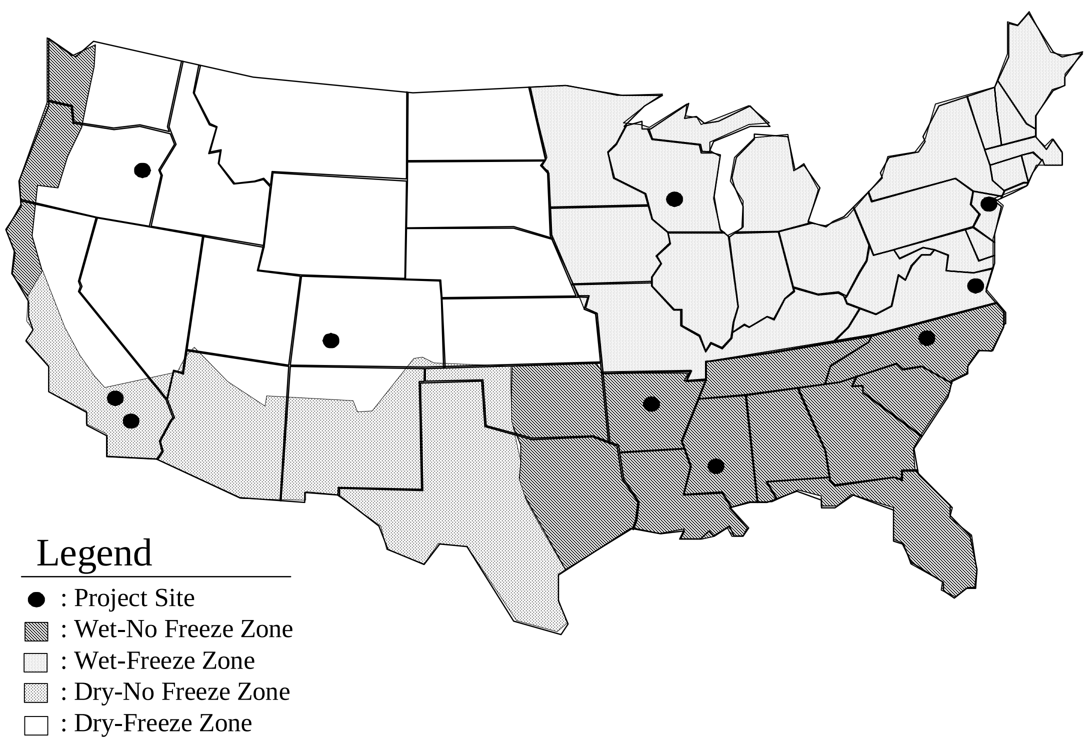

Figure 8: Locations of Visited Airfields

Table 15: Airport Field Visit Traffic Level Designations

Traffic Level*	Light	Medium	Heavy
Airport	Jacqueline Cochran Regional Airport	Jackson-Evers International Airport	Naval Air Station - Oceana
	Mineral County Memorial Airport	Little Rock Air Force Base	Volk Field
	Oxford-Henderson Airport	Newark Liberty International Airport	
		Palm Springs International Airport	
		Spokane International Airport	

* Light traffic level airfields are considered to experience aircraft less than 60,000 lbs, medium traffic level airfields experience air traffic with tire pressures greater than 100 psi but less than 200 psi or gross aircraft weights in excess of 60,000 lbs, and the heavy traffic level receives aircraft with tire pressures in excess of 200 psi.

Airfield Information

Jacqueline Cochran Regional Airport

The Jacqueline Cochran Regional Airport (TRM) in Thermal, California was visited on March 14, 2007. TRM is considered a General Aviation airport with 76,390 total operations in 2005. The airfield is owned by the County of Riverside, California and is used mainly by recreational and business aircraft. The airport is located in the FAA Western Pacific (AWP) region and is designated in the Dry-No Freeze LTPP climatic zone for the TRM is at an elevation of 114 feet below sea level. The airport is located in a desert climate region with an average yearly temperature of 89°F. The average high is in July at 108°F and the average low is in December and January with a temperature of 42°F. The average rainfall for the year is 5.5 inches. LTPPBind 3.1 Software indicates that the air temperature during the year ranges from 29°F to 114°F and the pavement temperature ranges from 35°F to 159°F.

The pavement examined at TRM was Taxiway F. Taxiway F leads to Runway 17-35 that has a critical aircraft rating of a Boeing Business Jet 2 and a rated pavement strength of 174,700 pounds (dual wheel main landing gear configuration). Boeing Business Jet 2 aircraft have design tire pressures of 200 psi.

The pavement structure of Taxiway F is approximately 2 inches of ¾ inch maximum aggregate size HMA utilizing AR-4000 binder under 2 inches of ¾ inch maximum aggregate size HMA also utilizing an AR-4000 binder. The performance of the pavement appears excellent. A visual condition survey of a large section of the pavement indicated no rutting and only one low severity crack. Figures 9 and 10 show the crack propagation from the construction joint and a close up of the crack and pavement texture. Other visible distresses included some oxidation with occasional raveling and some construction related segregation.

Figure 9: Transverse Crack Initiated at Construction Joint

Figure 10: Close-up of Transverse Crack and Surface Texture

Mineral County Memorial Airport

The Mineral County Memorial Airport (FAA Designation - C24) in Creede, Colorado was visited on April 4, 2007. C24 is a General Aviation airport with 2,000 estimated operations in 2006. The airfield is owned by Mineral County, Colorado and is used mainly by recreational aircraft with only three single engine aircraft based at this location.

The airport is located in the FAA Northwest Mountain (ANM) region and is designated in the Dry- Freeze climatic LTPP climatic zone. C24 is at an elevation of 8,680 feet above sea level. The airport is located in a temperate climate region with an average yearly temperature of 43°F. The average high is in July at 78°F and the average low is in December and January with a temperature of 6°F. The average precipitation for the year is 13.5 inches. LTPPBind Software indicates that the air temperature during the year ranges from -33°F to 81°F and the pavement temperature ranges from -11°F to 111°F.

The pavement examined at C24 was runway 07/25. Runway 07/25 has a critical pavement rating of 12,500 pounds (single wheel landing gear configuration).

The pavement structure of Runway 07/25 is approximately 3 inches of ¾ inch maximum aggregate size HMA utilizing PG 58-34 binder on top of improved subgrade. The pavement was laid in 2000 and has had one fog seal since that time. The performance of the pavement appears very good. A visual condition survey of a large section of the pavement indicated no rutting; only a few low severity transverse cracks; and all of the longitudinal construction joints had moderate cracks. Figure 11 is an overall view of the runway showing the sealed longitudinal cracks. Figure 12 presents a close up of a transverse crack and the pavement surface texture. Other visible distresses included occasional raveling.

Figure 11: Runway 07/25 Showing Longitudinal Joint Cracks

Figure 12: Transverse Crack and Surface Texture on Runway 07/25

Oxford-Henderson Airport

The Oxford-Henderson Airport (HNZ), located in Oxford, North Carolina, was visited on September 20, 2007. HNZ is a publicly owned airport (controlled by the Oxford-Henderson Airport Authority) that is open to public use. According to reported aircraft data, HNZ has an average of 58 operations per day, or about 21,170 operations per year. Among the aircraft based at the airport are 42 single-engine general aviation planes and 3 multi-engine general aviation planes.

The Oxford-Henderson Airport is located in the FAA Southern region and is designated in the Wet – No Freeze LTPP climatic zone. The airport is at an elevation of 527 feet above sea level. The average yearly high and low temperatures for the airport are 71 and 49°F, respectively. The highest temperatures occur during the month of July (average of 89°F), while the lowest temperatures occur in January (average of 29°F). The annual average rainfall for the airport is 38.0 inches. LTPPBind Software indicates that the ambient air temperature ranges from 8 to 94°F during the year, while pavement temperatures range from 18 to 140°F.

The pavement that was examined at the Oxford-Henderson Airport was the parallel taxiway accompanying the 6/24 runway and was constructed in 2003. The 6/24 runway has dimensions of 100 feet wide by 5,002 feet long. This runway is the only runway for the airport, and has a weight capability of up to 30,000 pounds for a single wheel aircraft.

A visual inspection of the entire taxiway was performed. There are actually two separate Superpave mixes that make up the complete taxiway, therefore both mixes were evaluated. These mixes were labeled 9.5mm A and 9.5mm B. Each mix used the same aggregates; the difference between the two mixtures is in the aggregate percentages and the design compaction level. The 9.5mm A mix was designed using a gyration level of 50 gyrations, and the 9.5mm B mix used 75 gyrations as the design level. Figure 13 shows the location of the two HMA mixes, as well as the corresponding core locations for each HMA mixture. Overall, it was determined that the taxiway, as a whole, was in excellent condition. No visible rutting was observed. Minor longitudinal cracking was observed in the 9.5mm B mix, with moderate longitudinal cracking seen in the 9.5mm A mixture (Figure 14). The majority of the cracking was observed in the turnout area where the runway meets the taxiway (Figure 15). One interesting pavement distress is shown in Figure 16. Moisture trapped in the pavement is thought to have evaporated and traveled up through the pavement, and created little bumps in the surface when it was released. The bumps are dark colored due to asphalt binder being contained in the vapor as it traveled up through the pavement.

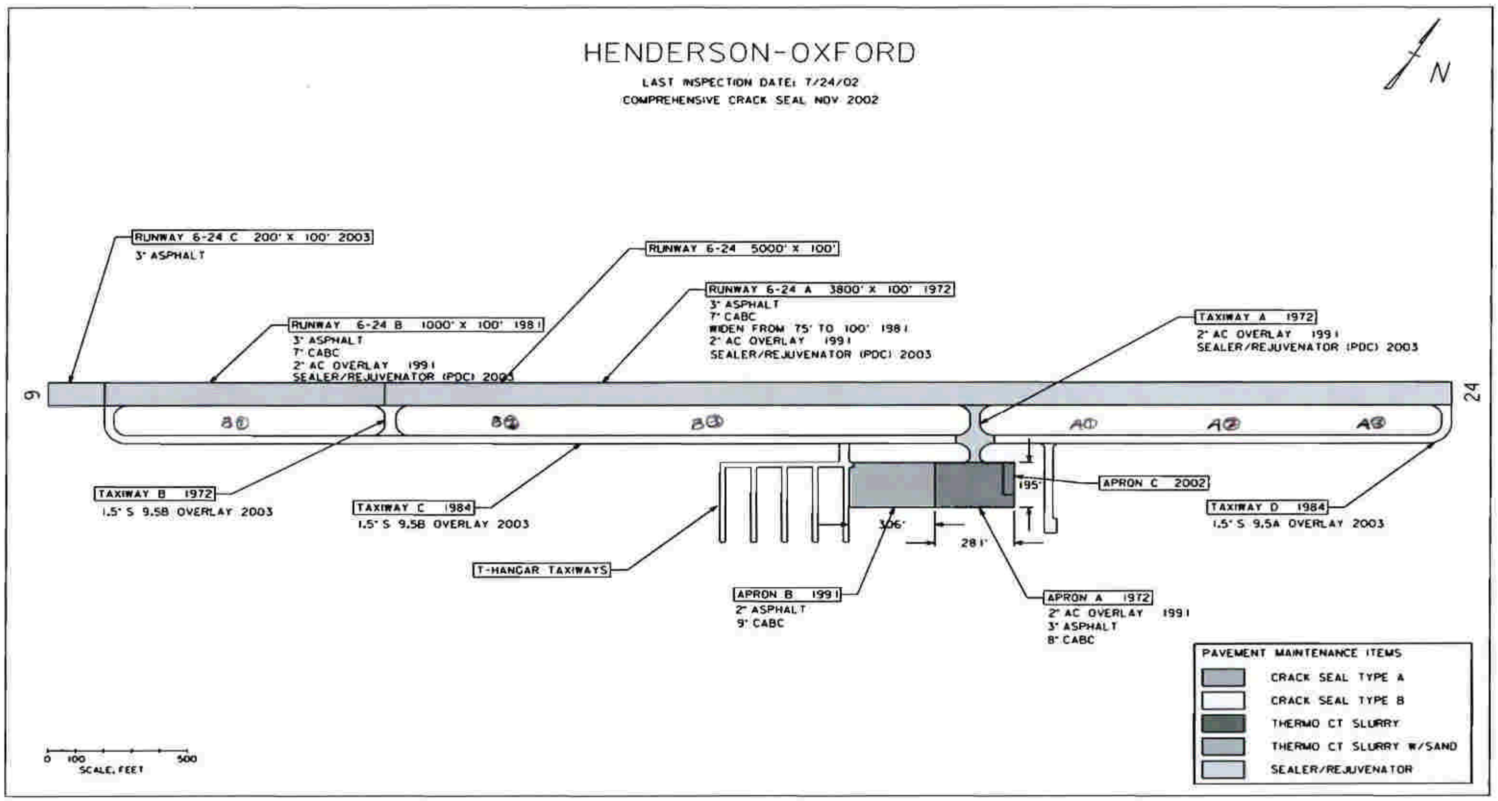

Figure 13: Asphalt Mixture and Core Locations

Figure 14: Longitudinal Cracking Along Taxiway, 9.5mm A

Figure 15: Cracking in the Turnout to Taxiway, 9.5mm A.

Figure 16: Pavement Distress Caused by Vapor Pressure, 9.5mm A

Little Rock Air Force Base

The Little Rock Air Force Base (FAA Designation - LRF) was visited on April 11, 2007. LRF is a United States Air Force airfield home to approximately 100 C-130 aircraft with 61,350 operations in 2006. The assault strip is a 3,482 foot airstrip that is 60 feet wide and is used for short runway practice by the C-130 aircrews.

The airport is located in the FAA Southwest (ASW) region and is designated in the Wet- No Freeze climatic zone. LRF has an elevation of 311 feet above sea level. The airport is located in a temperate climatic region with an average yearly temperature of 62°F. The average high is in July at 93°F and the average low is in January with a temperature of 31°F. The average precipitation for the year is 50.9 inches. LTPPBind Software indicates that the air temperature during the year ranges from 8°F to 98°F and the pavement temperature ranges from 19°F to 142°F.

The pavement examined at LRF was runway 069/249. Runway 069/249 is the aforementioned assault strip and was designed for the C-130 with a maximum takeoff weight of 175,000 pounds (single tandem landing gear configuration) with tire pressure of 95 psi. The assault strip receives approximately 27,000 operations a year.

The pavement structure of the assault strip is approximately 15 inches of hot mix asphalt. The improvements to the airstrip in 1998 consisted of a milling and replacing the top 3 inches with 12.5 mm NMAS HMA utilizing PG 70-22 binder on top of improved subgrade. The HMA within the top 3 inches was designed using the Superpave mix design system with an N_{design} of 139 gyrations. The only maintenance to the airstrip since construction appeared to be sealing of the longitudinal joints and occasional rubber removal. Overall, the performance of the pavement appeared good. A condition survey of the pavement revealed some raveling. Bleeding, shown in Figure 17, was apparent in about a 30 foot section of the runway. There were no transverse cracks, but the longitudinal joints were cracked and had been treated. The last 500 feet of the west end of the runway contained rutting and cracking in the wheel path. Figure 18 shows water from the coring operation collecting in the ruts. Figure 19 shows measured rut depths of one inch. Figure 20 shows the cracking at the end of the runway. Most landings by the C-130 approach from the east and taxi off to the west end of the runway. This channelized-stacked behavior, slow speeds and the aircraft turning off of the assault strip likely contributed to the rutting and cracking.

Figure 17: Bleeding on Assault Strip

Figure 18: Core Water Collecting in Rut

Figure 19: Rut Depth of One Inch

Figure 20: Cracking Near End of Runway

Naval Air Station-Oceana

Naval Air Station (NAS) Oceana (FAA Location Identifier NTU) was visited on October 6, 2007. NAS Oceana is a United States Navy master jet base and home to approximately 17 squadrons of F/A-18 Hornets and F/A-18 Super Hornets (more than 200 aircraft). Prior to their retirement in October 2006, the base also housed F-14 Tomcats. Heavier aircraft, such as C-17 and C-141 cargo planes also make routine landings. Approximately 250,000 takeoffs and landings were reported in 2006 (NAS Oceana Web Site Accessed 12/18/07). NAS Oceana is served by one 12,000 foot and three 8,000 foot runways.

The airport is located in the FAA Eastern (AEA) region and is designated in the Wet-Freeze LTPP climatic zone. NAS Oceana is at an elevation of approximately 23 feet above sea level. The airport is located in a temperate climatic region with an average yearly temperature of 70 °F. The average high air temperature of 87 °F occurs in July and the average low air temperature of 32 °F occurs in January. The average rainfall is 45.2 inches. LTPPBind Software indicates that the historical air temperature extremes during the year range from 13 to 93 °F and the pavement temperature ranges from 21°F to 134°F.

Rutting measurements and cores were taken from Taxiway Alpha. Taxiway Alpha is a 75 foot wide parallel taxiway to the primary runway. The taxiways and runways at NAS Oceana are comprised of 6 to 7 inches of HMA over 10 inches of concrete. In some areas, an asphalt treated paving fabric was used in an attempt to reduce reflective cracking.

Taxiway Alpha receives approximately 60 to 150 passes per day. A centerline is painted on the taxiway for the nose gear to follow. There are two distinct pairs of wheel-path ruts, particularly on the southwestern end of Taxiway Alpha. The two different pairs of wheel-path ruts correspond to the main gear widths of the F/A-18 (10.2 feet) and F-14 (approximately 17.6 feet). Both planes use tricycle-type landing gear.

Taxiway Alpha was paved in 1986. A portion of Taxiway Alpha was overlaid in 2000. The mix would be defined as a 19.0mm NMAS gradation by Superpave definitions. The Navy UFGS-32 12 15 mix used for the overlay was a 75-blow Marshall design with a PG 70-22 binder. In an inspection of Taxiway Alpha, rut depths up to 1 3/8 in were measured. Figure 21 shows water from the coring operation ponding along one of the longitudinal joints near core location 1. The general view of the rutting is shown in Figure 22. Limited reflective cracks were observed on Taxiway Alpha (Figure 23).

Figure 21: Water Ponding along Longitudinal Joint (Right) and Wheel Rut (Left)

Figure 22: Rutting on Taxiway Alpha

Figure 23: Reflective Cracking

Volk Field

Volk Field (FAA Designation - VOK) was visited on June 12, 2007. VOK is a United States Air National Guard airfield located in Camp Douglas, Wisconsin. The airfield, built in the 1950's, is generally used for practice maneuvers and experiences traffic from all varieties of aircraft. The airfield received over 6,800 operations in 2006. This number includes 424 operations by F-16 and F-18 aircraft.

The airport is located in the FAA Great Lakes (AGL) region and is designated in the Wet-Freeze LTPP climatic zone. VOK is at an elevation of 912 feet above sea level. The airport is located in a temperate climatic region with an average yearly temperature of 46°F. The average high is in July at 84°F and the average low is in January with a temperature of 6°F. The average precipitation for the year is 32.3 inches. LTPPBind

Software indicates that the air temperature during the year ranges from -27°F to 91°F and the pavement temperature ranges from -12°F to 127°F.

The pavement examined at VOK was runway 09/27. Runway 09/27 is approximately 9,000 feet long. An aerial view of the runway is shown in Figure 24 with the approximate core locations shown. The first and last 1,600 feet of the runway is paved with Portland Cement Concrete that has been grooved. The center section of the runway is 5,800 ft of grooved asphalt.

The pavement structure of the assault strip is approximately 7 inches of hot mix asphalt over a crushed aggregate base, rapid draining material and granular subbase. The runway was reconstructed in 1999 with a 12.5 mm NMAS Superpave designed mixture for the surface course. The mixture was designed using 109 gyrations from the Superpave gyratory compactor as the N_{design} level and utilized a PG 64-28 asphalt binder. The only maintenance to the airstrip since construction of the surface layer appeared to be sealing of the longitudinal joints.

Overall, the performance of the pavement was good. A visual condition survey of the pavement revealed an occasional transverse crack like the one shown in Figure 25, the construction joints were opening and there were also longitudinal cracks developing approximately midway between the longitudinal construction joints. Figure 26 illustrates that some of the longitudinal cracks had been treated, but most had not. There were occasional aggregate pop outs in the pavement like the one shown in Figure 27, but the pavement is swept often so there was no loose material on the runway. There was no rutting observed on the runway and the grooving of the asphalt was performing well.

Figure 24: Aerial View of Airfield with Core Locations

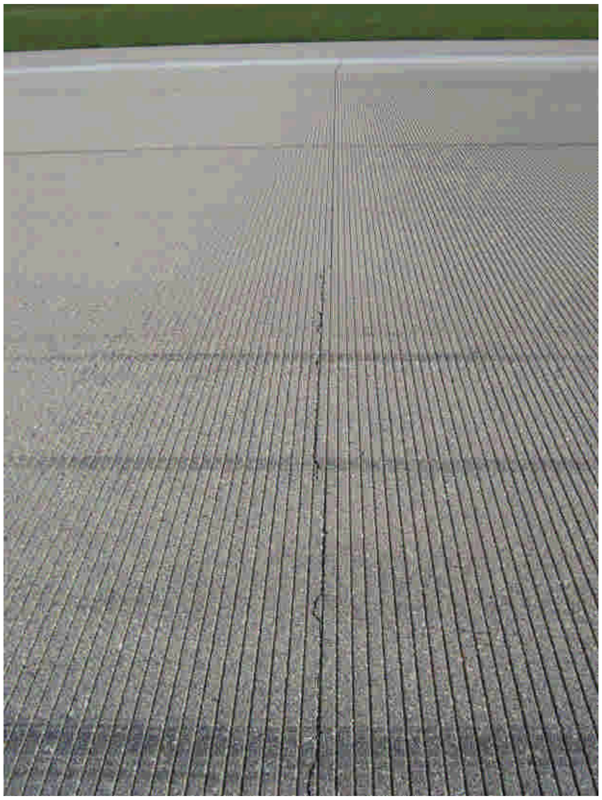

Figure 25: Horizontal Crack on Runway

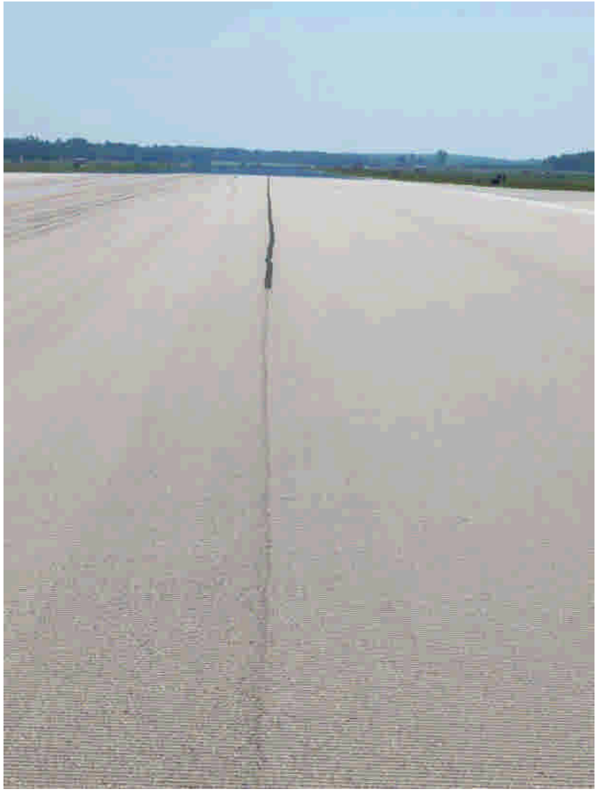

Figure 26: Partially Sealed Longitudinal Crack at Constructional Joint

Figure 27: Example of Occasional Popout and Performance of Grooves

Jackson-Evers International Airport

The Jackson-Evers International Airport (FAA Designation - JAN) was visited on October 12, 2007. JAN is a small primary hub commercial service airport located in Jackson, MS. JAN had 57, 393 operations in 2006 with two 8,500 foot runways.

The airport is located in the FAA Southern (ASO) region and is designated in the Wet- No Freeze, LTPP climatic zone. JAN is at an elevation of 346 feet above sea level. The airport is located in a temperate climatic region with an average yearly temperature of 64°F. The average high is in July at 91°F and the average low is in January with a temperature of 35°F. The average precipitation for the year is 56.0 inches. LTPPBind Software indicates that the air temperature during the year ranges from 13°F to 97°F and the pavement temperature ranges from 24°F to 146°F.

The pavement examined at JAN was runway 16R/34L. Runway 16R/34L is known as the west runway and carries commercial passenger and cargo planes. The runway also receives traffic from military aircraft such as the C-17 and C-137 from the Army National Guard Base located on the grounds.

The pavement structure of the runway is approximately 16 inches thick. The runway was last overlain in 1996; therefore, the examined pavement layer was 11 years old. The HMA mixture was a ¾ in. maximum aggregate size gradation mixture designed with as AC-30 asphalt binder. Overall, the performance of the pavement is good. A condition survey of the pavement revealed some cracking at the construction joints and minimal ravelling (shown in Figure 28) and pop-outs. Most of the cracking in the runway had not been sealed. There was localized severe bleeding and blistering in the paving lane of the runway closest to the terminal as shown in Figure 29. There was no apparent rutting and the groove condition was excellent. The proper evacuation of the water used in the coring operations by the grooving and cross-slope of the pavement is shown in Figure 30. Structurally, the pavement appeared to be performing very well.

Figure 28: Ravelling of Pavement

Figure 29: Bleeding and Blistering in Runway

Figure 30: Proper Evacuation of Core Water by the Pavement Grooving and Cross-Slope

Newark Liberty International Airport

The Newark Liberty International Airport (FAA Designation - EWR) in Newark, New Jersey was visited on May 9, 2007. EWR is large primary hub airport with 435,600 operations in 2004. In 2005, EWR was listed as the thirteenth busiest airport in the United States with 16,444,959 enplanements. The airfield is owned by the Port Authority of New York and New Jersey and has been in existence since 1928 with the first commercial airline terminal in North America opening in 1935 (dedicated by Amelia Earhart). The airport was temporarily closed to passenger traffic as EWR was used as a logistics center by the United States Army during World War II.

The airport is located in the FAA Eastern (AEA) region and is designated in the Wet- Freeze LTPP climatic zone. EWR is at an elevation of 18 feet above sea level. The airport is located in a temperate climate region with an average yearly temperature of 63°F. The average high is in July at 73°F and the average low is in January with a temperature of 28°F. The average precipitation for the year is 43.7 inches. LTPPBind Software indicates that the air temperature during the year ranges from 3°F to 93°F and the pavement temperature ranges from 13°F to 131°F.

The pavement examined at EWR was runway 11/29. Runway 11/29 has a critical pavement rating of 191,000 pounds (single wheel landing gear configuration). The shoulder adjacent to Runway 11/29 was also examined. The shoulder and runway are basically the same mix except that the runway used a PG 76-28 binder and the shoulder used a PG 64-22 binder.

The surface course of Runway 11/29 is 2 to 3 inches of ¾ inch maximum aggregate size HMA utilizing a PG 76-28 asphalt binder. The total depth of asphalt on runway 11/29 ranges from 10 to 14 inches. Both the runway and the shoulder were paved in 1999. The runway appears to be in good structural condition. No rutting was observed, however moderately severe transverse cracking was seen throughout the runway surface as well as raveling with extensive loss of fines, as shown in Figure 31. Figure 31 shows both the runway and shoulder pavements side-by-side. The shoulder has moderately severe transverse cracking also throughout the pavement though not as extensive as in the runway (illustrated in Figure 32). The shoulder is also not experiencing raveling like was observed on the runway. According to Port Authority personnel, the runway began experiencing the loss of fines within one year after construction. Within that year the runway began turning brown and within two years the runway had turned grey while the shoulder was still black. The shoulder pavement did not turn grey until approximately year five. The Port Authority personnel believe this was caused by the binder differences since the mixes are almost exactly the same except for the binder. The runway also has a few large patches that are in good condition and longitudinal joints between the runway and shoulder have been routed and filled. There is no noticeable joint in the runway pavement itself except for the centerline as both sides of the runway were paved in echelon. Both the shoulder and the runway are grooved and the grooves appear to be holding up well in both pavements.

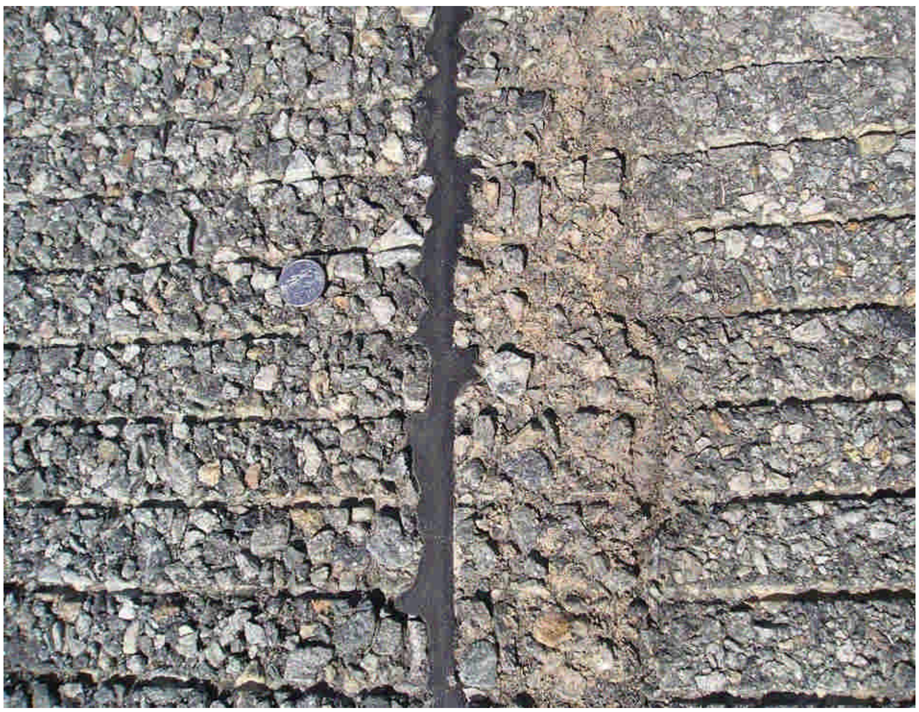

Figure 31: Sealed Construction Joint between Runway (Left) and Shoulder Pavement

Figure 32: Transverse Cracking in Ungrooved Section of Runway 11/29

Palm Springs International Airport

The Palm Springs International Airport (PSP) was visited on March 12, 2007. PSP is considered a small-hub primary airport with 10,287 commercial flights, 1,259 military and 94,578 total operations in 2006. Aircraft operations are highly seasonal with many flights not operating in the summer months. The major commercial carriers in 2006 (over 10,000 passengers) include: Alaska Airlines, American Airlines, American Eagle, Delta, Delta Connection, Continental Express, Harmony Airways, Horizon Air, Northwest, Sun Country, US Airways, United, United Express and WestJet. The Fixed Base Operators (FBO's) operating from this airport include: Signature Flight Support and Atlantic Aviation.

The airport is located in the FAA Western Pacific (AWP) region and is designated in the Dry-No Freeze LTPP climatic zone. PSP is at an elevation of 477 feet above sea level. The airport is located in a desert climate region with an average yearly temperature of 89°F. The average high is in July at 108°F and the average low is in December and

January with a temperature of 42°F. The average rainfall for the year is 5.5 inches. LTPPBind Software indicates that the air temperature during the year ranges from 29°F to 114°F and the pavement temperature ranges from 35°F to 159°F.

The pavement examined at PSP is the 13R/31L runway. The 13R/31L runway was rated by the Federal Aviation Administration (FAA) to carry a maximum load of 600,000 pounds for the DC-10-10 and L-1011 and 700,000 pounds for the DC-10-30.

Runway 13R/31L was rebuilt in 1995 with approximately 5 inches of ¾ inch maximum aggregate size HMA utilizing AR-4000 asphalt binder under 4 inches of ¾ inch maximum aggregate size HMA utilizing a AC-20P asphalt binder topped off with a ¾ inch lift of porous friction course (PFC). Figure 33 illustrates the layers of the Palm Springs pavement. (Note: Cores 1 and 3 broke at layer interfaces after coring. Core 2 was the only full-depth core recovered.) The performance of the pavement appears excellent. A condition survey of the pavement indicated no rutting and no cracking. The only visible distresses included some oxidation of the PFC with occasional raveling and pop-outs. The surface of the PFC pavement is shown in Figure 34.

Figure 33: Cores Taken from Palm Springs Runway 13R/31L

Figure 34: Typical Texture of Palm Springs Runway 13R/31L

Spokane International Airport

The Spokane International Airport (FAA Designation - GEG) in Spokane, Washington was visited on August 1, 2007. GEG is a small primary hub airport with

103,975 operations in 2006. In 2005, GEG was listed as the seventy-second busiest airport in the United States with 1,583,737 enplanements.

The airport is located in the FAA Northwest Mountain (ANM) region and is designated in the Dry - Freeze LTPP climatic zone. GEG is at an elevation of 2,376 feet above sea level. The airport is located in a temperate climate region with an average yearly temperature of 46°F. The average high is in July at 84°F and the average low is in January with a temperature of 22°F. The average precipitation for the year is 16.1 inches. LTPPBind Software indicates that the air temperature during the year ranges from -8°F to 94°F and the pavement temperature ranges from 0°F to 123°F.

The pavement examined at GEG was the main parallel Taxiway A. Taxiway A is parallel to runway 03/21 which is designed for a loading of 400,000 lbs (DTW landing gear configuration).

The surface course of Taxiway A is 2 to 3 inches of 1/2 inch NMAS HMA utilizing 6.3 percent AR 4000W asphalt binder. The total depth of asphalt on Taxiway A is approximately 14 inches. The taxiway appears to be in good structural condition. The taxiway was last paved in 1991; therefore, the pavement age at the time of the site visit was 16 years. No rutting was observed; however, severe transverse cracking, longitudinal joint cracking, pop-outs and raveling with loss of fines were observed. Figure 35 shows an intersection of a longitudinal and transverse crack. Figure 36 illustrates typical raveling of the surface. Some of the cracks have been sealed, but many have not; some of the crack sealing in shown in Figure 37. This pavement is scheduled for rehabilitation in 2008.

Figure 35: Intersection of Longitudinal Crack and Transverse Crack

Figure 36: Typical Raveling

Figure 37: Typical Crack Sealing

Chapter 5
Materials and Test Methods

Mix Design and In-Place Core Information

For each of the ten airfield mixes, the researchers obtained cores from the pavements in order to determine in-place density, asphalt binder content and gradation. Additionally, materials from the original source that were utilized to produce the various HMA mixes were obtained.

The original mix design data and quality control information (if available) was obtained from either an airfield representative or a Civil Engineer that worked on the project. Cores obtained from the various airfield pavements were subjected to a series of tests in order to obtain in-place properties. The top layer, or layer of interest, was cut from each core. Initially, the bulk specific gravity of the core was determined utilizing ASTM D2726 *Bulk Specific Gravity & Density of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens* as well as measuring the height and diameter of each core. The core was then tested for its indirect tensile strength. Once the tensile strength was determined the core was heated and the cut faces were carefully removed from the cores. After the cut faces were removed, the sample was broken down and the theoretical maximum specific gravity was determined using ASTM D2041 *Standard Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures.*

After the material was dried, the binder was extracted using ASTM D 2172 *Standard Test Methods for Quantitative Extraction of Bitumen from Bituminous Paving Mixtures* and recovered using ASTM D 5404 Standard *Practice for Recovery of Asphalt from Solution Using the Rotary Evaporator*. Due to the small sample of binder actually extracted and recovered, only limited binder tests could be run. The binder was tested using AASHTO T 315, *Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR),* ASTM D 4402, *Standard Test Method for Viscosity Determination of Asphalt at Elevated Temperatures Using a Rotational Viscometer* and ASTM D 2171 *Standard Test Method for Viscosity of Asphalts by Vacuum Capillary Viscometer.*

The gradation of the remaining aggregate was determined using ASTM C 117 *Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing* and ASTM C 136 *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*. The aggregate was then tested for flat and elongated particles using ASTM D 4791 *Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate*, crushed faces using ASTM D 5821 *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate*, and fine aggregate angularity using ASTM 1252 *Standard Test Methods for Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading)*. The above described aggregate tests were also conducted on each of the stockpiles obtained from the original sources described above.

Table 16 summarizes the test methods used to evaluate the various materials. Table 17 provides various acronyms that are included within this chapter to provide material properties.

Table 16: Summary of Test and Test Methods

Test Name	Specifying Agency	Test Number
Bulk Specific Gravity of Compacted HMA	ASTM	D 2726
Maximum Specific Gravity of HMA	ASTM	D 2041
Binder Extraction	ASTM	D 2172
Binder Recovery	ASTM	D 5404
Dynamic Shear Rheometer	AASHTO	T 315
Brookfield Viscosity	ASTM	D 4402
Absolute Viscosity	ASTM	D 2171
Washed Gradation	ASTM	C 117, C 136
Flat and Elongated Particles	ASTM	D 4791
Crushed Faces	ASTM	D 5821
Fine Aggregate Angularity	ASTM	D 1252

Table 17: Definitions of Commonly Used Acronyms

Acronym	Definition
AC	Binder Content, %
VTM	Voids Total Mix, % (Air Voids)
VMA	Voids in the Mineral Aggregate, %
VEA	Volume of Effective Asphalt, % (VMA-VTM)
VFA	Voids Filled with Asphalt, %
G_{mm}	Maximum Specific Gravity of HMA Mixture
G_{se}	Effective Specific Gravity of Aggregate
G_{sb}	Bulk Specific Gravity of Aggregate
G_{sa}	Apparent Specific Gravity of Aggregate
G_{mb}	Bulk Specific Gravity of Compacted HMA
N_{ini}	Initial Gyration Level
P_{ba}	Percent of Absorbed Binder
P_{be}	Percent of Effective Binder
UVCA	Uncompacted Voids of Coarse Aggregate, %
FAA	Fine Aggregate Angularity, %
F&E	Flat and Elongated, %
SE	Sand Equivalent

Material Properties

Jacqueline Cochran Regional Airport

The pavement examined at the Jacqueline Cochran Regional Airport was comprised of a ¾ in. maximum aggregate size Marshall designed HMA that was placed in 1997. A granite aggregate was used for the mixture from Granite Construction's Indio Quarry in Indio, CA. The asphalt binder used was an AR-4000 and the mixture was designed with 75 blows per face of the Marshall hammer. Under the Superpave system the nominal maximum aggregate size (NMAS) would be considered 12.5 mm and the AR-4000 would likely grade as a PG 64-10. More information regarding the Jacqueline Cochran Regional Airport is given in Table 18.

The column labeled "JMF" is the data that was taken directly from the job mix formula worksheet supplied by the Granite Construction Company. The column labeled "In-Place" is the properties measured from the core samples taken from the examined pavement at Jacqueline Cochran. The in-place gradation differs significantly from the mix design gradation as does the effective specific gravity of the aggregates. This difference could be caused by aggregate breakdown over the life of the pavement or a mix that was out of specification when it was placed or a JMF that was not representative of the selected HMA layer. The quality control data could not be located for this job, nor could a representative from the contractor or engineer be located that actually worked on this job. The in-place data was used for comparisons in this paper because it is the mixture that has performed in the field.

Table 18: Mix Design and In-Place Data for Jacqueline Cochran Regional Airport – Thermal, CA

	JMF	In-place
Mix Data		
Marshall Blows	75	N/A
Binder Grade	AR-4000	AR-4000
DSR, °C	N/A	86.8
Brookfield, cP	N/A	1,550
Absolute, P	N/A	N/A
Tensile Strength, psi	N/A	398.1
AC, %	5.0	5.2
VTM, %	3.8	5.2
VMA*, %	15.2	14.2
VEA, %	11.4	9.0
VFA, %	75.0	63.3
$\%G_{mm}$ @ N_{ini}	N/A	N/A
G_{mm}	2.440	2.518
G_{se}	2.629	2.735
P_{ba}, %	-0.10	1.41
P_{be}, %	5.1	3.9
Aggregate Data		
UVCA, %	42.3	N/A
Crushed Faces	1/83 2/91	1/100 2/100
Fine Aggregate Angularity, %	43.1	43.2
Flat and Elongated, %	4.1	0
Sand Equivalent	63	N/A
G_{sb}	2.636	N/A
G_{sa}	2.709	N/A
Absorption, %	1.02	N/A
Sieve Size	Gradation, % Passing	
3/4" (19.0 mm)	100	100
1/2" (12.5 mm)	91	90
3/8" (9.5 mm)	76	80
# 4 (4.75 mm)	50	64
# 8 (2.36 mm)	38	53
# 16 (1.18 mm)	28	39
# 30 (600 µm)	20	27
# 50 (300 µm)	13	16
# 100 (150 µm)	8	8
#200 (75 µm)	5.0	4.7

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Mineral County Memorial Airport

The pavement examined at the Mineral County Memorial Airport was a Superpave designed 12.5 mm NMAS mixture that was paved in 2000. The airport was paved at the same time with the same Superpave designed mix as Colorado State Highway 149 that runs adjacent to the airport property. The binder used was a PG 58-34 because of the low temperatures experienced by the area. Additional mix design and in-place information for the Mineral County Memorial Airport can be found in Table 19.

The aggregate was taken from the Spring Creek Pit. Due to the remote location of Creede, Co, an aggregate with a very low bulk specific gravity (2.246) and a high water absorption (5.05 percent) was used. The binder content is relatively high (7.3 percent) in the mix design, but was measured much lower in-place (5.4 percent). While the binder content may be slightly lower during production, the much more likely scenario is that the extraction of the binder from the cores was not complete due to the extremely high absorption level of the aggregate. Experience has shown that the higher the aggregate absorption the more difficult it is to extract the binder from the HMA.

Table 19: Mix Design and In-Place Data for Mineral County Memorial Airport – Creede, CO

	JMF	In-place*
Mix Data		
Superpave Gyrations	76	N/A
Binder Grade	PG 58-34	PG 58-34
DSR, °C	N/A	70.3
Brookfield, cP	N/A	725
Absolute, P	N/A	6,477
Tensile Strength, psi	N/A	121.3
AC, %	7.3	5.4
VTM, %	4.0	5.3
VMA*, %	15.0	13.1
VEA, %	11.0	7.8
VFA, %	73.0	59.5
%G_{mm} @ N_{ini}	N/A	N/A
G_{mm}	2.170	2.179
G_{se}	2.377	2.327
P_{ba}, %	2.53	1.60
P_{be}, %	5.0	3.9
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	2+ /100	2+ /100
Fine Aggregate Angularity, %	46	41.7
Flat and Elongated, %	N/A	0
Sand Equivalent	77	N/A
G_{sb}	2.246	N/A
G_{sa}	2.527	N/A
Absorption, %	5.05	N/A
Sieve Size	Gradation, (% Passing)	
3/4" (19.0 mm)	100	100
1/2" (12.5 mm)	97	98
3/8" (9.5 mm)	82	86
# 4 (4.75 mm)	56	58
# 8 (2.36 mm)	37	37
# 16 (1.18 mm)	26	26
# 30 (600 µm)	18	19
# 50 (300 µm)	12	14
# 100 (150 µm)	7	9
#200 (75 µm)	5.1	6.4

* Average QA data listed for gradation and binder content due to highly absorptive aggregate
N/A - Data not available or not applicable

Oxford-Henderson Airport

The pavement examined at the Oxford-Henderson Airport actually contained two different Superpave mixtures. The two mixtures were identified as 9.5mm A and 9.5mm B. They were both 9.5 mm NMAS Superpave mixtures. The 9.5 mm A mixture was designed at 50 gyrations and the 9.5 mm B HMA was designed at 75 gyrations. The gradations for the two mixtures are very similar, but as expected, the optimum binder content for the 75 gyration HMA is significantly less than the 50 gyration designed HMA. After examination of the layout of the airport, it was determined that most of the aircraft traveled over the 9.5 mm B HMA, so this was chosen as the mixture to be studied.

The binder used in the HMA was a PG 64-22. This binder contained 0.75% liquid anti-strip to combat moisture damage in the HMA. Fifteen percent reclaimed asphalt pavement (RAP) was also used in the mixture. The aggregate was a granite gneiss material from the Greystone Quarry in Henderson, NC owned by Vulcan Materials Company. Table 20 reveals additional mix design and in-place data for the Oxford-Henderson 9.5 mm B mix.

Table 20: Mix Design and In-Place Data for Oxford-Henderson Airport – Henderson, NC

	JMF	In-place
Mix Data		
Superpave Gyrations	75	N/A
Binder Grade	PG 64-22	PG 64-22
DSR, °C	N/A	85.2
Brookfield, cP	N/A	1,600
Absolute, P	N/A	10,988
Tensile Strength, psi	N/A	161.1
AC, %	5.8	5.9
VTM, %	4.0	7.4
VMA*, %	17.1	20.4
VEA, %	13.1	13.0
VFA, %	76.0	63.7
%G_{mm} @ N_{ini}	N/A	N/A
G_{mm}	2.431	2.424
G_{se}	2.650	2.649
P_{ba}, %	-0.04	-0.06
P_{be}, %	5.8	6.0
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	1/100 2/100	1/100 2/100
Fine Aggregate Angularity, %	50.4	45.9
Flat and Elongated, %	1.4	1.3
Sand Equivalent	72.7	N/A
G_{sb}	2.653	N/A
G_{sa}	2.690	N/A
Absorption, %	0.52	N/A
Sieve Size	Gradation, % Passing	
1/2" (12.5 mm)	99	100
3/8" (9.5 mm)	96	96
# 4 (4.75 mm)	74	71
# 8 (2.36 mm)	55	50
# 16 (1.18 mm)	43	38
# 30 (600 µm)	32	29
# 50 (300 µm)	21	21
# 100 (150 µm)	11	12
#200 (75 µm)	5.6	6.0

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Military Airfields

Little Rock Air Force Base

The HMA on the assault strip at the Little Rock Air Force Base was designed using 139 gyrations of the Superpave gyratory compactor. The pavement was placed in 1998 and the 139 gyrations matched a specification of 30 to 100 million equivalent single axle loads (ESALs) at that time.

The HMA was a 12.5 mm NMAS mixture made of sandstone from the Cabot Quarry in Cabot, AR currently owned by the Rogers Group and granite from the Granite Mountain Quarries in Little Rock, AR. The binder used in this HMA was PG 70-22. Additional mix design information is shown in Table 21. The data in Table 21 was taken from the mix design information supplied by the 314th Civil Engineering Squadron based at the Little Rock Air Force Base and from the in-place core testing.

Table 21: Mix Design and In-Place Data for Little Rock Air Force Base – Jacksonville, AR

	JMF	In-place
Mix Data		
Superpave Gyrations	139	N/A
Binder Grade	PG 70-22	PG 70-22
DSR, °C	N/A	83.6
Brookfield, cP	N/A	1,750
Absolute, P	N/A	35,481
Tensile Strength, psi	N/A	118.5
AC, %	5.8	5.2
VTM, %	4.0	4.4
VMA*, %	15.1	14.7
VEA, %	11.1	10.3
VFA, %	73.7	70.1
$\%G_{mm}$ @ N_{ini}	84.5	N/A
G_{mm}	2.411	2.414
G_{se}	2.628	2.606
P_{ba}, %	0.95	0.62
P_{be}, %	4.9	4.6
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	1/100 2/100	1/100 2/100
Fine Aggregate Angularity, %	47	43
Flat and Elongated, %	3	0.3
Sand Equivalent	65	N/A
G_{sb}	2.566	N/A
G_{sa}	2.662	N/A
Absorption, %	2.02	N/A
Sieve Size	Gradation, (% Passing)	
3/4" (19.0 mm)	100	100
1/2" (12.5 mm)	94	93
3/8" (9.5 mm)	76	76
# 4 (4.75 mm)	41	48
# 8 (2.36 mm)	28	32
# 16 (1.18 mm)	19	22
# 30 (600 µm)	14	17
# 50 (300 µm)	10	13
# 100 (150 µm)	7	9
#200 (75 µm)	3.5	5.7

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Naval Air Station-Oceana

The HMA on the examined runway at Naval Air Station-Oceana was placed in the year 2000. The HMA is a ¾ in. maximum aggregate size mixture. Under the Superpave system the mixture would also be classified as a 19.0 mm NMAS. The mixture was designed with 75 blows from a Marshall hammer and used a PG 70-22 binder. The granite aggregate used at NAS Oceana was supplied by the Vulcan Materials Quarry in Skippers, VA. Twenty percent RAP and twenty percent natural sand were also utilized in this mixture.

More information regarding the NAS-Oceana HMA can be found in Table 22. The data in Table 22 was taken from the information supplied from the mix design by Asphalt Roads and Material Co, Inc. and from the in-place core testing.

Table 22: Mix Design and In-Place Data for Naval Air Station Oceana

Naval Air Station Oceana - Virginia Beach, VA

	JMF	In-place
Mix Data		
Marshall Blows	75	N/A
Binder Grade	PG 70-22	PG 70-22
DSR, °C	N/A	77.9
Brookfield, cP	N/A	720
Absolute, P	N/A	8614
Tensile Strength, psi	N/A	191.4
AC, %	6.6	5.5
VTM, %	3.7	5.3
VMA*, %	17.4	17.3
VEA, %	13.7	12.0
VFA, %	78.2	69.3
%G_{mm} @ N_{ini}	N/A	N/A
G_{mm}	2.431	2.451
G_{se}	2.689	2.665
P_{ba}, %	0.53	0.19
P_{be}, %	6.1	5.3
Aggregate Data		
Crushed Faces	1/96 2/92	1/100 2/100
Fine Aggregate Angularity, %	N/A	45.7
Flat and Elongated, %	0	0
Sand Equivalent	N/A	N/A
G_{sb}	2.652	N/A
G_{sa}	N/A	N/A
Absorption, %	N/A	N/A
Sieve Size	Gradation, % Passing	
3/4" (19.0 mm)	100	97
1/2" (12.5 mm)	87	90
3/8" (9.5 mm)	83	85
# 4 (4.75 mm)	74	70
# 8 (2.36 mm)	59	56
# 16 (1.18 mm)	45	43
# 30 (600 µm)	31	28
# 50 (300 µm)	19	18
# 100 (150 µm)	10	11
#200 (75 µm)	5.5	6.7

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Volk Field

The Volk Field mixture is a ¾ in. maximum aggregate size mixture or 12.5 mm NMAS in the Superpave system. The HMA was designed to meet the specified criteria of a 109 gyration Superpave mixture; however, a mirror 75 blow Marshall mixture was also designed at the same binder content. The only difference in the two mixtures was the compactive equipment, effort and the resulting lab volumetrics.

This was the first Superpave mixture placed on an airfield in the state of Wisconsin. Performing both the Marshall and the Superpave designs gave a level of confidence in the performance of the mixture. The acceptable range of VTM used for the Marshall designed mixtures was 3 to 4 percent. For the Superpave mixtures the VTM range was 2.5 to 3.5 percent.

The binder used for this mixture was a PG 64-28. Limestone aggregate from the Moser quarry was used with fifteen percent blend sand from the Frozene Pit. More information on the mixtures at Volk field can be found in Table 23. The data in Table 23 was taken from the mix design information supplied by Mathy Construction Company and from the in-place core testing.

Table 23: Mix Design and In-Place Data for Volk Field – Camp Douglas, WI

	JMF/Superpave	JMF/Marshall	In-place
Mix Data			
Compaction Level	109	75	N/A
Binder Grade	PG 64-28	PG 64-28	PG 64-28
DSR, °C	N/A	N/A	79.3
Brookfield, cP	N/A	N/A	1,350
Absolute, P	N/A	N/A	20,714
Tensile Strength, psi	N/A	N/A	178.4
AC, %	5.5	5.5	5.23
VTM, %	2.9	3.7	4.7
VMA*, %	14.2	14.9	15.4
VEA, %	11.3	11.2	10.7
VFA, %	80.7	75.2	69.5
$\%G_{mm}$ @ N_{ini}	90.3	90.3	N/A
G_{mm}	2.509	2.509	2.505
G_{se}	2.738	2.738	2.720
P_{ba}, %	0.88	0.88	0.64
P_{be}, %	4.7	4.7	4.6
Aggregate Data			
UVCA, %	N/A	N/A	N/A
Crushed Faces	100	100	100
Fine Aggregate Angularity, %	45.3	45.3	42.3
Flat and Elongated, %	2.7	2.7	0
Sand Equivalent	82	82	N/A
G_{sb}	2.675	2.675	N/A
G_{sa}	N/A	N/A	N/A
Absorption, %	N/A	N/A	N/A
Sieve Size	Gradation, %Passing		
3/4" (19.0 mm)	100	100	100
1/2" (12.5 mm)	95	95	96
3/8" (9.5 mm)	84	84	86
# 4 (4.75 mm)	64	64	70
# 8 (2.36 mm)	46	46	52
# 16 (1.18 mm)	35	35	38
# 30 (600 µm)	28	28	30
# 50 (300 µm)	18	18	20
# 100 (150 µm)	8	8	11
#200 (75 µm)	5.1	5.1	6.0

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Jackson-Evers International Airport

The pavement examined at the Jackson International Airport was a 19.0 mm maximum aggregate size mixture designed with 75 blows from the Marshall hammer. The pavement was placed in 1996. The binder used was a viscosity graded AC-30. The aggregate was eighty percent limestone and twenty percent natural sand.

Some difficulty was encountered when trying to match the in-place gradation to the mix design gradation. To the best of the knowledge of everyone involved in the paving project that could be located the correct mix design was obtained. The in-place VMA and VTM are all a little low with the main difference being the bulk specific gravity of the aggregate. Given these differences it was decided to utilize the gradation of the in-place samples. This data and more can be found in Table 24. The data in Table 24 was taken from the mix design information supplied by Superior Asphalt Inc. and from the in-place core testing.

Table 24: Mix Design and In-Place Data for Jackson International Airport – Jackson, MS

	JMF	In-place
Mix Data		
Marshall Blows	75	N/A
Binder Grade	AC-30	AC-30
DSR, °C	N/A	75.18
Brookfield, cP	N/A	735
Absolute, P	N/A	N/A
Tensile Strength, psi	N/A	114.2
AC, %	5.4	5.3
VTM, %	3.5	2.7
VMA*, %	15.2	13.0
VEA, %	11.7	10.3
VFA, %	77.0	79.3
%G_{mm} @ N_{ini}	N/A	N/A
G_{mm}	2.471	2.459
G_{se}	2.685	2.666
P_{ba}, %	0.32	0.04
P_{be}, %	5.1	5.3
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	1/100 2/100	1/100 2/100
Fine Aggregate Angularity, %	N/A	40.3
Flat and Elongated, %	<1	0
Sand Equivalent	73	N/A
G_{sb}	2.663	2.605
G_{sa}	2.722	2.635
Absorption, %	0.81	1.15
Sieve Size	Gradation, % Passing	
3/4" (19.0 mm)	100	100
1/2" (12.5 mm)	95	96
3/8" (9.5 mm)	81	85
# 4 (4.75 mm)	53	57
# 8 (2.36 mm)	37	44
# 16 (1.18 mm)	30	38
# 30 (600 µm)	23	31
# 50 (300 µm)	11	16
# 100 (150 µm)	8	9
#200 (75 µm)	5.4	6.4

*In-Place VMA calculated using In-Place G_{sb}

N/A - Data not available or not applicable

Newark Liberty International Airport

The shoulder pavement examined at the Newark Liberty International Airport was a ¾ in. maximum aggregate size mixture designed with 75 blows from the Marshall hammer. Under the Superpave nomenclature, the mixture would be a 12.5 mm NMAS. The runway and shoulder were paved in 1999. The mixture on the shoulder used a PG 64-22 binder and granite gneiss aggregate.

Additional mix design information can be found in Table 25. The data in Table 25 was taken from the mix design information supplied by The Port Authority of New York and New Jersey and from the in-place core testing.

Table 25: Mix Design and In-Place Data for Newark Liberty International Airport – Newark, NJ

	JMF	In-place
Mix Data		
Marshall Blows	75	N/A
Binder Grade	PG 64-22	PG 64-22
DSR, °C	N/A	78.12
Brookfield, cP	N/A	945
Absolute, P	N/A	19,101
Tensile Strength, psi	N/A	75.0
AC, %	5.1	4.8
VTM, %	4.0	4.4
VMA*, %	16.3	15.9
VEA, %	12.3	11.5
VFA, %	75.7	72.2
$\%G_{mm}$ @ N_{ini}	N/A	N/A
G_{mm}	2.523	2.526
G_{se}	2.742	2.726
P_{ba}, %	0.14	-0.09
P_{be}, %	5.0	4.9
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	2+ /100	2+ /100
Fine Aggregate Angularity, %	N/A	45.1
Flat and Elongated, %	N/A	0.9
Sand Equivalent	N/A	N/A
G_{sb}	2.732	N/A
G_{sa}	N/A	N/A
Absorption, %	N/A	N/A
Sieve Size	Gradation, % Passing	
3/4" (19.0 mm)	99	100
1/2" (12.5 mm)	92	95
3/8" (9.5 mm)	79	86
# 4 (4.75 mm)	40	45
# 8 (2.36 mm)	28	31
# 16 (1.18 mm)	22	23
# 30 (600 µm)	15	17
# 50 (300 µm)	9	12
# 100 (150 µm)	5	7
#200 (75 µm)	2.5	4.2

*In-Place VMA calculated using JMF G_{sb}

JMF G_{mm}, G_{se} and G_{sb} are from quality control data

N/A - Data not available or not applicable

Palm Springs International Airport

The HMA layer examined at the Palm Springs International Airport was a 25.0 mm maximum aggregate size mixture designed with 75 blows from the Marshall hammer. This layer was topped with a ¾ inch layer of permeable friction course HMA. A granite aggregate was used for the mixture from Granite Construction's Indio Quarry in Indio, CA. The asphalt binder used was an AC-20P which was used because of previous experience with the binder. The AC-20P is comprised of an AC-7.5 that is modified to meet the criteria of an AC-20. Under the Superpave system the NMAS would be considered a 19.0 mm NMAS and the AC-20P would probably grade as a PG 58-XX. It is interesting that the binder would likely grade so low given the average temperature at the airport is 89°F and LTPPBind would recommend a minimum PG graded asphalt of 76-XX. Intuitively, one would think that the lower grade of binder would cause significant rutting; however, this is not the case. The airport engineer at the time of paving indicated that the binder was chosen because of previous good experience. No rutting was noted during the visit to the airport. This AC-20P asphalt was chosen because of past success on airfield using this binder. Most of the traffic at the Palm Springs International Airport occurs during the winter and spring months at the height of the tourist season for the area. More information regarding the Palm Springs International Airport is given in Table 26.

The column labeled "JMF" in Tables 26 is data that was taken directly from the job mix formula worksheet supplied by the Granite Construction Company. The column labeled in-place is the properties measure from the core samples taken from the examined pavement at Palm Springs. The in-place data was used for comparisons in this paper because it is the mixture that has performed in the field.

Table 26: Mix Design and In-Place Data for Palm Springs International Airport – Palm Springs, CA

	JMF	In-place
Mix Data		
Marshall Blows	75	N/A
Binder Grade	AC-20P	AC-20P
DSR, °C	N/A	N/A
Brookfield, cP	N/A	N/A
Absolute, P	N/A	N/A
Tensile Strength, psi	N/A	193.3
AC, %	5.5	5.4
VTM, %	3.0	1.97
VMA*, %	14	13.9
VEA, %	11.0	12.0
VFA, %	78.6	85.9
$\%G_{mm}$ @ N_{ini}	N/A	N/A
G_{mm}	2.461	2.446
G_{se}	2.678	2.654
P_{ba}, %	0.61	0.27
P_{be}, %	4.9	5.1
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	N/A	2+ /100
Fine Aggregate Angularity, %	N/A	44.6
Flat and Elongated, %	N/A	0.6
Sand Equivalent	N/A	N/A
G_{sb}	2.636	N/A
G_{sa}	2.67	N/A
Absorption, %	1.10	N/A
Sieve Size	Gradation, (% Passing)	
1" (25.0 mm)	100	100
3/4" (19.0 mm)	90	98
1/2" (12.5 mm)	73	85
3/8" (9.5 mm)	69	77
# 4 (4.75 mm)	53	61
# 8 (2.36 mm)	40	50
# 16 (1.18 mm)	32	38
# 30 (600 µm)	21	26
# 50 (300 µm)	14	17
# 100 (150 µm)	9	10
#200 (75 µm)	4.9	5.7

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Spokane International Airport

The main parallel taxiway (Taxiway Alpha) was examined at the Spokane International Airport. The mixture used was a ¾ in. maximum aggregate size or 12.5 mm NMAS mixture. The HMA was designed with 75 blows from a Marshall hammer. The aggregate was obtained from Pit C-318 and included 13 percent natural sand. The binder used was an AR-4000W supplied by Koch Asphalt from the Spokane, Washington plant. This would likely grade as a PG 64-22 binder under the Superpave system.

More information regarding the Spokane International Airport is given in Table 27. The column labeled “JMF” is the data that was taken directly from the job mix formula worksheet supplied by Inland Asphalt Company. The column labeled in-place is the properties measure from the core samples taken from the examined pavement at Spokane.

Table 27: Mix Design and In-Place Data for Spokane International Airport - Spokane, WA

	JMF	In-place
Mix Data		
Marshall Blows	75	N/A
Binder Grade	AR-4000W	AR-4000W
DSR, °C	N/A	75.1
Brookfield, cP	N/A	695
Absolute, P	N/A	2,910
Tensile Strength, psi	N/A	210.3
AC, %	6.3	5.3
VTM, %	4.0	1.9
VMA*, %	16.0	12.3
VEA, %	12.0	10.4
VFA, %	78	84.6
%G_{mm} @ N_{ini}	N/A	N/A
G_{mm}**	2.524	2.526
G_{se}	2.797	2.749
P_{ba}, %	1.65	1.01
P_{be}, %	4.8	4.3
Aggregate Data		
UVCA, %	N/A	N/A
Crushed Faces	1/96 2/92	1/100 2/100
Fine Aggregate Angularity, %	N/A	45.5
Flat and Elongated, %	3	0
Sand Equivalent	N/A	N/A
G_{sb}	2.677	N/A
G_{sa}	2.870	N/A
Absorption, %	2.51	N/A
Sieve Size	Gradation, % Passing	
3/4" (19.0 mm)	100.0	100
1/2" (12.5 mm)	95.2	97
3/8" (9.5 mm)	85.2	89
# 4 (4.75 mm)	57.1	62
# 8 (2.36 mm)	40.7	46
# 16 (1.18 mm)	27.6	31
# 30 (600 µm)	16.8	20
# 50 (300 µm)	12.2	16
# 100 (150 µm)	8.0	11
#200 (75 µm)	6.0	8.8

*In-Place VMA calculated using JMF G_{sb}

N/A - Data not available or not applicable

Ancillary Mixtures

Four other airfield mixtures were utilized for various purposes within the research project. The following sections briefly describe these mixes.

John Bell Williams Airport

John Bell Williams was a 75 blow Marshall design HMA mixture that was used as a shadow specification project. It was a 19.0 mm NMAS mixture utilizing 71 percent crushed gravel, 11 percent natural sand and 18 percent limestone. Mix design information for John Bell Williams is shown in Table 28.

Mid-Delta Regional Airport

The 75 blow Marshall designed HMA used at Mid-Delta Regional Airport in Greenville, MS was used to examine permeability of HMA. This mixture was a 12.5 mm NMAS that was comprised of 57 percent crushed gravel, 32 percent limestone, 10 percent natural sand and 1 percent hydrated lime. Mix design information for the Mid-Delta Regional Airport is shown in Table 29.

Portland International Airport

The 100 gyration Superpave designed mixture placed at the Portland International Airport was utilized as a shadow specification project. The mixture was a 19.0 mm NMAS mixture. Mix design information for the Portland International Airport is shown in Table 30.

Table 28: Mix Design Data for John Bell Williams Airport – Bolton, MS

	JMF
Mix Data	
Marshall Blows	75
Binder Grade	PG 67-22
AC, %	5.4
VTM, %	3.6
VMA, %	15.2
VEA, %	11.6
VFA, %	76.3
G_{mm}	2.366
G_{se}	2.555
P_{ba}, %	0.16
P_{be}, %	5.2
Aggregate Data	
Crushed Faces	2+ /100
Fine Aggregate Angularity, %	46.4
Flat and Elongated, %	<1
Sand Equivalent	58
G_{sb}	2.545
G_{sa}	2.648
Absorption, %	1.53
Sieve Size	Gradation, % Passing
3/4" (19.0 mm)	100
1/2" (12.5 mm)	87
3/8" (9.5 mm)	76
# 4 (4.75 mm)	52
# 8 (2.36 mm)	34
# 16 (1.18 mm)	23
# 30 (600 µm)	17
# 50 (300 µm)	10
# 100 (150 µm)	6
#200 (75 µm)	4.3

Table 29: Mix Design Data for Mid-Delta Regional Airport – Greenville, MS

	JMF
Mix Data	
Marshall Blows	75
Binder Grade	AC 30
AC, %	5.4
VTM, %	3.5
VMA, %	15.4
VEA, %	11.9
VFA, %	77.0
G_{mm}	2.397
G_{se}	2.593
P_{ba}, %	0.15
P_{be}, %	5.3
Aggregate Data	
Crushed Faces	1/100 2/98
Fine Aggregate Angularity, %	
Flat and Elongated, %	0.3
Sand Equivalent	76
G_{sb}	2.584
G_{sa}	2.656
Absorption, %	1.05
Sieve Size	**Gradation, % Passing**
3/4" (19.0 mm)	100
1/2" (12.5 mm)	92
3/8" (9.5 mm)	83
# 4 (4.75 mm)	57
# 8 (2.36 mm)	38
# 16 (1.18 mm)	27
# 30 (600 µm)	18
# 50 (300 µm)	10
# 100 (150 µm)	7
#200 (75 µm)	5.6

Table 30: Mix Design Data for Portland International Airport – Portland, OR

	JMF
Mix Data	
Superpave Gyrations	100
Binder Grade	PG 70-22
AC, %	5.3
VTM, %	4.0
VMA, %	13.7
VEA, %	9.7
VFA, %	70.8
$\%G_{mm}$ @ N_{ini}	86.0
G_{mm}	2.510
G_{se}	2.729
P_{ba}, %	1.19
P_{be}, %	4.2
Aggregate Data	
Crushed Faces	2+ /100
Fine Aggregate Angularity, %	46.4
Flat and Elongated, %	<1
Sand Equivalent	58
G_{sb}	2.646
G_{sa}	2.732
Absorption, %	1.19
Sieve Size	Gradation, % Passing
1" (25.0 mm)	100
3/4" (19.0 mm)	96
1/2" (12.5 mm)	79
3/8" (9.5 mm)	67
1/4" (6.4 mm)	53
# 4 (4.75 mm)	44
# 8 (2.36 mm)	28
#10 (2.00 mm)	24
# 16 (1.18 mm)	19
# 30 (600 µm)	14
#40 (420 µm)	11
# 50 (300 µm)	10
# 100 (150 µm)	8
#200 (75 µm)	5.9

CHAPTER 6
Test Results and Analysis

As stated in the objectives, the purpose of this research project was to adapt the Superpave gyratory compactor procedures for use on airfields. This Chapter discusses test results and analyses conducted in order to accomplish the project objectives.

As discussed in Chapter 3, the Superpave mix design system includes four primary steps: 1) materials selection; 2) selection of design aggregate gradations; 3) selection of optimum asphalt binder content; and 4) performance testing. Each of these steps is equally important to the overall mix design system; however, likely the most critical parameter needed to successfully implement a Superpave mix design system for airfield HMA pavements is the design compactive effort. The design compactive effort, or design number of gyrations, will have a direct impact on the volumetric properties of the designed mix. The volumetric properties, combined with the design compactive effort, will be related to the materials allowed within the mix (materials selection). All of these factors will have an effect on selection of a mix with acceptable properties. The following sections describe the test results and analyses conducted to develop a Superpave mix design system for airfield HMA.

SELECTION OF DESIGN COMPACTIVE EFFORT

With the data collected during this project, the researchers investigated three different methods for selecting the appropriate design compactive effort with the Superpave gyratory compactor. The first method entailed evaluating in-place densities in a manner utilized by researchers during the development of the Marshall mix design procedure for airfields (1). The second method involved comparisons in the bulk specific gravity values of HMA mixes compacted utilizing both the Marshall hammer and Superpave gyratory compactor. The final method for determining the appropriate design compactive effort when using the Superpave gyratory compactor entailed evaluating the results of performance testing. The goal of the performance testing was to determine the asphalt binder content at which the airfield mixes began to exhibit permanent deformation tendencies.

During the original research that selected the Marshall method of mix design for airfields, one criterion utilized in the selection of 50 blows per face as the design compactive effort was the density of the pavement after traffic. The following describes one of the philosophies for selecting the design laboratory compactive effort during the original Marshall research conducted for airfield HMA design (1):

> *"The original procedure for compacting laboratory specimens for the Marshall stability test … had produced densities that were comparable to those obtained in the construction of pavements on the airfields investigated … However, during tracking operations on the asphalt test section, it became apparent, from results of tests on cored samples, that compaction of the pavement was taking place under the traffic applied. The increased density of the pavement also reduced the void space and allowed the asphalt to fill the available voids completely. In the event the pavements contained sufficient asphalt to overfill the voids when the density is increased by traffic, the pavements would flush and may become unstable … it was decided to investigate the possibility of increasing the (laboratory) compactive effort in the Marshall test to obtain densities approximating those attained under a moderate amount of traffic, such that the*

lower optimum asphalt content would provide a pavement which would not have excess asphalt."

A simple method of evaluating the various design compactive efforts utilized for the HMA at the ten airfields would be to evaluate the ultimate densities at the time of the field investigations, similar to that described above. The term "ultimate density" indicates the in-place density of the pavement after years of trafficking. In-place densities that remained low would indicate that the in-place asphalt binder content was too low, while very high in-place densities would indicate that the in-place asphalt binder content was too high.

In order to evaluate the data from the various airfields, the in-place density was determined for each core that was obtained. In-place density was determined by first determining the bulk specific gravity of each core. Next, the respective cores from each airfield were combined and used to determine the theoretical maximum specific gravity. Finally, the bulk specific gravity of each individual core and the combined theoretical maximum specific gravity for the airfield were used to calculate a percent theoretical maximum specific gravity (or percent density) of each core.

In order to present this data, the in-place densities were divided into two data sets: one for airfields using HMA designed using the Marshall hammer and one for airfields using HMA designed with the SGC. These two data sets are illustrated in Figures 38 and 39, respectively. Also included on these figures are the respective design compactive efforts, age of HMA evaluated and performance at the time of our field evaluation.

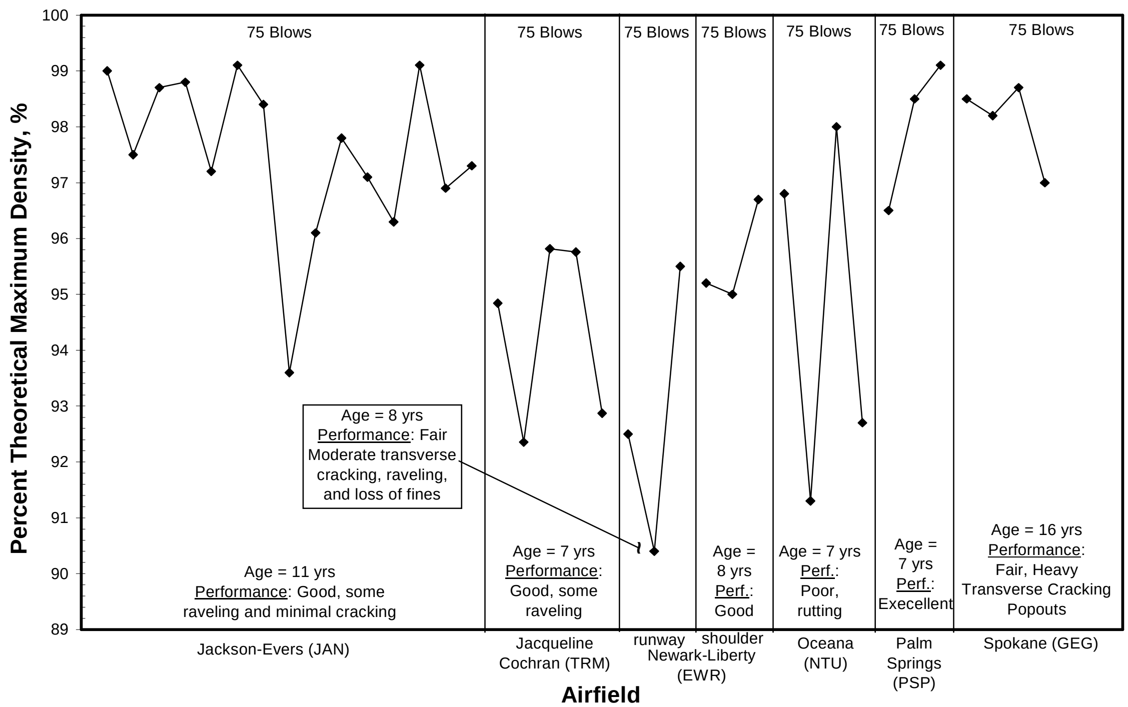

Figure 38: Ultimate Densities of Airfield Pavements Designed Using Marshall Hammer

Figure 38 illustrates the in-place densities of the seven airfield pavements investigated that were designed using the Marshall hammer. Note that two pavements were cored at Newark-Liberty (EWR): one being a runway and the other a shoulder. Layer ages for the HMA mixes shown in Figure 38 ranged from 7 to 16 years with the average age of the airfields being 9.3 years. Based upon the core data, the average in-place density of all the cores shown in Figure 38 was 96.4 percent of theoretical maximum specific gravity. As shown in the figure, significantly more cores were obtained from JAN. The additional cores were obtained to validate that the JMF identified for the test layer was the proper JMF. If all the JAN cores except the initial four cores are removed from the data set, the average in-place density of 96.4 percent of theoretical maximum specific gravity did not change.

The performance of the different HMA layers ranged from poor to good. Of the seven pavements, four were categorized as performing good or excellent. For these four pavements, the average in-place density of the pavement was 96.8 percent of theoretical maximum specific gravity. This in-place density corresponds to an average in-place air void content of 3.2 percent which is representative of a typical design air void content when conducting mix designs in accordance with the Marshall method for airfields. This would indicate that the 75 blows per face were appropriate for these pavements.

Figure 38 also shows that three pavements were categorized as having poor or fair performance. The average in-place density for these three pavements was 95.4 percent of theoretical maximum specific gravity. However, of the three categorized as having poor or fair performance, the HMA pavement at Spokane (GEG) was 16 years old. Being this old it is obvious that the pavement had adequately performed its intended purpose. If this pavement is removed from the poor/fair data set, the average in-place density of the remaining two pavements would be 93.9 percent of theoretical maximum density. This in-place density constitutes an average in-place air void content of 6.1 percent. Of the two fair/poor airfield pavements (runway at Newark-Liberty and Oceana NAS), the modes of distress were different. At Newark-Liberty (EWR), the primary mode of distress was cracking and raveling while at Oceana NAS (NTU) the primary mode of distress was rutting. It is not known whether the poor/fair performance of these two pavements is related to materials selection, mix design, and/or construction; however, the in-place density data does suggest a clear difference between the pavements that have performed and those that have not. The average in-place density for those pavements that are performing good was 96.8 percent while those that performed poor/fair was 93.8

percent. More importantly, the in-place density for the good performing HMA layers suggests that the Item P-401 and UFGS-32 12 15 specifications for design compactive effort and design air void contents are appropriate if the intention is to select an asphalt binder content that will result in ultimate in-place densities similar to the laboratory design density.

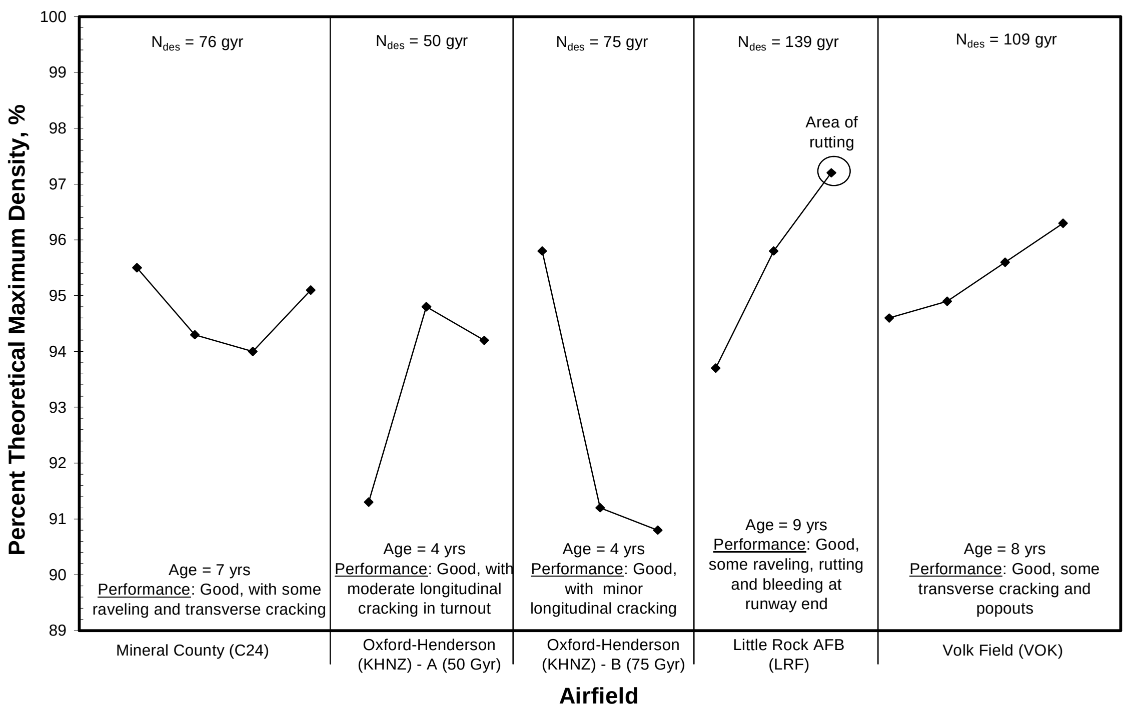

Figure 39: Ultimate Densities of Airfield Pavements Designed with the Superpave Gyratory Compactor

Figure 39 illustrates the in-place densities of the five airfield pavements investigated that were designed using the Superpave gyratory compactor. Note that two pavements were sampled at Oxford-Henderson (KHNZ). One pavement included HMA designed using 50 gyrations while the HMA for the other pavement was designed using 75 gyrations of the Superpave gyratory compactor. Layer ages ranged from 4 to 9 years with an average age of 7 years. Performance was categorized as good for each of the five pavement layers. The average in-place density for all of the cores obtained from the layers was 94.4 percent of theoretical maximum specific gravity, or 5.6 percent air voids. This average air void content is 2 percent higher than the overall average observed for the Marshall hammer designed HMA layers. Only one of the seventeen cores depicted in Figure 39 had an in-place density equal to or above the 96.4 percent of theoretical maximum specific gravity average found in the cores obtained in pavements designed using the Marshall hammer. Though different design gyration levels were utilized for the different HMA mixes, the data suggests that the design compactive efforts utilized were too high.

The above discussion on ultimate densities reflects a very small subset of data. Because of the small subset of data, no specific conclusions will be made; however, a couple of general observations are noted. Based upon the HMA layers designed with the Marshall hammer, the 75 blows per face compactive effort combined with the current design air void contents used in the Marshall mix design method, appears to accurately reflect the ultimate design density of airfield pavements. However, the design gyration levels and 4 percent design air voids used for the Superpave designed mixes did not accurately reflect the ultimate densities within the airfield pavements. These general observations suggest that the Superpave designed mixes were designed at too low of an asphalt binder content.

The second method of evaluating the proper design compactive effort with the SGC entailed comparing densities obtained from compacting samples with the Marshall hammer and Superpave gyratory compactor. Materials from each of the airfields were used to compact HMA utilizing both the Marshall hammer and Superpave gyratory compactor. Mix was compacted using 75 blows per face for all airfield mixes and 50 blows per face for six airfield mixes. Likewise, mix was also compacted using the Superpave gyratory compactor at various gyration levels for all mixes. In order to compare densities from the various compactive efforts, the laboratory density values were normalized by dividing the resulting Marshall density by the density of a companion Superpave gyratory compactor sample (at the same asphalt binder content). This resulted in a ratio of bulk specific gravities for individual mixes. A bulk specific gravity (G_{mb}) ratio of 1.0 indicates that the laboratory density resulting from the two compaction methods were equal.

Figures 40 through 49 present the comparisons in laboratory density between samples compacted with the Marshall hammer and Superpave gyratory compactor for the ten airfields. Within each of these figures, the x-axis presents the number of gyrations used to compact the specimens while the y-axis shows the ratio of the Marshall hammer bulk specific gravity to the Superpave gyratory compactor bulk specific gravity at similar asphalt binder contents. If available, the G_{mb} ratio for both 50 blow and 75 blow samples are provided. Also included on each of these figures is a best-fit line for the data. This best-fit line was used to determine the average gyration level that corresponded to the density of the Marshall compacted samples (ratio of 1.0).

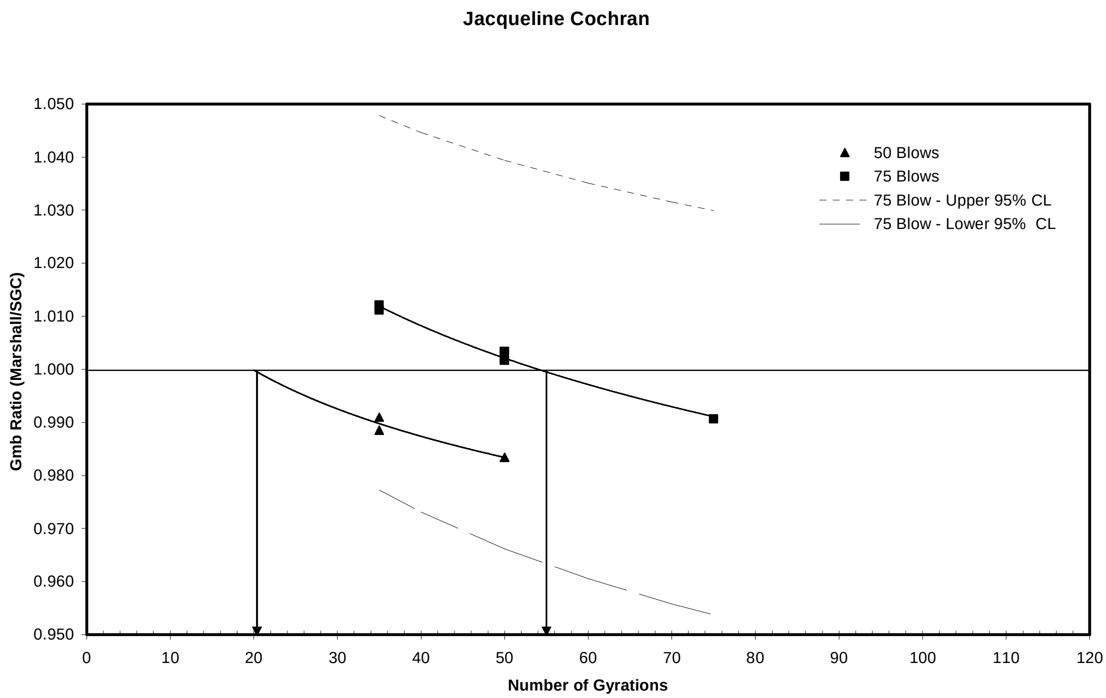

Figure 40: Comparison of Marshall Hammer and Superpave Gyratory Compactor – TRM

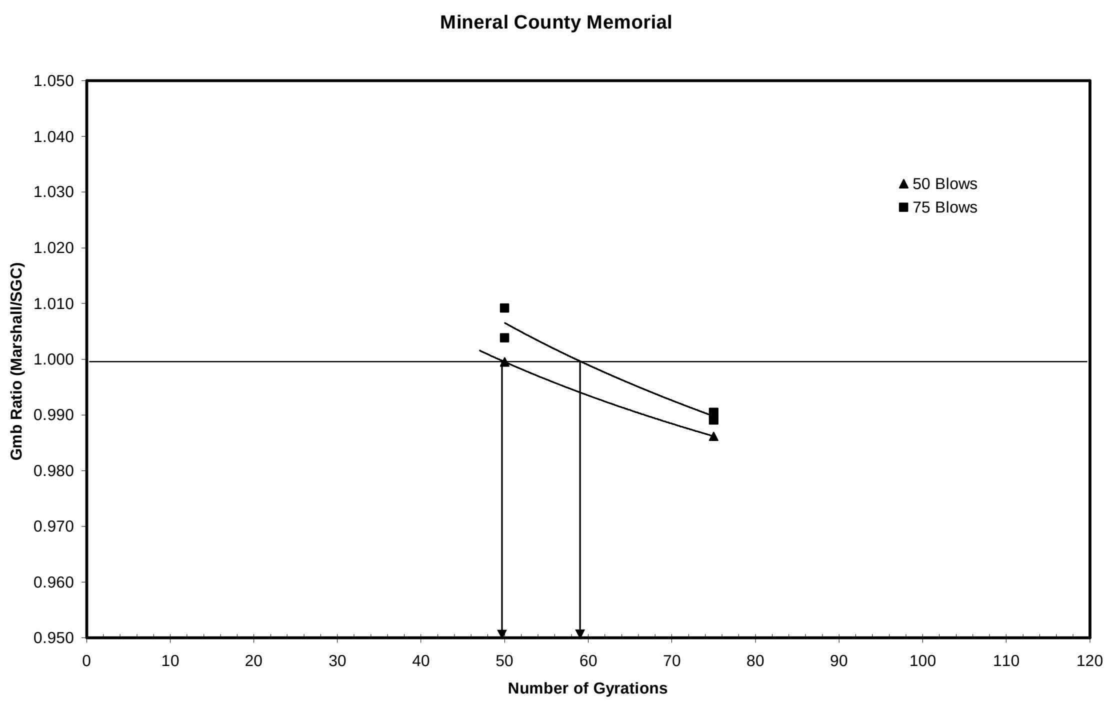

Figure 41: Comparison of Marshall Hammer and Superpave Gyratory Compactor - C24

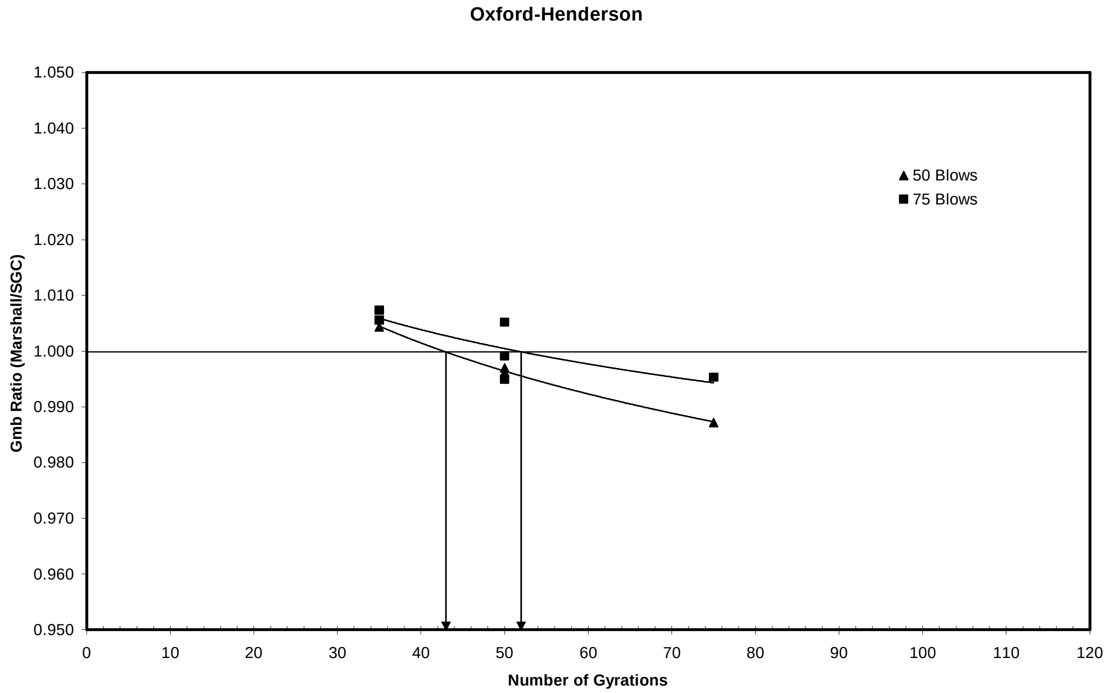

Figure 42: Comparison of Marshall Hammer and Superpave Gyratory Compactor - (KHNZ)

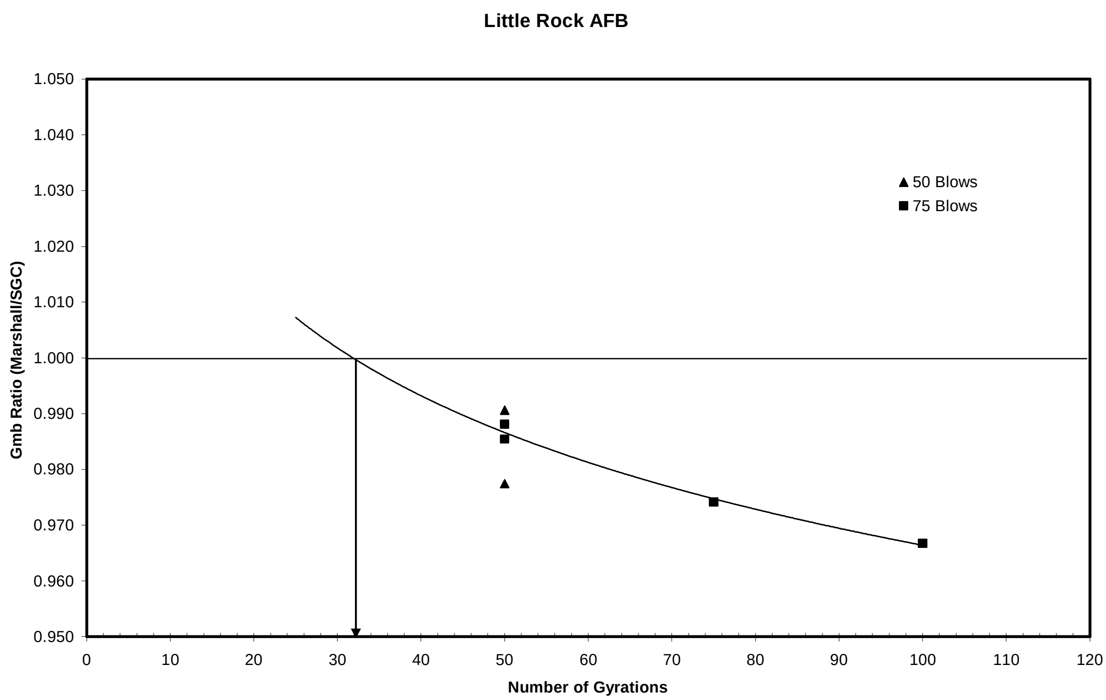

Figure 43: Comparison of Marshall Hammer and Superpave Gyratory Compactor – LRF

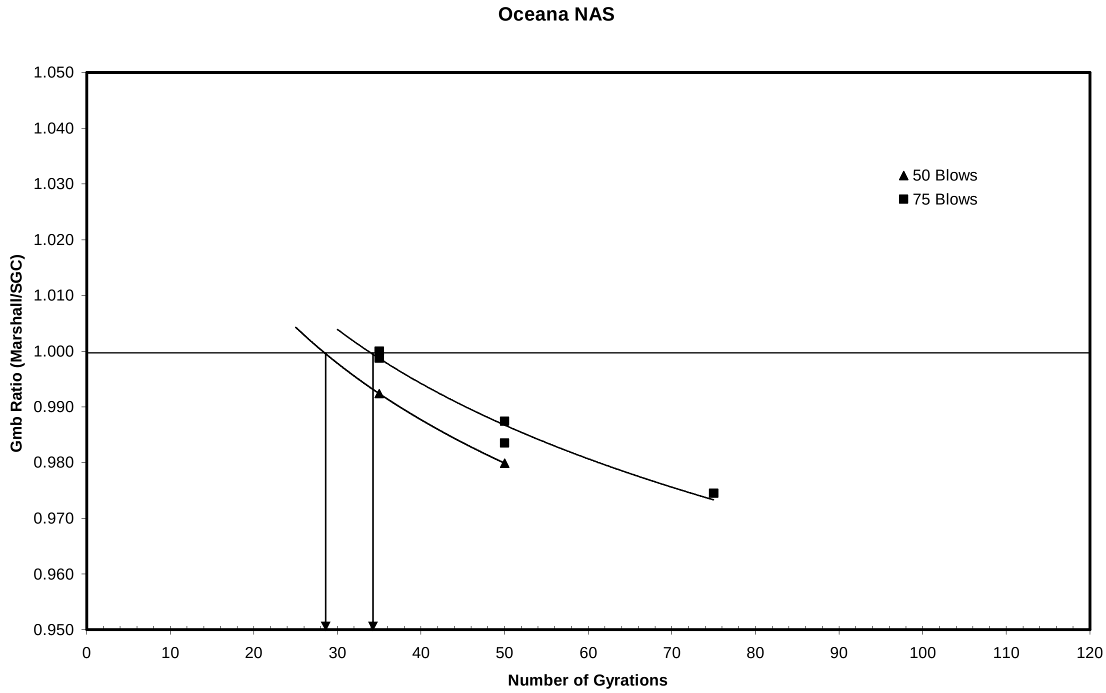

Figure 44: Comparison of Marshall Hammer and Superpave Gyratory Compactor – NTU

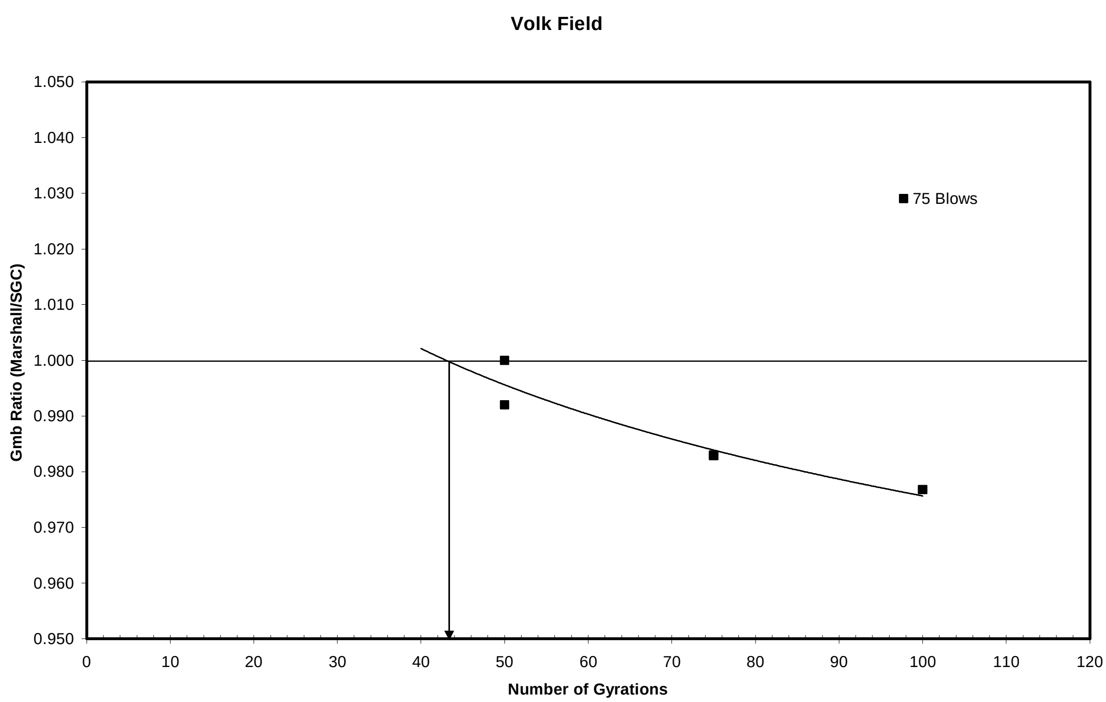

Figure 45: Comparison of Marshall Hammer and Superpave Gyratory Compactor – VOK

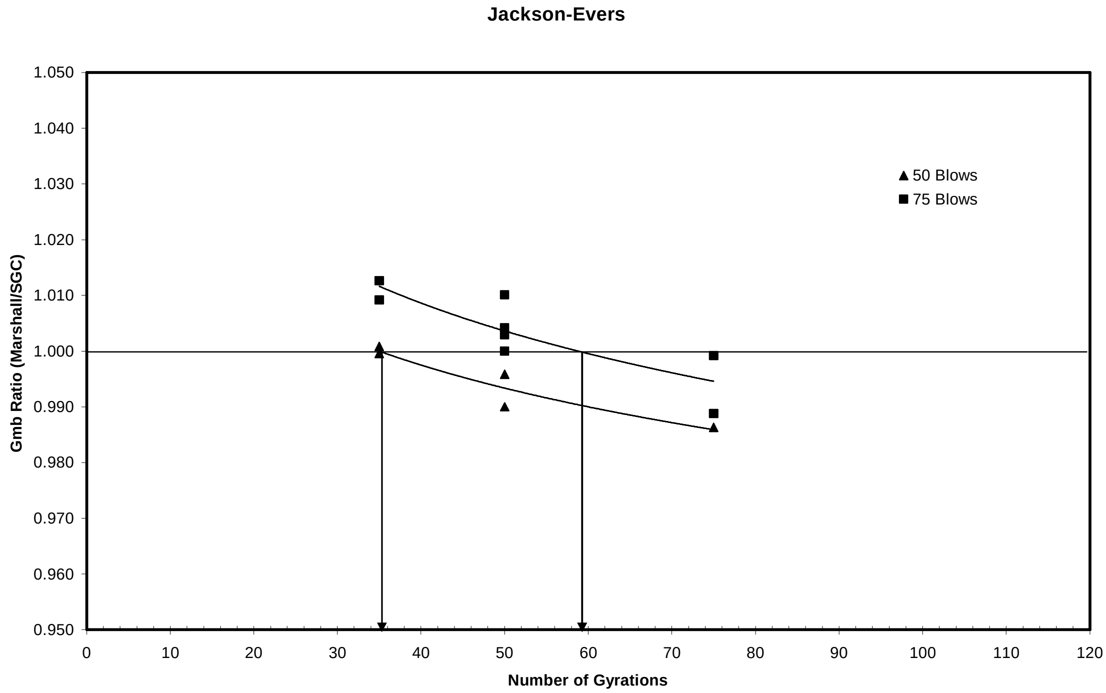

Figure 46: Comparison of Marshall Hammer and Superpave Gyratory Compactor – JAN

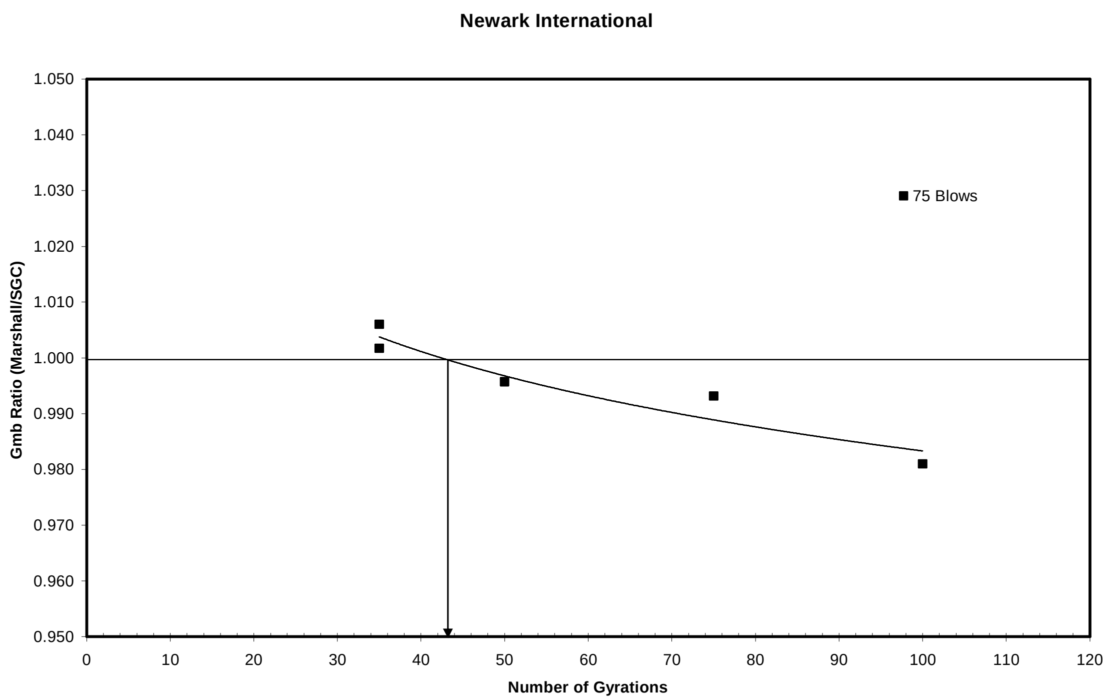

Figure 47: Comparison of Marshall Hammer and Superpave Gyratory Compactor – EWR

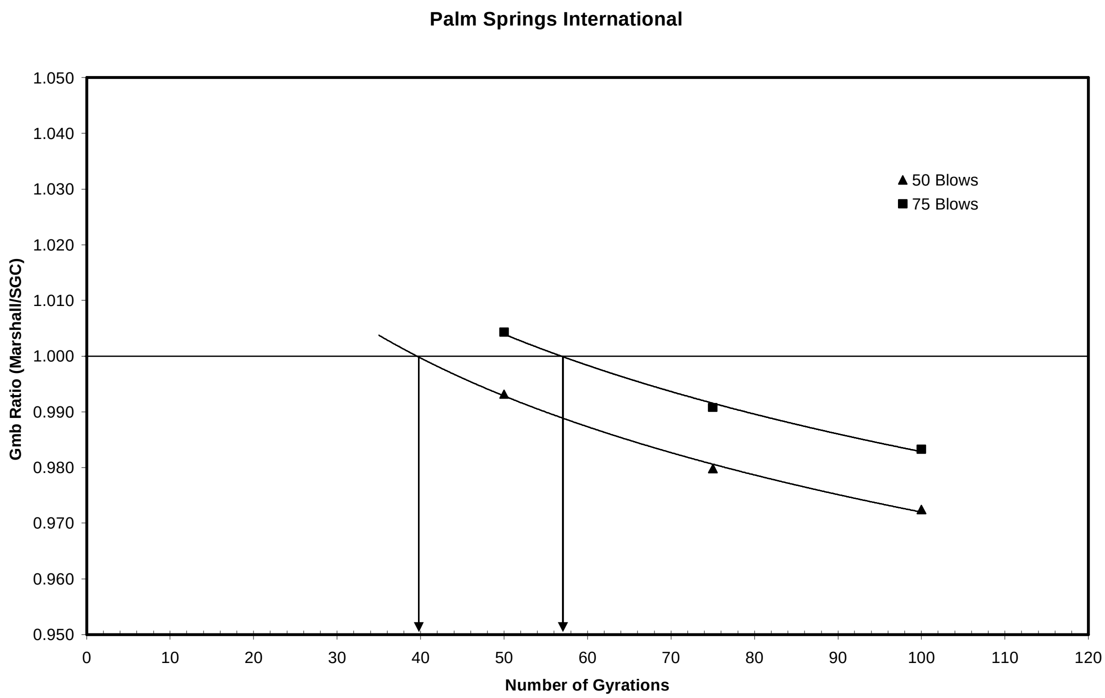

Figure 48: Comparison of Marshall Hammer and Superpave Gyratory Compactor – PSP

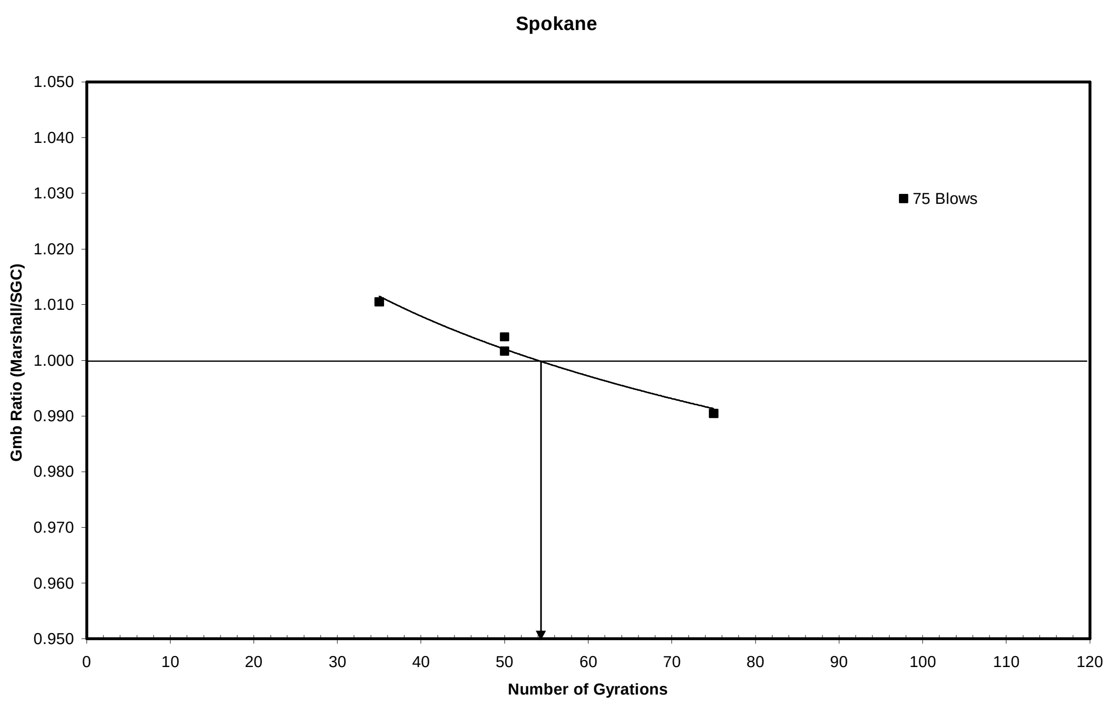

Figure 49: Comparison of Marshall Hammer and Superpave Gyratory Compactor – GEG

Figures 40 through 42 present the bulk specific gravity ratios for the three light traffic airfields (TSR, C24 and KHNZ). For each of these airfields, samples were compacted using both 50 and 75 blows per face of the Marshall hammer. For Jacqueline Cochran (TRM), the gyration levels corresponding to 50 and 75 blows per face were 20 and 55 gyrations, respectively. Also included on Figure 40 are upper and lower 95 percent confidence intervals for the 75-blow Marshall data. As can be seen, the 95 percent confidence interval is very wide which suggests a wide range of potential gyration values equivalent to the 75 blows per face data. There are a number of reasons that the confidence interval is large. First, the sample size for the regression line is relatively small which increases the t-statistic used in calculating the confidence interval. Secondly, there is variability in the density of samples used to develop the regression. Variability exists for both compaction methods and the determination of bulk specific gravity. The confidence interval is provided on Figure 40 to provide the reader an idea of the potential variability associated with selecting the gyration level equivalent to the compactive effort provided by the Marshall hammer. On subsequent figures, the confidence intervals are not shown. Identified gyration levels equivalent to Marshall compaction should be considered average values selected from the regressions. Using materials from Mineral County (C24), 49 and 59 gyrations corresponded to the 50 and 75 blows per face of the Marshall hammer compactive effort (Figure 41). Gyration levels of 43 and 52 corresponded to the 50 and 75 blows compactive effort for Oxford-Henderson (KHNZ) materials (Figure 42).

Figures 43 through 45 present the bulk specific gravity ratios for the military airfields (LRF, NTV and VOK). For Little Rock (LRF), only 75 blow Marshall samples were compacted. The gyration level corresponding to the density resulting from 75 blows per face of the Marshall hammer was 32. For materials from Oceana NAS (NTU), 28 and 34 gyrations corresponded to the 50 and 75 blow compactive effort, respectively. A gyration level of 43 corresponded to the 75 blow compactive effort using materials from Volk Field (VOK).

Figures 46 though 49 present the Marshall hammer and SGC comparisons for the commercial airfields (JAN, FWR, PSP, and GEG). For Jackson-Evers (JAN), 35 and 59 gyrations corresponded to the 50 and 75 blow compactive efforts, respectively. For materials from Newark-Liberty (EWR), 43 gyrations corresponded to 75 blows per face of the Marshall hammer. Gyration levels corresponding to 50 and 75 blows per face utilizing materials from Palm Springs (PSP) were 40 and 57, respectively. Finally, 55 gyrations corresponded to 75 blows per face using materials from Spokane (GEG).

As described above, there were a wide range of gyration levels that corresponded to 50 and 75 blows per face of the Marshall hammer. Figure 50 illustrates a histogram that presents the distribution of gyration levels obtained from Figures 40 to 49. For the data representing the 50 blows per face compactive effort, gyration levels ranged from a low of 20 gyrations to a high of 49 gyrations. Based on Figure 50, roughly 67 percent of the data suggests that the gyration level corresponding to a 50 blow compactive effort is between 40 and 50 gyrations. For the 75 blow data, the gyration level ranged from a low of 32 to a high of 59. Figure 50 shows that 60 percent of the gyration levels corresponding to the density achieved by 75 blows are between 50 and 60 gyrations. Also, 50 percent of the data is between 55 and 60 gyrations.

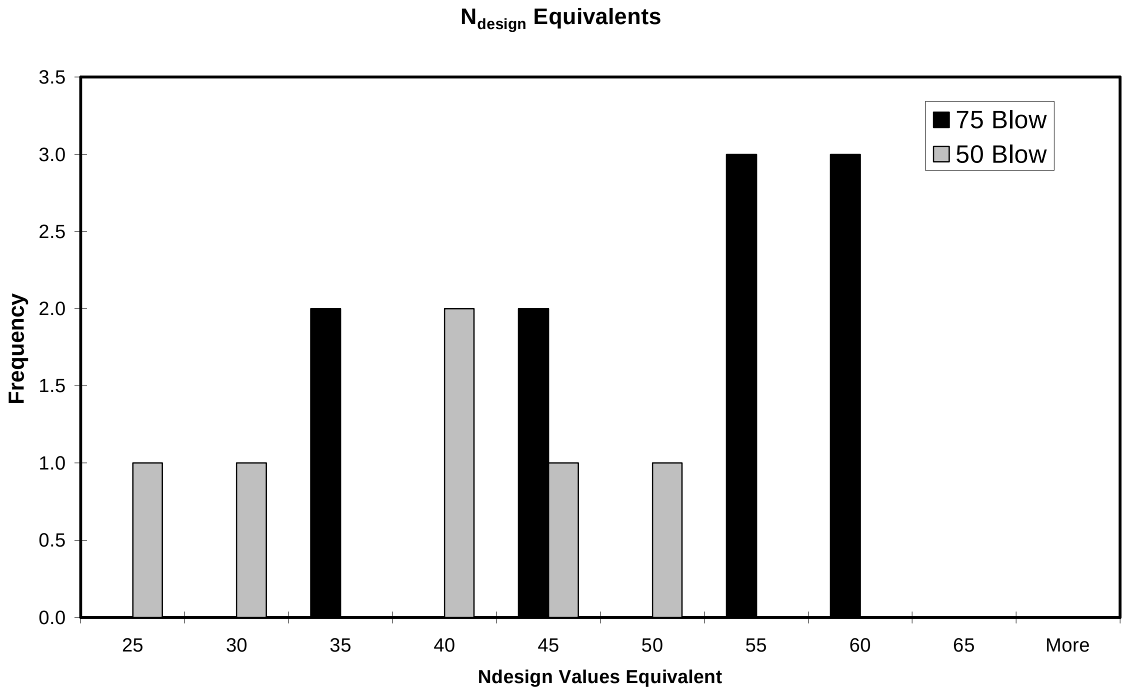

Figure 50: Histogram of N_{design} Equivalents to Marshall Compaction

The ten airfields visited as part of this project represent a relatively small sample of data. It would be anticipated that an equivalent number of gyrations to result in a similar density as the compactive effort provided by the Marshall hammer would be different for different aggregate types and different aggregate gradations. The range of equivalent gyration levels shown in Figure 50 seems to prove this assumption. Therefore, the data depicted in Figure 50 is considered only a sample of the overall population of gyration levels equivalent to 50 and 75 blows per face of the Marshall hammer. Assuming that the sample populations are normally distributed, confidence intervals can be developed to estimate where the true average would fall. Data needed to develop confidence intervals include the mean and standard error of the mean from the sample population. Confidence intervals are calculated as shown in Equation 1 for a 95 percent confidence interval.

$$\bar{x} \pm 1.96\,\frac{\sigma}{\sqrt{n}} \;=\; \textit{Confidence Interval at } 95 \textit{ percent} \qquad \textbf{Equation 1}$$

where:

x = mean of sample population;

σ = standard deviation of sample population; and

n = number of observations

For the 75 blows per face data depicted in Figure 50, the mean equivalent number of gyrations was 49 with a standard deviation of 10 gyrations for ten observations. Therefore, the estimated gyration level that provides an equivalent density to 75 blows per face falls between 43 and 55 gyrations at a 5 percent level of significance. This range of equivalent gyration levels equal to the compactive effort of 75 blows is slightly lower than anticipated; however, Brown and Mallick (14) did find an average of 1.4 percent lower air voids in samples compacted to 68 gyrations in an SGC when compared to samples compacted with 75 blows per face of the Marshall hammer. Prowell and Haddock (15) found an average difference of 1.9 percent air voids when comparing 68 gyrations and 75 blows per face compactive efforts, with the SGC again providing more compaction. Therefore, the upper part of the range (near the 55 gyrations) seems an appropriate estimate for a gyration level that provides an average equivalent density as 75 blows per face of the Marshall hammer.

For the 50 blows per face data depicted in Figure 50, the mean equivalent number of gyrations was 36 with a standard deviation of 11 gyrations for six observations. Therefore, the estimated gyration level that provides an equivalent density to 50 blows per face falls between 32 and 40 gyrations.

To summarize this second method of evaluating the proper design compactive effort, a range of equivalent gyration levels were determined for both 75 blows per face and 50 blows per face compactive efforts. The purpose for developing the ranges is that the true gyration level equivalent to a specific number of blows of the Marshall hammer will most likely change depending upon the aggregate characteristics, aggregate gradation, aggregate maximum aggregate size, variability in compacting samples in the Superpave gyratory compactor, variability in compaction of samples using the Marshall hammer, variability in preparation of samples, etc. When making final recommendations for the design compactive efforts, it will be important to understand how the selected gyration levels relate to the compactive efforts of 75 blows per face and 50 blows per face.

The final method of evaluating the proper design compactive effort using the Superpave gyratory compactor entailed evaluating the results of the laboratory performance testing, confined repeated load permanent deformation test (or commonly called the Flow Number test). Results from the Flow Number test are the number of loading cycles to failure. Samples were loaded to tertiary flow or 20,000 cycles, whichever came first. As stated previously, samples from each airfield were tested at varying deviator stresses. The deviator stresses were selected to represent various aircraft tire pressures. In order to evaluate the Flow Number results for the various airfields, information on the typical aircraft operating on the various airfield pavements was needed. For the civilian airfields, the Airport Master Record was obtained. The Airport Master Records contained data on the gross weights and gear configurations utilized on each airfield. LEDFAA was then used to determine the typical gross taxi weights and tire pressures for aircraft meeting the requirements depicted on the Airport Master Records. For the military airfields, the design aircraft were obtained from representatives from each respective airfield. Table 31 presents the typical aircraft along with specifics on the types of gears, gross taxi weights and tire pressures.

Table 31: Typical Aircraft Characteristics for Selected Airfields

Airfield	Design/Typical Aircraft*	Main Gear Wheels	Gross Taxi Weight (lbs)**	Gross Taxi Wt. Per Tire (lbs)	Tire Press. (psi)
Jacqueline Cochran Regional Airport (TRM)	Generic Sngl Whl-20	2	20,000	10,000	75
Mineral County Memorial Airport (C24)	Generic Sngl Whl-12.5	2	12,500	6,250	90
Oxford-Henderson Airport (KHNZ)	Generic Sngl Whl-30	2	30,000	15,000	75
Little Rock Air Force Base (LRF)	C - 130	4	155,000	38,750	105
Naval Air Station Oceana (NTV)	F/A-18	2	66,000	33,000	180
	F-14		45,000	22,500	240
Volk Field (VOK)	F-16	2	42,500	21,250	215
Jackson International Airport (JAN)	Generic Sngl Whl-75	2	75,000	37,500	120
	Generic Dual Whl-200	4	200,000	50,000	200
	Generic Dual Tan-400	8	390,000	48,750	200
	DDTW	16	890,000	55,625	200
Newark Liberty International Airport (EWR)	Generic Dual Whl-200	4	191,000	47,750	200
	Generic Dual Tan-400	8	358,000	44,750	200
	DDTW	16	873,000	54,563	200
Palm Springs International Airport (PSP)	Generic Sngl Whl-75	2	105,000	52,500	120
	Generic Dual Whl-200	4	200,000	50,000	200
	Generic Dual Tan-300	8	330,000	41,250	180
	DDTW	16	800,000	50,000	200
Spokane International Airport (GEG)	Generic Sngl Whl-75	2	200,000	100,000	120
	Generic Dual Whl-200	4	200,000	50,000	200
	Generic Dual Tan-400	8	400,000	50,000	200

* LEDFAA provided Design Aircraft tire pressure and Main Gear Wheel Numbers except for LRF, NTV and VOK

** all Gross Taxi Weights are from Master Airport list Except for LRF, NTV and VOK

Results of the Flow Number tests are presented with Figures 51 through 60 for each of the ten airfields. Results within each of these figures are presented as the measured Flow Number versus asphalt binder content. Additionally, for each asphalt binder content used for a specimen, the laboratory data was used to determine the gyration level required to result in 4 percent air voids. This data is also shown in parentheses next to the asphalt binder content. Recall from Chapter 2 that the goal of the performance testing was to determine the asphalt binder content at which the various airfield mixes would begin to exhibit rutting potential. The gyration level that corresponds to the asphalt binder content at which the mixes began to exhibit rutting potential would then be considered the minimum value at which the design compactive effort (N_{design}) could be selected. The goal of this methodology was to select a design compactive effort that would maximize durability while minimizing rutting potential.

As stated previously, all performance testing was conducted utilizing a confining stress of 40 psi. Test temperatures for the materials from each of the airfields were based upon historical temperature data. Pavement high temperatures were obtained from LTPPBind 3.1 and utilized for performance testing.

Figure 51 presents the results of performance testing conducted at 158° F on materials from Jacqueline Cochran (TRM). TRM is considered a general aviation airfield with relatively light aircraft utilizing the airfield. Table 31 showed a typical gross taxi weight of 20,000 lbs for a single wheel gear configuration with a typical design tire pressure of 75 psi for the runway. Figure 51 shows that the recreated mix from TRM did not reach tertiary flow within 20,000 cycles at a deviator stress of 100 psi up to an asphalt binder content of 5.5 percent. The mix also performed at a deviator stress of 200 psi with an asphalt binder content of 5.5 percent. At an asphalt binder content of 5.7 percent, the mix failed at all three deviator stresses. The data suggests that the potential for rutting significantly increased at an asphalt binder content between 5.5 and 5.7 percent. According the volumetric data from recreated Jacqueline Cochran mix, a design gyration level of 50 gyrations and design air void content of 4 percent would result in an asphalt binder content of 5.5 percent.

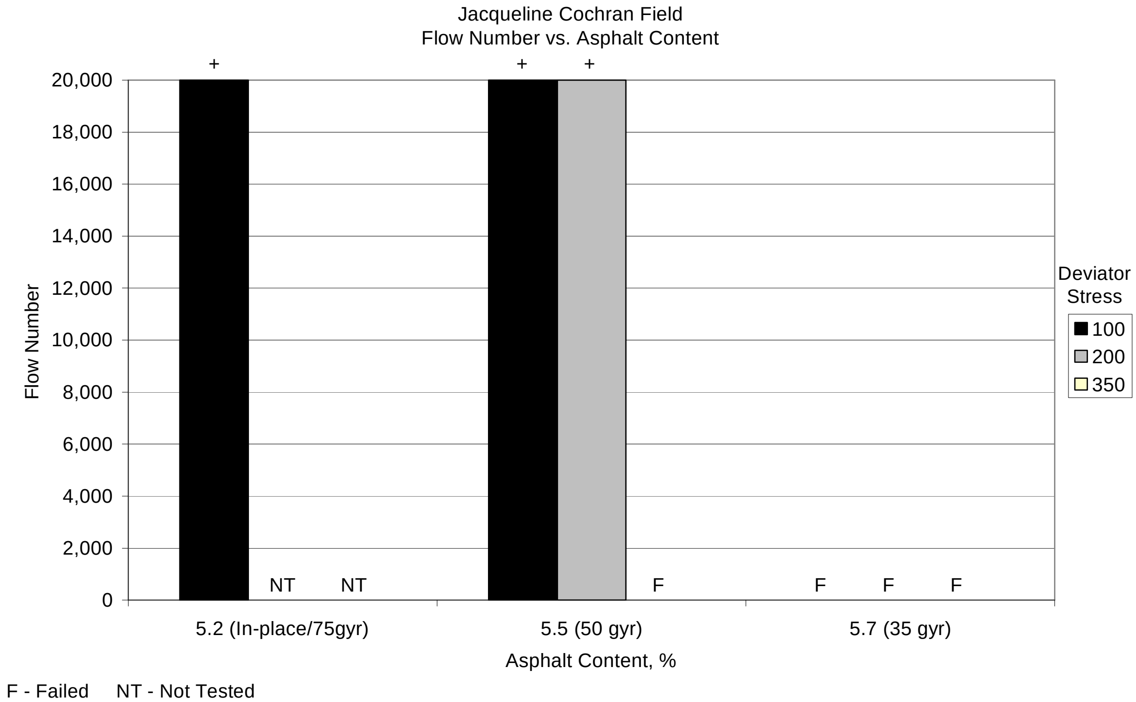

Figure 51: Flow Number Test Results for Jacqueline Cochran Regional Airport (TRM)

Figure 52 presents the results of Flow Number testing conducted at 126° F for materials from Mineral County Memorial Airport (C24). As shown in Table 31, aircraft using C24 are generally small with the design aircraft having a gross taxi weight of 12,500 lbs and tire pressures up to 90 psi. Based upon Figure 52, the recreated mix from C24 performed well at a deviator stress of 200 psi up to an asphalt binder content of 7.3 percent. The 7.3 percent asphalt binder content for the recreated mix represents a design number of gyrations of 50 and design air void content of 4 percent.

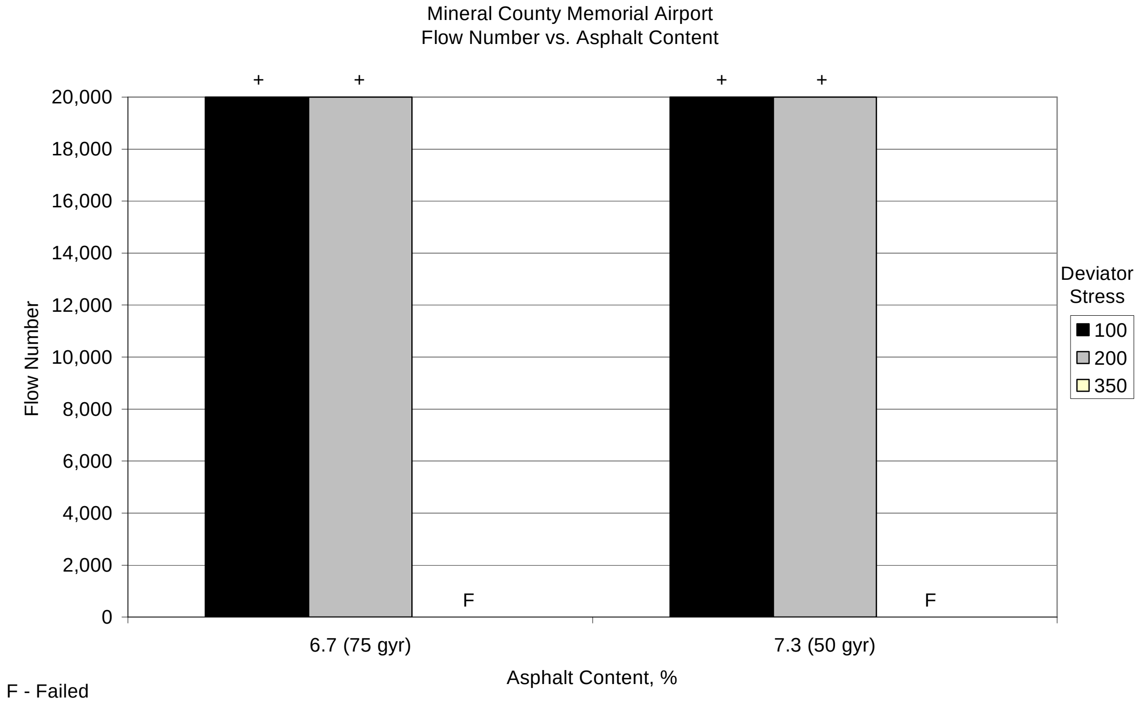

Figure 52: Flow Number Test Results for Mineral County Memorial Airport (C24)

Figure 53 presents the results of performance testing conducted at 140° F for Oxford-Henderson Airport (KHNZ). KHNZ is also considered a general aviation airfield with the runway rated for a single wheel main gear and gross weight of 30,000 lbs. The design aircraft also had a tire pressure of 75 psi. Performance testing was conducted utilizing three different asphalt binder contents which correspond to design compactive efforts (to achieve 4 percent air voids) of 35, 50 and 75 gyrations. Based upon the performance testing, the recreated mix from KHNZ did not achieve tertiary flow (passed) for any of the asphalt binder contents at deviator stresses of 100 and 200 psi. However, mixes did fail at the 350 psi deviator stress at the two highest asphalt binder contents. Considering the design aircraft for Oxford-Henderson, a design compactive effort as low as 35 gyrations resulted in a rut resistant mix.

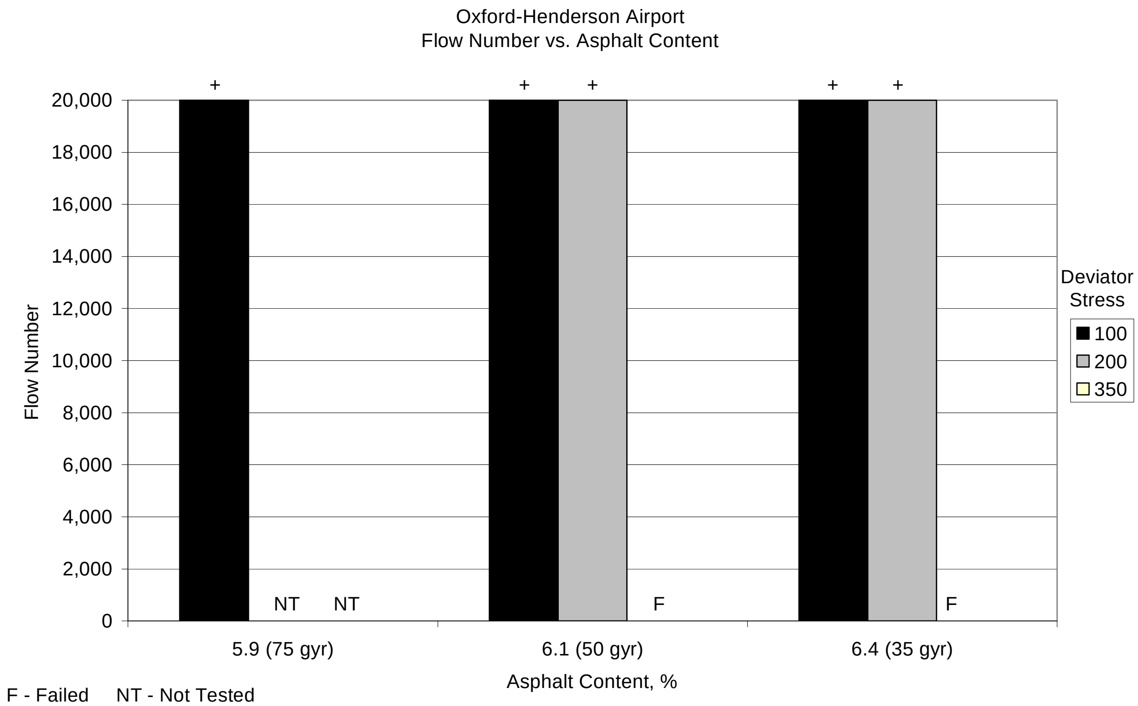

Figure 53: Flow Number Test Results for Oxford-Henderson Airport (KHNZ)

Results of the Flow Number testing conducted at 142° F on mix from Little Rock Air Force Base (LRF) are illustrated in Figure 54. Aircraft utilized at LRF are primarily C-130's with a gross taxi weight of 155,000 lbs and tires having a tire pressure of 105 psi. Again, performance testing was conducted at three different asphalt binder contents corresponding to design compactive efforts of 50, 75 and 100 gyrations. Samples at all three asphalt binder contents passed the Flow Number test at 100 and 200 psi deviator stresses; however, all reached tertiary flow before 20,000 cycles when tested at 350 psi. Considering the typical aircraft weight and tire pressure, the minimum design gyration that could be selected for the Little Rock Air Force Base materials would be 50 gyrations. However, the value could be lower considering the mixture reached 20,000 cycles at 100 and 200 psi deviator stresses at 50 gyration asphalt binder content

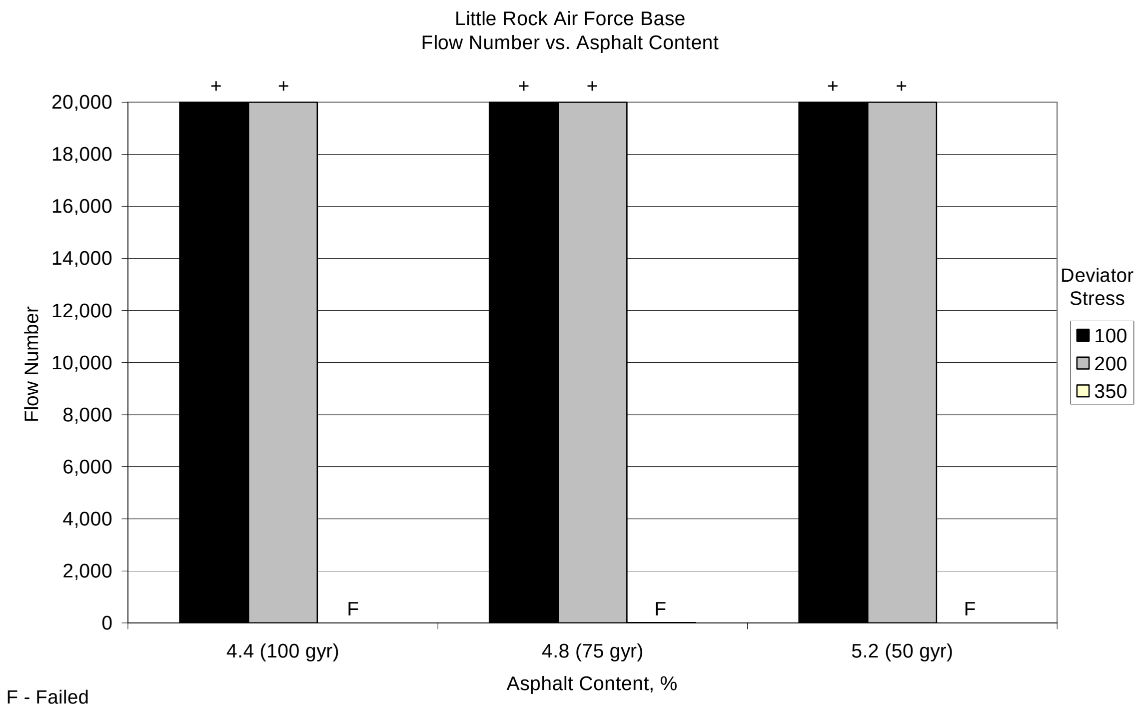

Figure 54: Flow Number Test Results for Little Rock Air Force Base (LRF)

Results from performance testing conducted at 136° F on materials from Naval Air Station Oceana (NTV) are presented on Figure 55. Performance testing was conducted at four different asphalt contents, representing design compactive efforts of 22, 35, 50, and 75 gyrations. The asphalt binder content corresponding to a design compactive effort of 22 gyrations was selected in order to test at an asphalt binder content equal to the job mix formula. Table 31 showed that the F/A-18 is the primary aircraft utilizing NTV; however, F-14's also operate at NTV, though to a lower extent. The tires on a F/A-18 are generally rated at 180 psi, while the tires for F-14's are rated at 240 psi. Based upon the test results, samples at all four asphalt binder contents reached 20,000 cycles during the Flow Number test when a deviator stress of 100 psi was utilized. At a deviator stress of 200 psi, only the sample with an asphalt binder content corresponding to a 75 gyration design compactive effort reached 20,000 cycles during testing. None of the samples reached 20,000 cycles when using a 350 psi deviator stress. Recall that the taxiway included within this investigation had rutted. The data suggests that the potential of rutting was observed at all asphalt binder contents (or design compactive efforts) except possibly the 75 gyration design compactive effort.

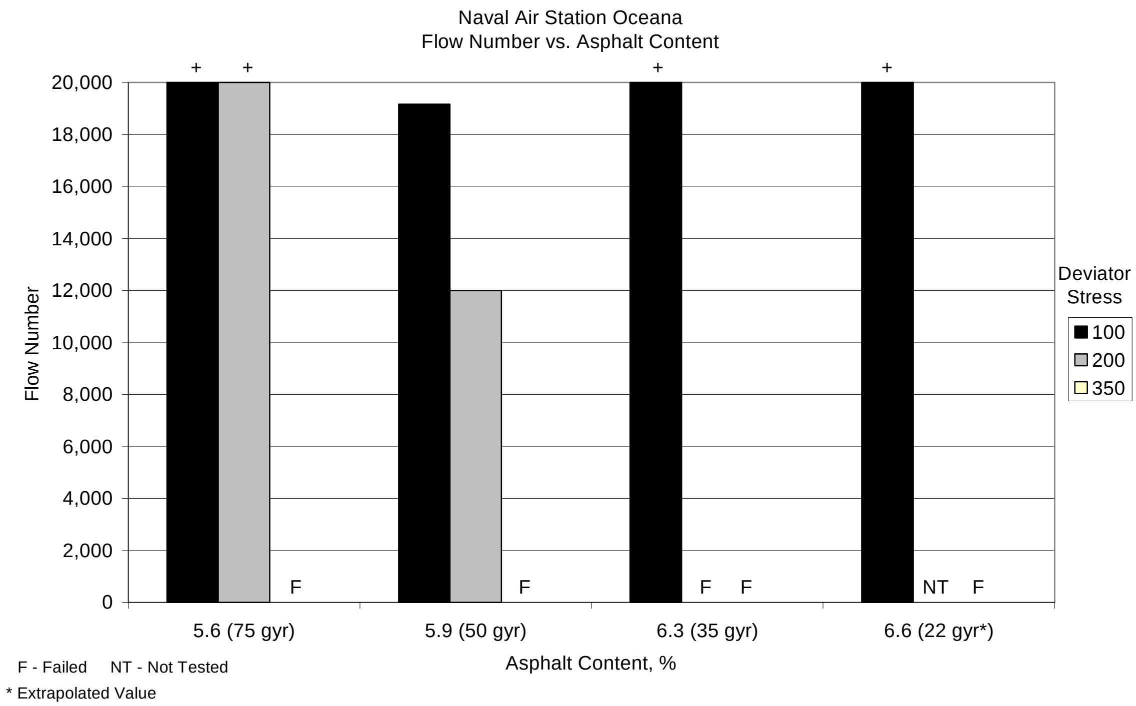

Figure 55: Flow Number Test Results for Naval Air Station Oceana (NTV)

Figure 56 presents the results of Flow Number Testing conducted at 127° F on recreated mixes from Volk Field (VOK). Mix from VOK was tested at three different asphalt binder contents representing design compactive efforts of 55, 75 and 100 gyrations. The primary aircraft utilizing the selected runway from VOK are the F-16 and F-18. Tire pressures for these aircraft are generally near 215 psi. Based upon the test results, mixes prepared at binder contents representing 100 and 75 gyrations achieved the 20,000 load cycles at deviator stresses of 100 and 200 psi. All of the mixes failed at a deviator stress of 350 psi. The typical tire pressure of 215 psi is slightly higher than the deviator stress of 200 psi utilized during testing, however, based upon the data a minimum design compactive effort of 75 to 100 gyrations appears to result in a mix with minimal rut potential.

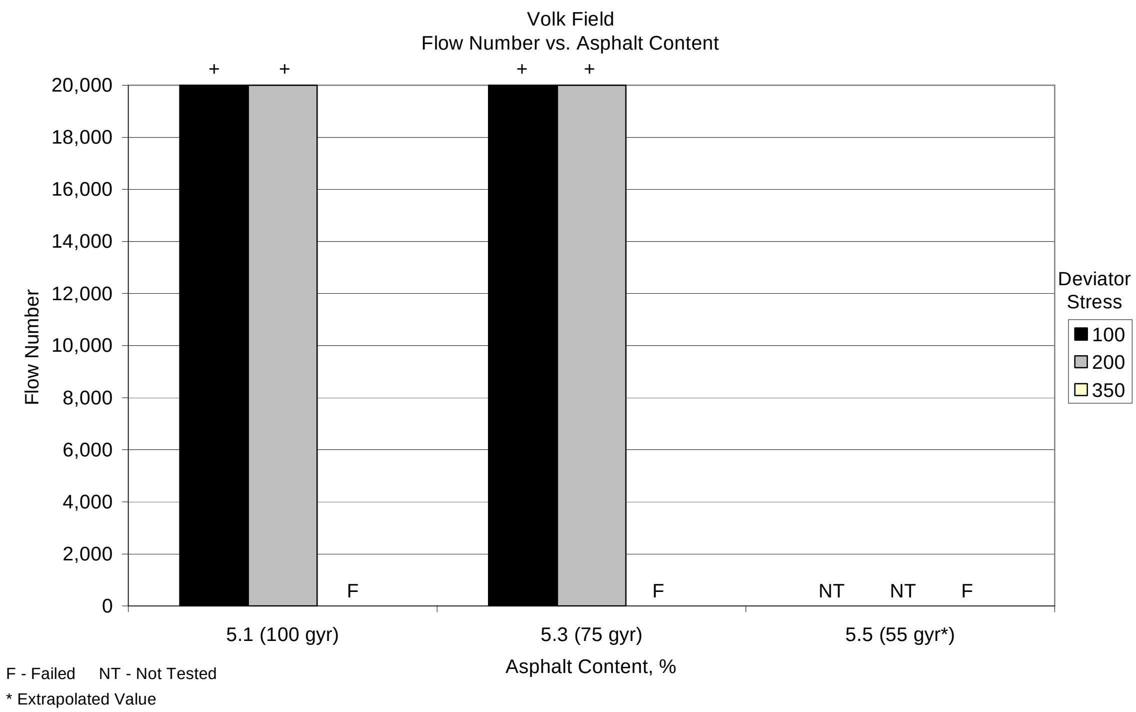

Figure 56: Flow Number Test Results for Volk Field (VOK)

Results of performance testing conducted at 145° F on mixes representing Jackson-Evers International Airport (JAN) are illustrated in Figure 57. A total of four asphalt binder contents were tested using materials obtained for the JAN runway. The four asphalt binder contents represent design compactive efforts of 16, 35, 50 and 75 gyrations. Table 31 showed four aircraft that are typical for JAN. Of the four typical aircrafts, the highest tire pressure anticipated is 200 psi. Figure 57 shows that mixes at three of the four asphalt binder contents achieved 20,000 cycles during the Flow Number test at both 100 and 200 psi. All of the samples tested at a deviator stress of 350 psi reached tertiary flow. Based upon this data, the minimum design compactive effort that would achieve rut resistance for the Jackson-Evers materials would be 35 gyrations.

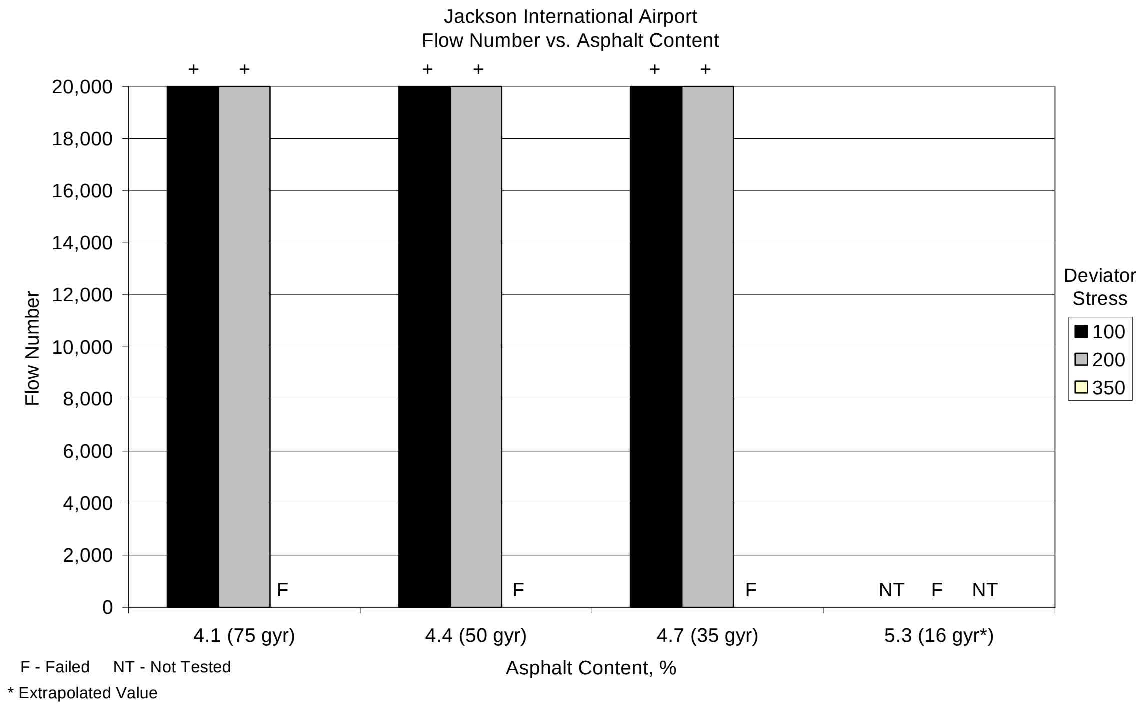

Figure 57: Flow Number Test Results for Jackson-Evers International Airport (JAN)

Figure 58 presents performance testing results conducted at 131° F for the Newark-Liberty International Airport (EWR). A total of four asphalt binder contents were used for materials obtained from EWR, representing design compactive efforts of 35, 50, 75 and 132 gyrations. From Table 31, the largest typical tire pressure for EWR is 200 psi. Figure 58 shows that mixes prepared at all four asphalt binder contents reached 20,000 cycles at a deviator stress of 200 psi. All of the samples tested at a 350 psi deviator stress reached tertiary flow prior to the 20,000 cycles. The rut resistance exhibited by this mix is somewhat surprising because the mixture selected from EWR for performance testing was from a shoulder. Recall that two pavements were sampled from

EWR: a shoulder and a runway mix. These two mixes were basically identical except that a polymer modified PG 76-28 was utilized on the runway. Based upon the data shown in Figure 58, the minimum design compactive effort that would result in a rut resistant mix would be 35 gyrations.

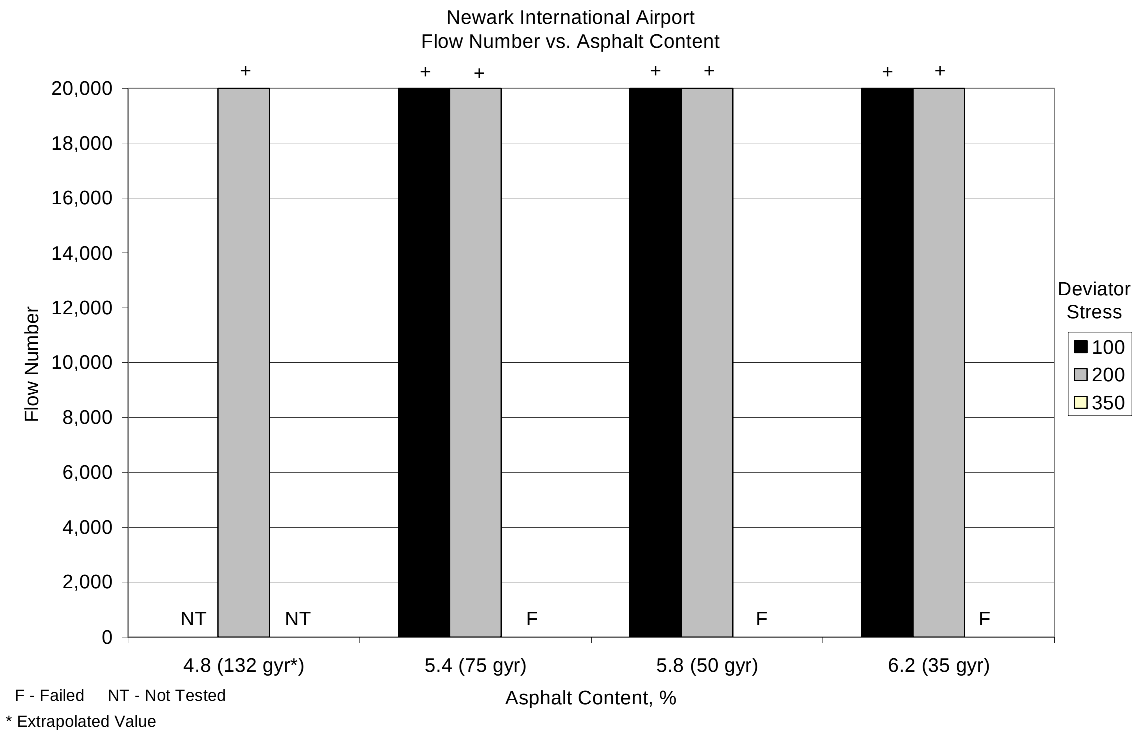

Figure 58: Flow Number Test Results from Newark-Liberty International Airport (EWR)

Figure 59 illustrates results of the performance testing conducted at 160° F on the recreated mix from Palm Springs International Airport (PSP). Of the ten airfield mixes evaluated within this study, the mix representing PSP did not perform well during the laboratory testing. Figure 59 shows that most of the samples failed when the deviator stresses were applied. Because of the failures, the researchers took a critical look at the materials and mixture characteristics of the PSP mix to determine if there was a potential reason for the lack of performance during the Flow Number testing. Of the materials used in the mix, the characteristic that seemed to explain the lack of performance for the mix was the grade of asphalt binder. As presented earlier, an AC-20P asphalt binder was specified for the PSP mixture. In order to formulate this binder to recreate the PSP mix, the asphalt binder supplier was contacted. According to the supplier, the asphalt binder was formulated by beginning with AC-7.5 viscosity graded binder and adding the appropriate amount of polymer to result in an AC-20 viscosity graded asphalt binder. The engineer who was on-site during construction of this pavement indicated that the AC-20P was chosen because of good past experience with the binder. The research team then graded this asphalt binder using the Superpave Performance Grading system and the binder graded as a PG 58-XX.. According to LTTPBind, pavement temperatures can reach as high as 70.6°C at PSP. Because of the discrepancy between the high temperature of the Performance Grade of the binder and the anticipated pavement temperatures, the researchers again checked with the asphalt binder supplier to verify the formulation of the binder. The supplier reiterated that the final product after formulation met an AC-20.

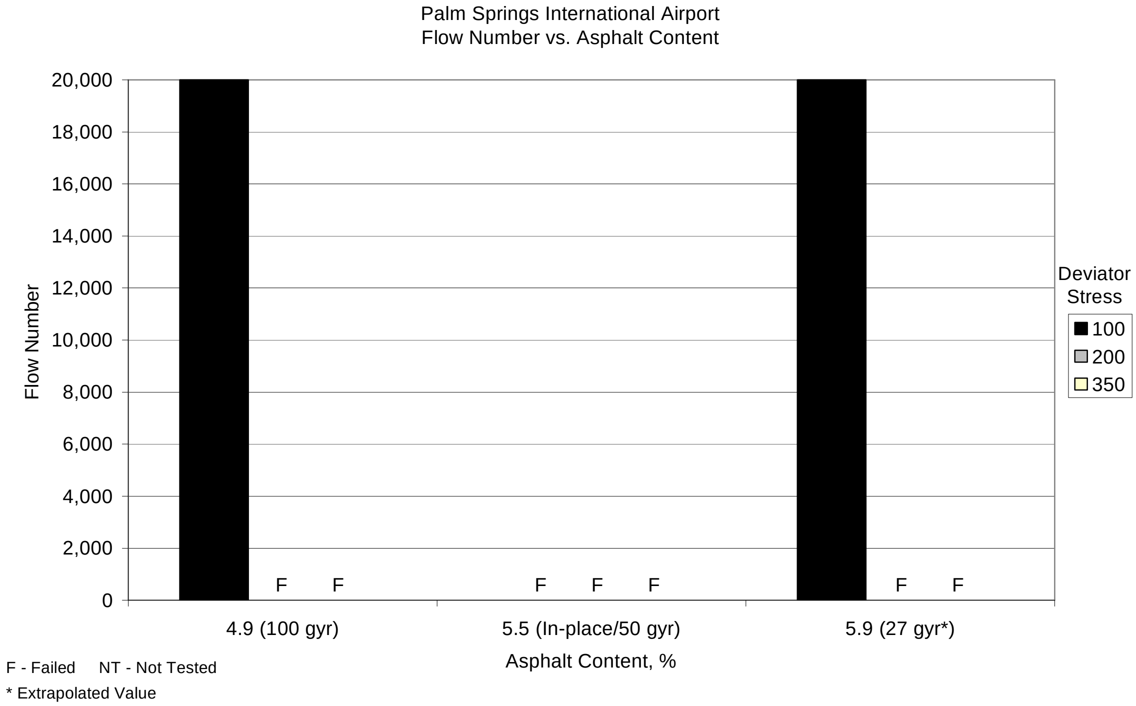

Figure 59: Flow Number Test Results from Palm Springs International Airport (PSP)

A potential reason that the pavement has performed well is shown in Figure 60. This figure shows the average number of aircraft operations and the average maximum air temperatures for each month of the year 2006. Aircraft operations are much higher during cooler months. During the hotter moths, operations are drastically reduced. Because of the concerns with the asphalt binder and lack of laboratory performance, no observations about the desirable design gyration level for PSP are made.

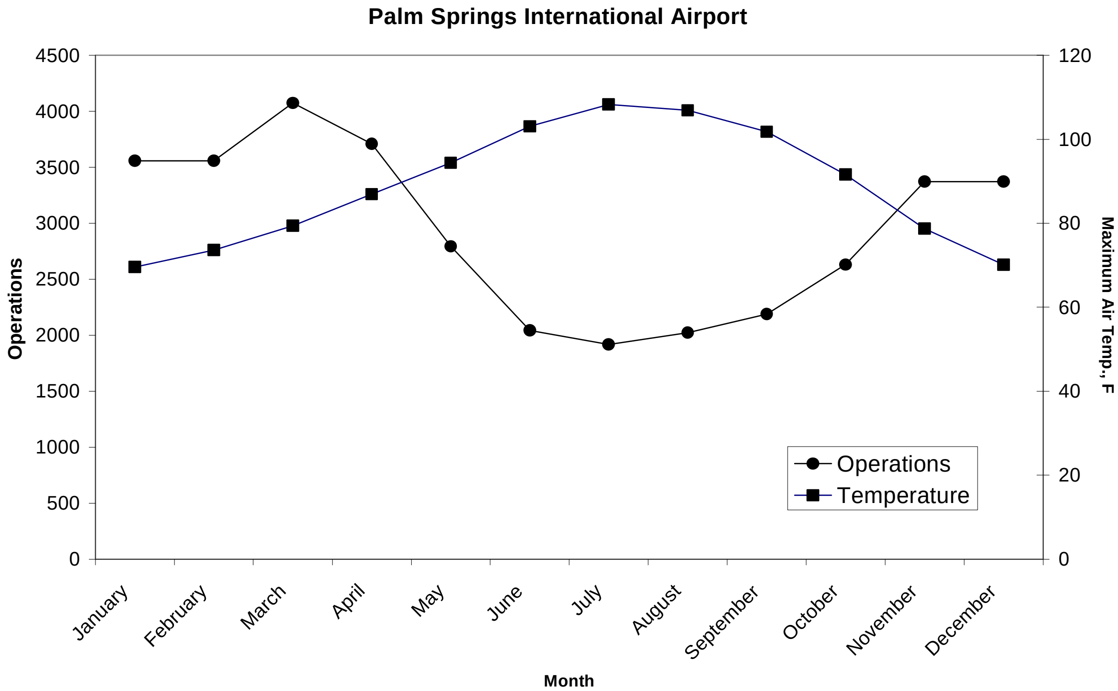

Figure 60: Operation and Temperature Characteristics for Palm Springs International Airport (PSP)

Results of the Flow Number testing conducted at 124° F on the recreated mix for Spokane International Airport (GEG) are illustrated on Figure 61. A total of three asphalt binder contents were utilized for testing of the GEG mix, representing design compactive efforts of 35, 50 and 75 gyrations. From Table 31, the largest tire pressure typically used on aircraft utilizing GEG is 200 psi. Figure 61 shows that all of the samples tested at 100 psi reached 20,000 cycles. At a deviator stress of 350 psi, mixtures created at all three asphalt binder contents failed. Results of testing at a deviator stress of 200 psi were mixed. The sample prepared at the lowest asphalt binder content failed; however, the two samples prepared at the two higher asphalt binder contents did not immediately fail upon application of the deviator stress. Since the values shown within the figure represent the average of three specimens, it is unclear if the results shown for the lowest or two highest asphalt binder contents are correct; therefore, no specific observations about the appropriate design gyration level is made.

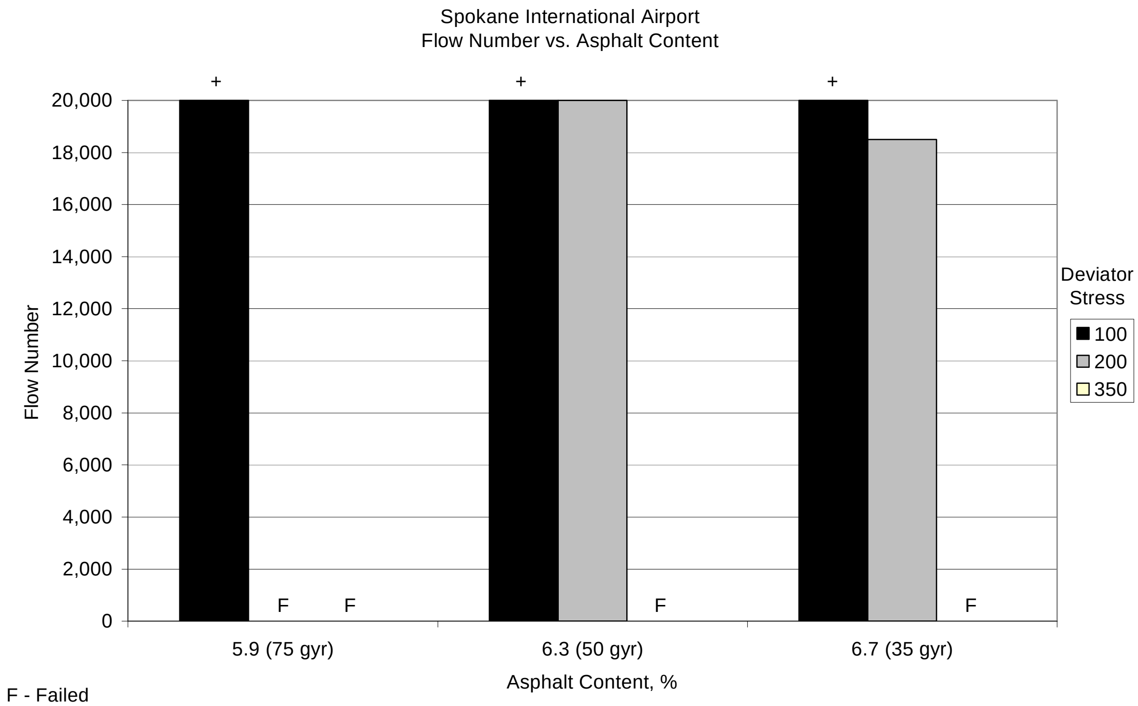

Figure 61: Flow Number Results for Spokane (GEG)

Table 32 summarizes the estimated gyration levels determined utilizing the results of the performance testing. Design compactive efforts for Marshall designed mixes currently contained within Item P-401 are based upon a combination of aircraft gross weights or tire pressure. The design compactive efforts are differentiated based upon a gross aircraft weight of 60,000 lbs or tire pressures of 100 psi. Within Table 32 are the maximum gross taxi weights and tire pressures that were shown in Table 31.

Table 32: Estimated N_{design} Values Based upon Performance Testing

Airfield	Max. Gross Wt. (lbs)	Max. Gross Wt. per Tire (lbs)	Tire Pressure (psi)	Estimated N_{design} Value
Jacqueline Cochran Regional Airport (TRM)	20,000	10,000	75	50
Mineral County Memorial Airport (C24)	12,500	6,250	90	50
Oxford-Henderson Airport (KHNZ)	30,000	15,000	75	35
Little Rock Air Force Base (LRF)	155,000	38,750	105	50
Naval Air Station Oceana* (NTU)	66,000	33,000	240	75
Volk Field (VOK)	42,500	21,250	215	75
Jackson International Airport (JAN)	890,000	55,625	200	35
Newark Liberty International Airport (EWR)	873,000	54,563	200	35
Palm Springs International Airport (PSP)	800,000	52,500	200	N/A
Spokane International Airport (GEG)	400,000	100,000	200	N/A

N/A – Insufficient Data to Estimate Appropriate N_{design} Value
* Evaluated mix rutted in the field.

The estimated design gyration levels shown in Table 32 are interesting. Estimated design gyration levels ranged from a low of 35 gyrations to a high of 75 gyrations. Based upon the performance testing, none of the airfields would have required design gyration levels above 75 gyrations. The low estimated design gyration level of 35 is interesting as there is little to no experience with mixtures designed using such a low N_{design}. However, the 35 gyrations does fall within the range of gyration levels found equivalent to 50 blows of the Marshall hammer.

The three general aviation mixtures (TRM, C24, and KHNZ) had estimated design compactive efforts ranging from 35 to 50. Two of the three had an estimated design compactive effort of 50 (TRM and C24) while the third had an estimated design compactive effort of 35 (KHNZ). The mix from C24 could potentially have an appropriate design compactive effort of less than 50 as the mix was rut resistant even at a 200 psi deviator stress at the highest asphalt binder content tested (representing a design compactive effort of 50 gyrations). The actual design compactive efforts for these mixes during mix design were as follows: TRM, 75 blows per face of the Marshall hammer; C24, 76 gyrations of the SGC; and KHNZ, 75 gyrations of the SGC. Figures 38 and 39 showed the ultimate densities for these three airfield pavements. None of the three mixes reached an ultimate density equal to the design density. This would suggest that the three mixes were designed using a compactive effort that was greater than what was needed. All three of these airfields had aircraft utilizing the pavements with maximum gross taxi weights of 30,000 lbs or less and tire pressures less than 90 psi. Based upon Item P-401, the appropriate design compactive effort for these three mixes would have been 50 blows per face of the Marshall hammer. Based upon the SGC gyration range equivalent to a 50 blow Marshall design, which was 32 to 40 gyrations, this data suggests that an N_{design} value of 40 would be appropriate for general aviation airfields.

Another interesting observation from Table 32 is that the three military airfields, LRF, NTU and VOK, had somewhat similar estimated design gyration levels ranging from 50 to 75 gyrations. Two of these military airfields also had the highest tire pressures anticipated to traffic the pavements with NTU having 240 psi tires and VOK having 215 psi tires. The 75 gyration levels shown in Table 32 are higher than the equivalent range of design gyrations (43 to 55) corresponding to 75 blows of the Marshall hammer that was found in this project. This would indicate that the 75 blow per face design compactive effort may not have been appropriate for these projects. Additionally, airfields, especially military, in which aircraft having very high tire pressures will need to be designed at higher design gyrations than at typical commercial airfields.

The final grouping of data was for commercial airfields, which included JAN, EWR, PSP and GEG. Design tire pressures for all four of these airfields were 200 psi. Unfortunately, the performance testing data from two of these airfields (PSP and GEG) did not pass the test of reasonableness, as described previously. For the remaining two airfields, the estimated design gyration level was 35 gyrations. As stated previously, this level of design gyrations corresponds to a 50 blow Marshall design compactive effort which experience suggests is unreasonable.

Discussion on Selection of Design Gyration Levels for Airfield Superpave Mix Designs

In order to develop N_{design} values for adapting the Superpave gyratory compactor methods for airfield pavements, the researchers evaluated the data using three different methods. The first method looked at the ultimate densities of the ten airfield pavements included within this project. Results from this analysis suggested that airfield mixes that had been designed using the Superpave gyratory compactor had been designed at N_{design} values that were too high. The ultimate densities of these pavements were all below the design densities, suggesting a lack of asphalt binder in the mixes.

The second method for evaluating the appropriate N_{design} levels involved comparing the density that resulted from Marshall hammer and SGC compaction. As expected, there was no single gyration level that was exactly equal to either 50 blows per face or 75 blows per face of the Marshall hammer. Therefore, ranges of gyration levels equivalent to each Marshall hammer compactive effort were developed using materials from the ten airfields visited during this project. Based upon the data, the gyration level equivalent to 50 blows per face of the Marshall hammer was somewhere between 32 and 40 gyrations. The gyration level equivalent to 75 blows per face of the Marshall hammer was somewhere between 43 and 55 gyrations.

The final method of evaluating the data was to analyze the results of the performance testing of mixes having different asphalt binder contents. The goal of this testing was to maximize durability, while minimizing rutting potential. For general aviation types of airfields which operate light aircraft with relatively low tire pressures, the analysis of the performance data indicated that an appropriate N_{design} value would be approximately 40. This value falls within the range of gyration levels equivalent to 50 blows per face of the Marshall hammer and seems reasonable. For the military type of airfields which operate aircraft with relatively high tire pressures, the analysis of performance data suggested that an appropriate N_{design} value of approximately 75 would be appropriate. However, the data did also indicate that for military airfields in which aircraft having relatively lower tire pressures, a lower N_{design} value would also be appropriate. Unfortunately, the data for commercial type airfields was inconclusive.

As shown in Table 32, selection of design gyration levels appear to be more related to tire pressures than aircraft gross weights. For instance, the maximum weight of aircrafts at LRF is 155,000 lbs. The estimated N_{design} value for this airfield was 50 gyrations. However, at NTV, the maximum gross weight of aircraft was 33,000 lbs. The estimated N_{design} value for this airfield was 75 gyrations. The big difference between these two airfields is that the aircraft operating at NTV had tire pressures of 240 psi, while at LRF the tire pressures were 105 psi. Therefore, tire pressures should be included within the criteria for selection of the proper N_{design} value when designing HMA for airfield applications.

Table 33 provides N_{design} values based upon the results of this research. The only criteria for selecting the appropriate N_{design} value included within Table 33 is tire pressure. Table 32 indicated no relevance of maximum gross taxi weight on selection of the design gyration value. This is not to state the aircraft weight is not important, because aircraft weight is very important in pavement thickness design. The design gyration levels of 40 and 55 were selected because they are roughly equivalent to 50 and 75 blows per face of the Marshall hammer, respectively. The 70 gyration design compactive effort was selected in order to provide a more rut resistant HMA mix at airfields that will be subjected to high tire pressures.

Table 33: N_{design} Values Based Upon Research

Tire Pressure, psi	N_{design}
Less than 100	40
100 to 200	55
More than 200	70

The N_{design} values shown in Table 33, specifically the gyration level of 40, are somewhat lower than was expected at the onset of this project. As stated previously, there is very little experience with HMA mixes designed with a compactive effort as low as 40 gyrations. The possibility for the lower than expected numbers could be that the researchers purposely developed a research approach that would maximize the durability of HMA mixes designed with the SGC. However, it is still very important to minimize the rutting potential of airfield HMA mixes. Rutting can cause directional control problems or lead to the increased potential for hydroplaning during rain events Therefore, the researchers will recommend a slightly different table of design gyration levels in which experience indicates that the mixes will be durable and rut resistant. Table 34 presents the recommended N_{design} values to be used for designing airfield HMA using the Superpave mix design method.

Table 34: Recommended N_{design} Values for Designing Airfield Mixes

Tire Pressure, psi	N_{design}
Less than 100	50
100 to 200	65
More than 200	80

Design gyration values within Table 34 are 10 gyrations higher for each tire pressure category. These additional gyrations were added to each category to minimize the potential for rutting while still providing durability. The 50 gyration design level is a slightly higher compactive effort than 50 blows per face of the Marshall hammer. The 65 gyration level is slightly higher compactive effort than 75 blows per face of the Marshall hammer as found in this project, while the 80 gyration level was added to specifically address airfield pavements that experience high tire pressure.

EVALUATION OF GRADATION REQUIREMENTS

Unfortunately, the time and budget constraints for AAPTP 04-03 did not allow for a complete evaluation of the influence of gradation characteristics on airfield Superpave designed mixes. Because the durability of airfield pavements is of paramount importance, there was concern by the research team with coarser gradations currently allowed within the highway version of the Superpave mix design system. Research (9) has shown that mixes having coarser gradations tend to be more permeable than mixes having finer gradation shapes at a given in-place density (or air voids). Permeable pavement layers can lead to durability problems during the life of the pavement. There are two aspects of a pavement that can be adversely affected by permeability, infiltration of air and water. Kumar and Goetz (11) have shown a direct relationship between permeability and asphalt age hardening. When air (oxygen) reacts with the asphalt binder within HMA, the binder becomes more brittle (age hardens) which increases the potential for both cracking and raveling. Both of these distresses can lead to FOD. Likewise, the infiltration of water into a pavement layer can lead to moisture damage. Moisture damage can reduce the structural capacity of a pavement layer as well as lead to raveling.

Because of the concerns with coarser gradations, an experiment was carried out that was intended to evaluate the effect of gradation on the permeability characteristics of HMA mixes. The intent of altering the gradations of airfield mixes was to evaluate the relationship between permeability and density. This evaluation allowed the research team to make recommendations on the applicability of the gradation requirements within the highway version of the Superpave mix design procedure for airfields. Because durability is the primary distress mechanism on airfield pavements, limiting the potential for permeable pavements was important. Five of the ten airfield mixes were selected for this experiment that had sufficient materials for the additional testing. The selected airfields were Oceana Naval Air Station (NTV), Oxford-Henderson Airport (KHNZ), Mineral

County International Airport (C24), Volk Field (VOK), and Jackson-Evers International Airport (JAN).

Figures 62 through 66 illustrate the design gradations for each of the selected airfields as well as the additional gradations that were developed and evaluated. In each instance, the finest gradation shown on the figures was the design gradation for the airfield mixes. Also included on each of these figures are the upper and lower gradation control limits for the Superpave mix design system for highways. Because each of the design gradations was relatively fine, the developed gradations were coarser than the design gradation. Of the five airfield mixes, three are considered a 12.5 mm nominal maximum aggregate size gradation (NMAS), one a 9.5 mm NMAS and one a 19.0 mm NMAS according to the Superpave definition of NMAS.

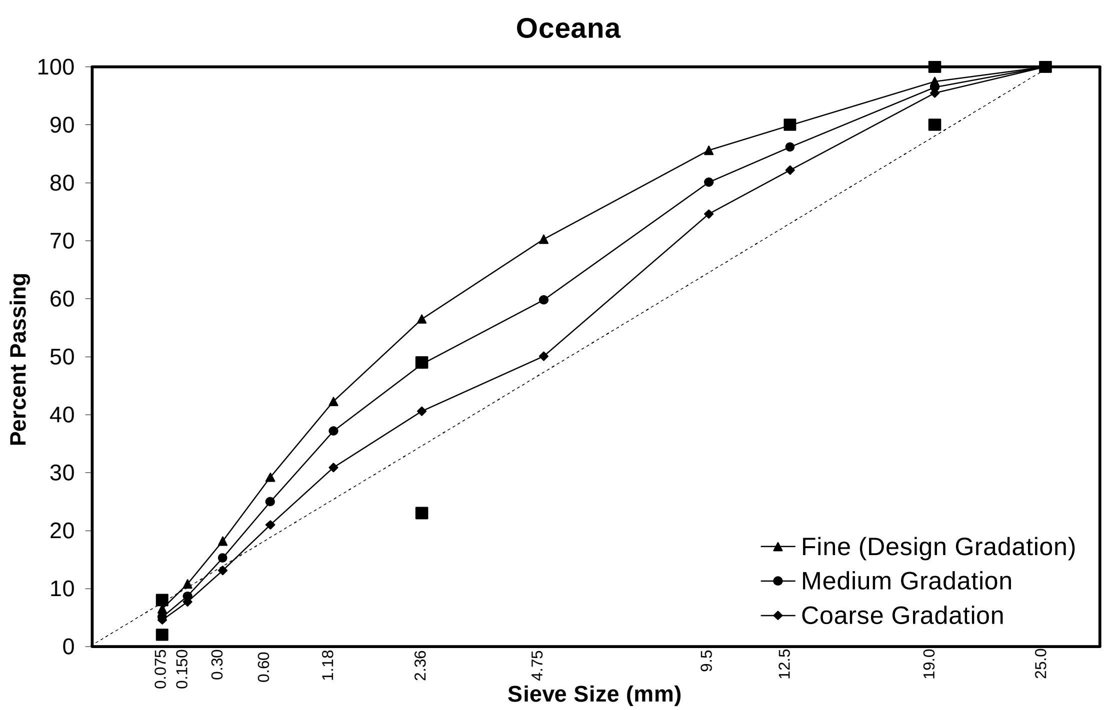

Figure 62: Gradations from NTV Used in Permeability Testing

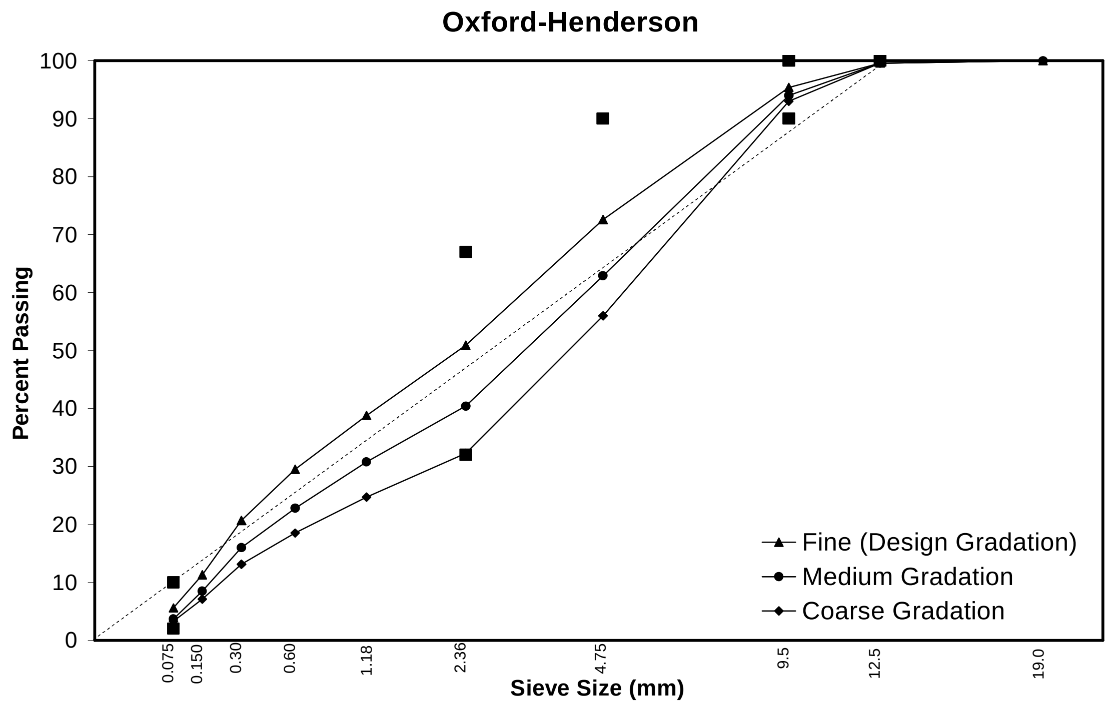

Figure 63: Gradations from KHNZ Used in Permeability Testing

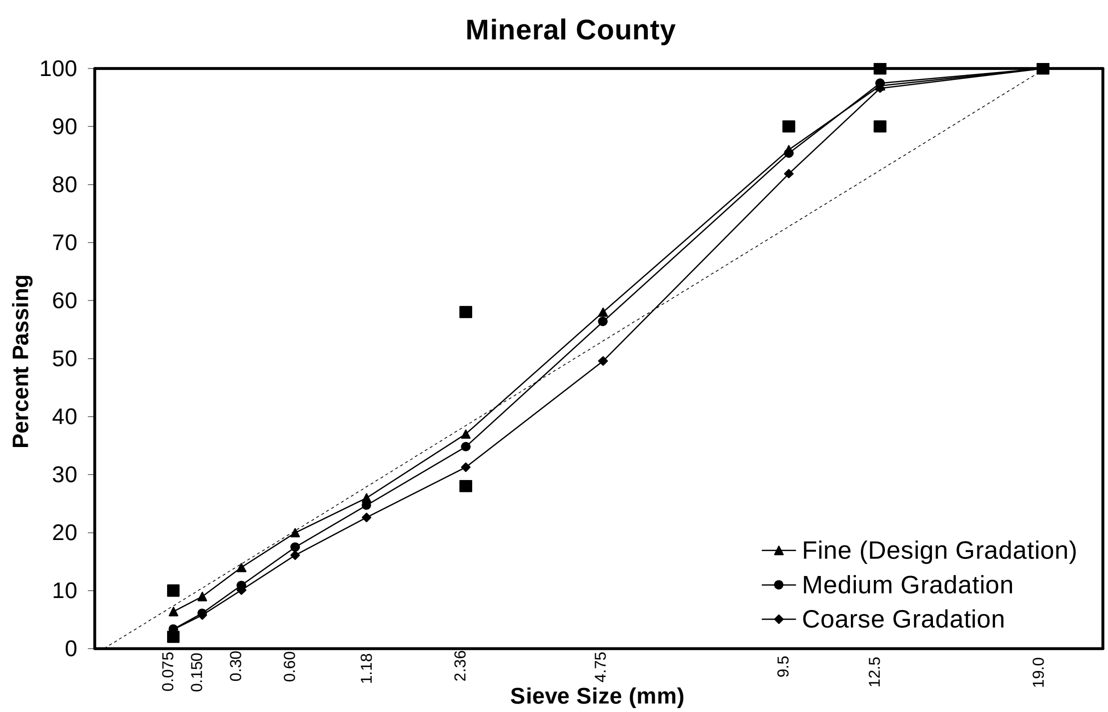

Figure 64: Gradations from C24 Used in Permeability Testing

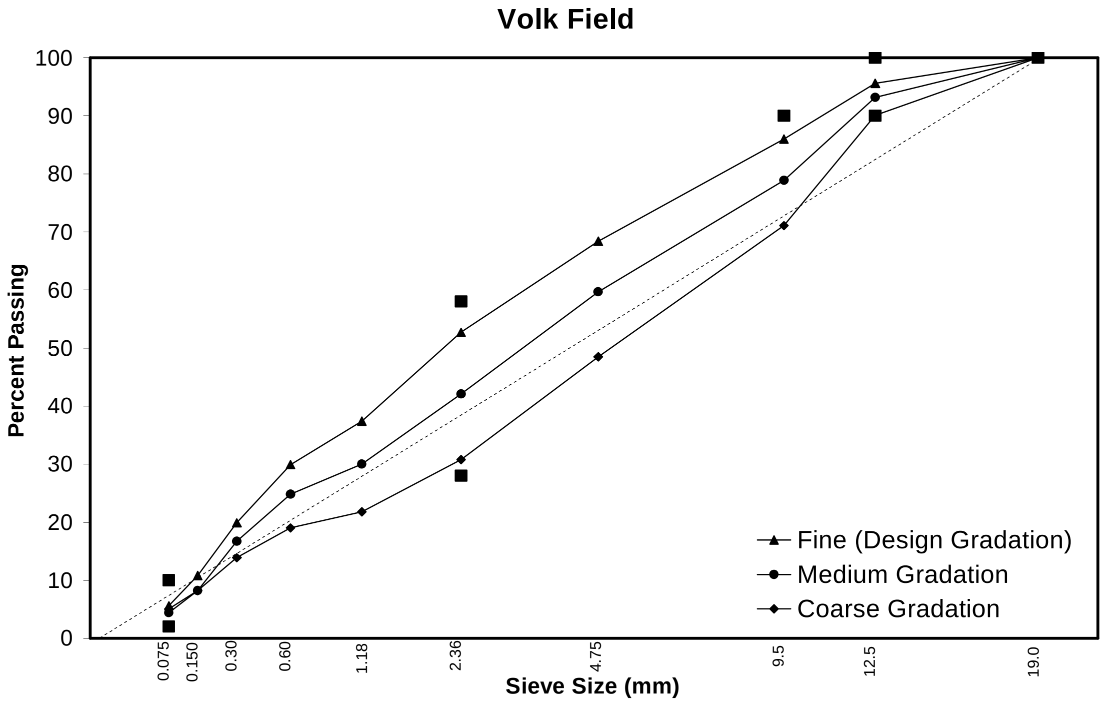

Figure 65: Gradations from VOK Used in Permeability Testing

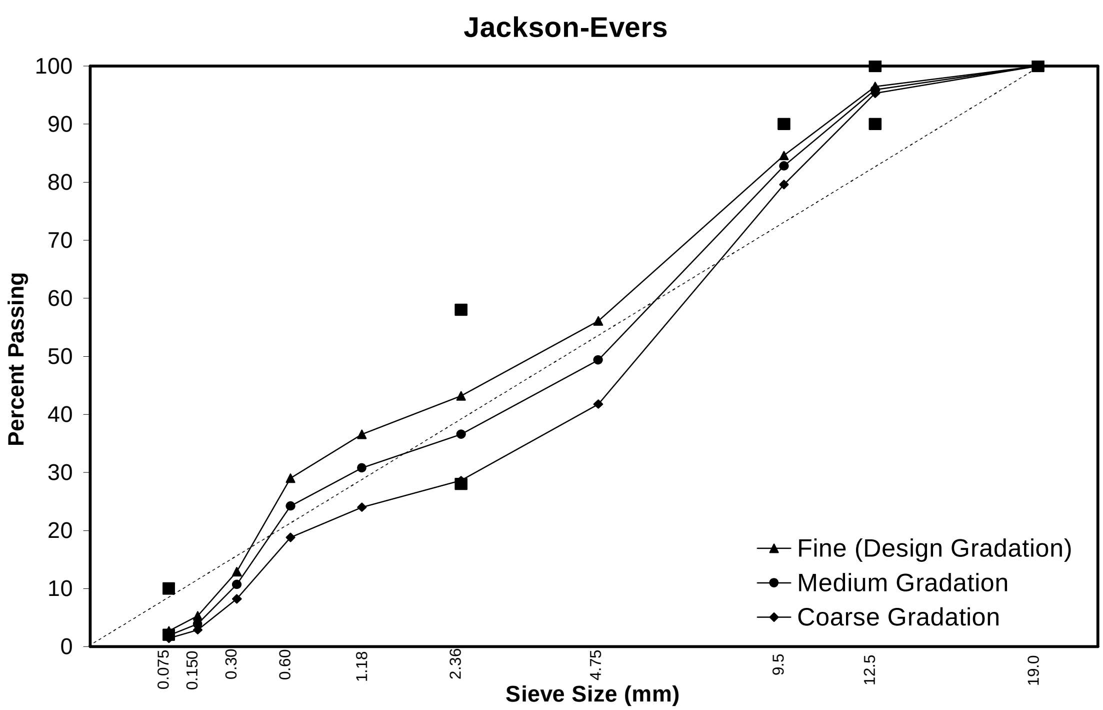

Figure 66: Gradations from JAN Used in Permeability Testing

The method for developing the relationship between permeability and density for the various mixes entailed first compacting mix in a Superpave gyratory compactor (SGC). Samples were compacted in the SGC to a height of 50 mm. In order to provide varying air void contents, the mass of mix placed in the molds was altered. Next, the samples were tested in the laboratory for permeability using a flexible wall, falling head permeameter. Results of permeability testing were then plotted versus the air void contents of the respective samples to develop the relationship between permeability and air voids (density).

Figures 67 through 71 illustrate the relationship between permeability and air void content for each of the mixes evaluated. Note that the scale for each of these figures were maintained the same so that the reader could visualize the relative amount of permeability exhibited by the mixes from figure to figure. Trend lines for each of the mixes are similar in that permeability values are relatively low at low air void contents and increase as the air void contents increase. Another trend in the data is that the mixes having coarser gradations are typically more permeable at a given air void content than mixes having finer gradations. These results are consistent with other research conducted using in-place permeability testing (9).

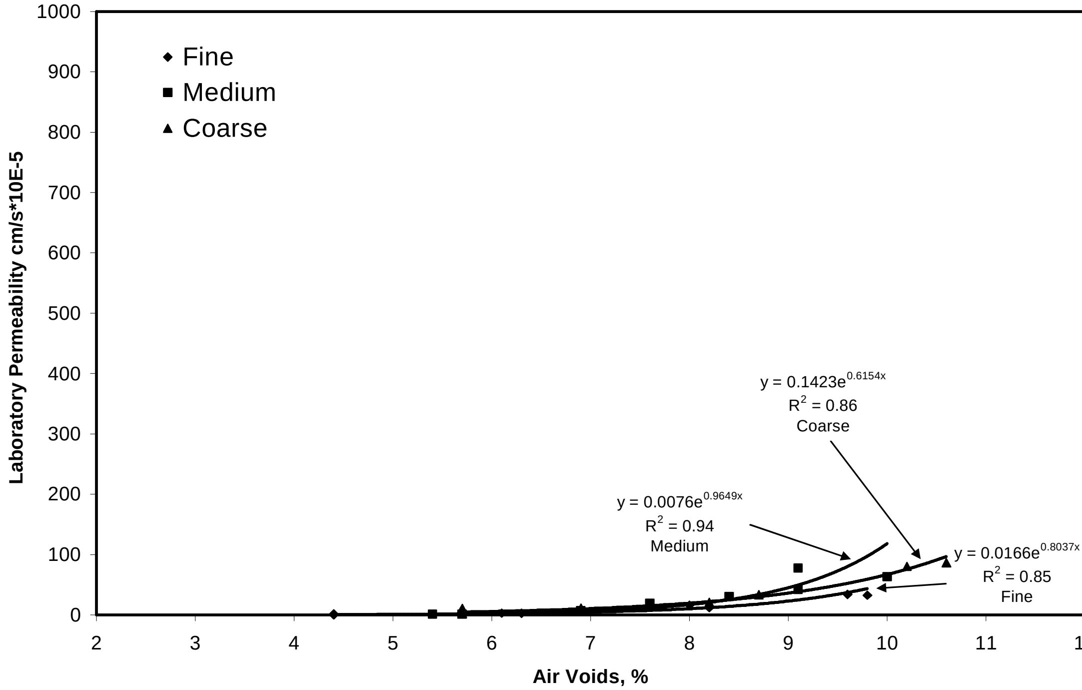

Figure 67: Relationship between Permeability and Air Voids - NTV

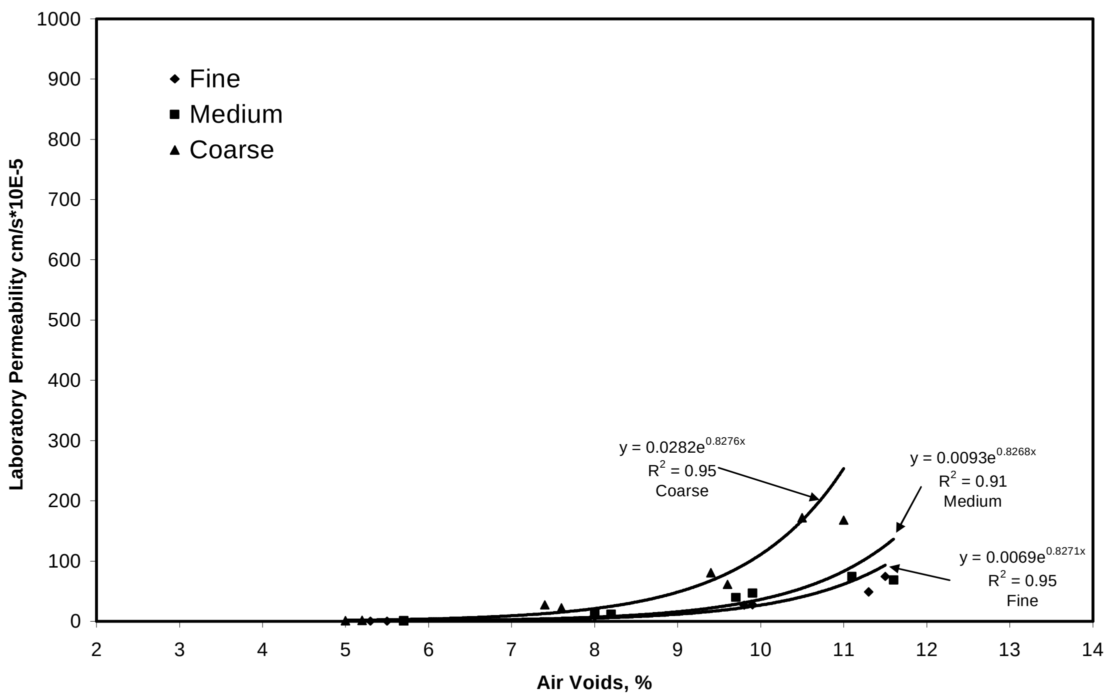

Figure 68: Relationship between Permeability and Air Voids - KHNZ

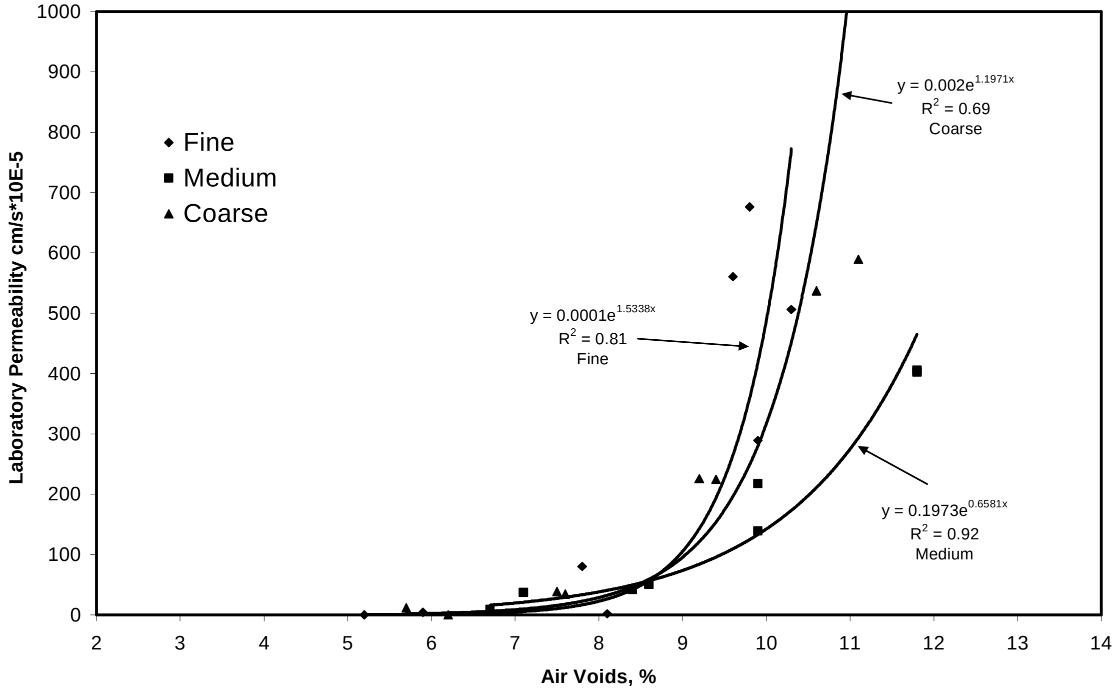

Figure 69: Relationship between Permeability and Air Voids – C24

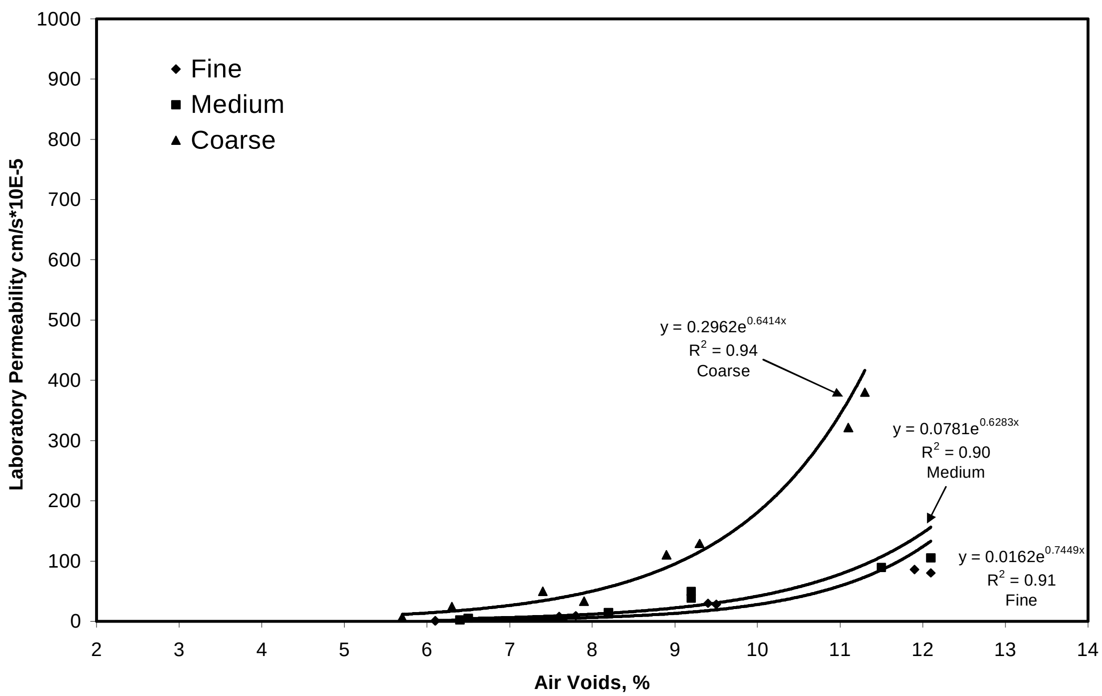

Figure 70: Relationship between Permeability and Air Voids - VOK

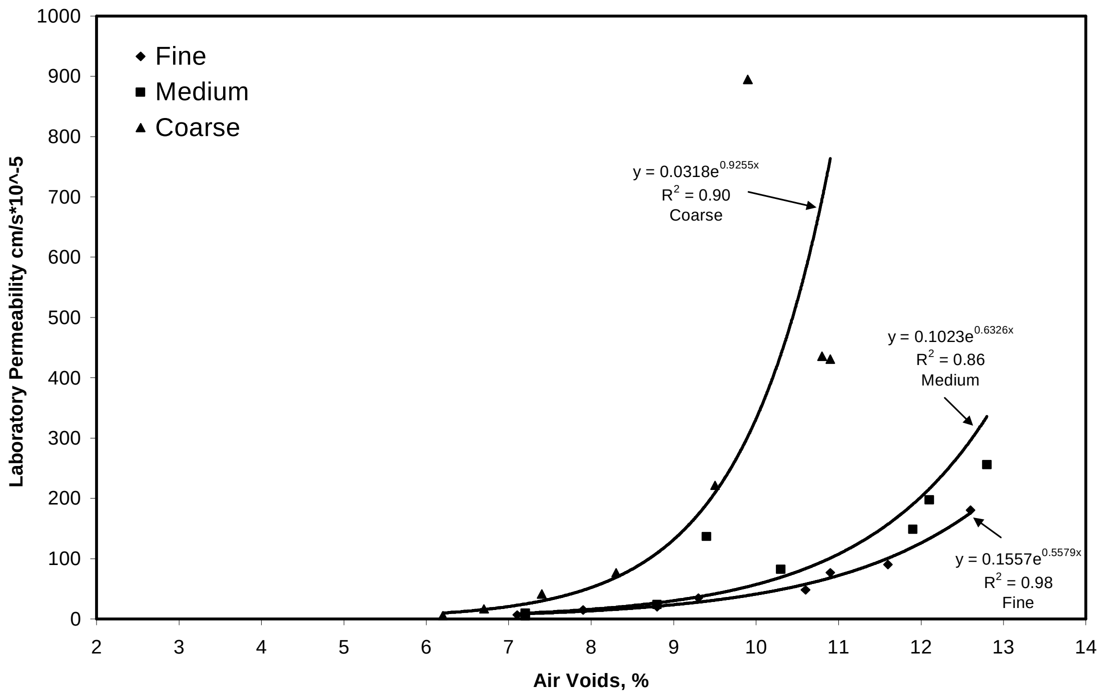

Figure 71: Relationship between Permeability and Air Voids - JAN

Figure 67 illustrates the relationships between permeability and air void content for the gradations evaluated for NTV. This figure shows that the permeability values are relatively low for all three gradations. These results are interesting in that NTV was the only 19.0 mm NMAS mix included in this experiment. Most research has shown that at a given air void content, larger NMAS gradations result in higher permeability values. However, referring back to Figure 62, this is not unexpected. All three gradations pass above the maximum density line. The finest gradation (and also design gradation) also passes above the upper control point for the specification from the highway version of the Superpave mix design system.

Figure 68 illustrates the relationships between permeability and air void content for the gradations evaluated from KHNZ. Gradations represented in this figure had a NMAS of 9.5 mm. Figure 68 shows that the coarsest gradation was the most permeable at a given air void content. The “medium” and “fine” gradations had relationships between permeability and air void content that were basically similar. Figure 63 showed that the “coarse” gradation passed near the lower control point of the highway Superpave gradation requirements. Both the “medium” and “fine” gradations passed near the maximum density line.

The relationships between permeability and air void content for the three gradations representing C24 are illustrated in Figure 69. Figure 64 showed that the C24 mixes also had a NMAS of 12.5 mm. Of the five airfields evaluated, the mixes developed for C24 had collectively the highest permeability values at a given air void content. Interestingly, all three gradations passed below the maximum density line at the No. 8 sieve.

The relationships between permeability and air voids for the three gradations representing VOK are illustrated in Figure 70. Gradations for VOK were a 12.5 mm NMAS. Figure 70 shows that at a given air void content, the “coarse” gradation had higher permeability than the medium and fine gradations. Figure 65 shows that the “coarse” gradation was the only one of the three that passed below the maximum density line at the No. 8 sieve. The “coarse” gradation also passed near the Superpave lower control point on the No. 8 sieve.

Figure 71 illustrates the relationship between permeability and air voids for the three gradations developed from materials obtained for JAN. These gradations have a 12.5 mm NMAS. Figure 71 shows that the “coarse” gradation was much more permeable at a given air void content than the other two gradations. Referring back to Figure 66, the “coarse” gradation passed near the lower Superpave control point on the No. 8 sieve, while the other two gradations were near the maximum density line or above on the No. 8 sieve.

Qualitatively, the results of this experiment suggest that coarser gradations lead to an increased potential for permeability problems within a constructed airfield pavement. In order to provide a quantitative evaluation of the influence of gradation on permeability characteristics, the relationships between permeability and air void content were utilized. For each of the relationships shown in Figures 67 through 71, permeability was calculated at 7, 8, 9 and 10 percent air voids. These values should represent the anticipated densities in a constructed airfield pavement assuming a normal distribution that is centered around 8 percent in-place air voids.

Previous research has shown that NMAS is a significant factor when considering the relationship between permeability and in-place density (9). Therefore, in order to evaluate the data, the permeability data was grouped by NMAS. Figures 72 through 74 present the permeability data for 9.5, 12.5 and 19 mm NMAS mixes, respectively. Within these figures, permeability is plotted versus the percent passing the No. 8 sieve for the respective gradations. Also included in each figure as vertical lines are the current Superpave control points for the No. 8 sieve.

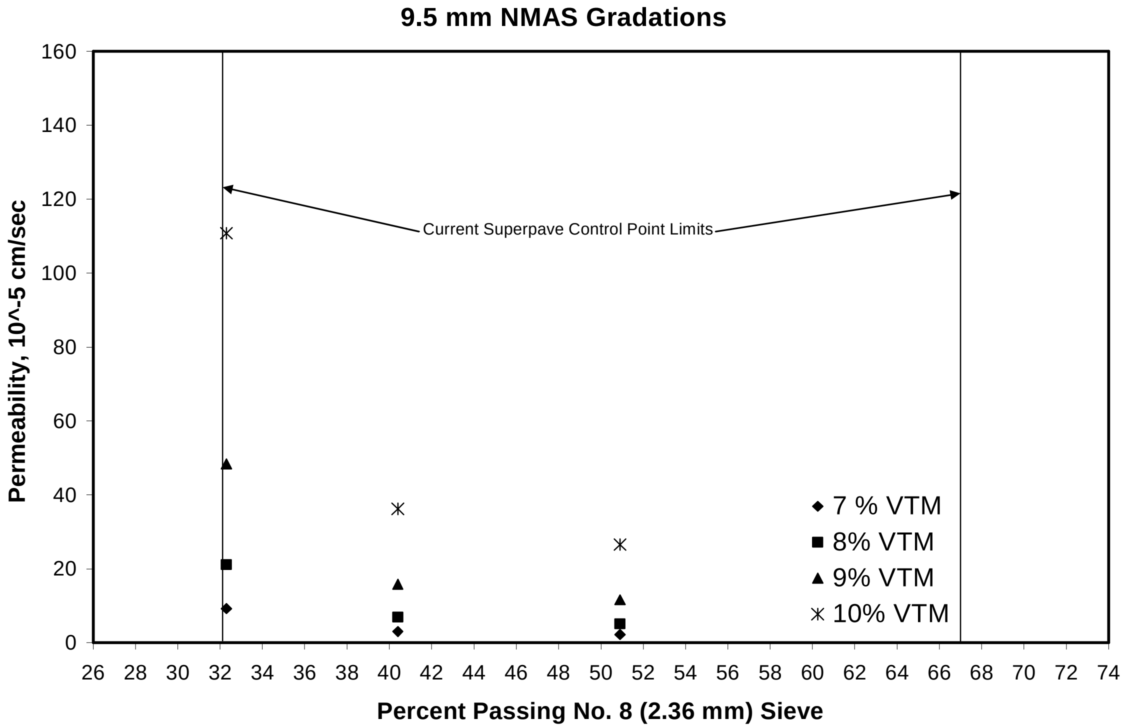

Figure 72: Relationship between Permeability and Percent Passing No. 8 Sieve - 9.5 mm NMAS

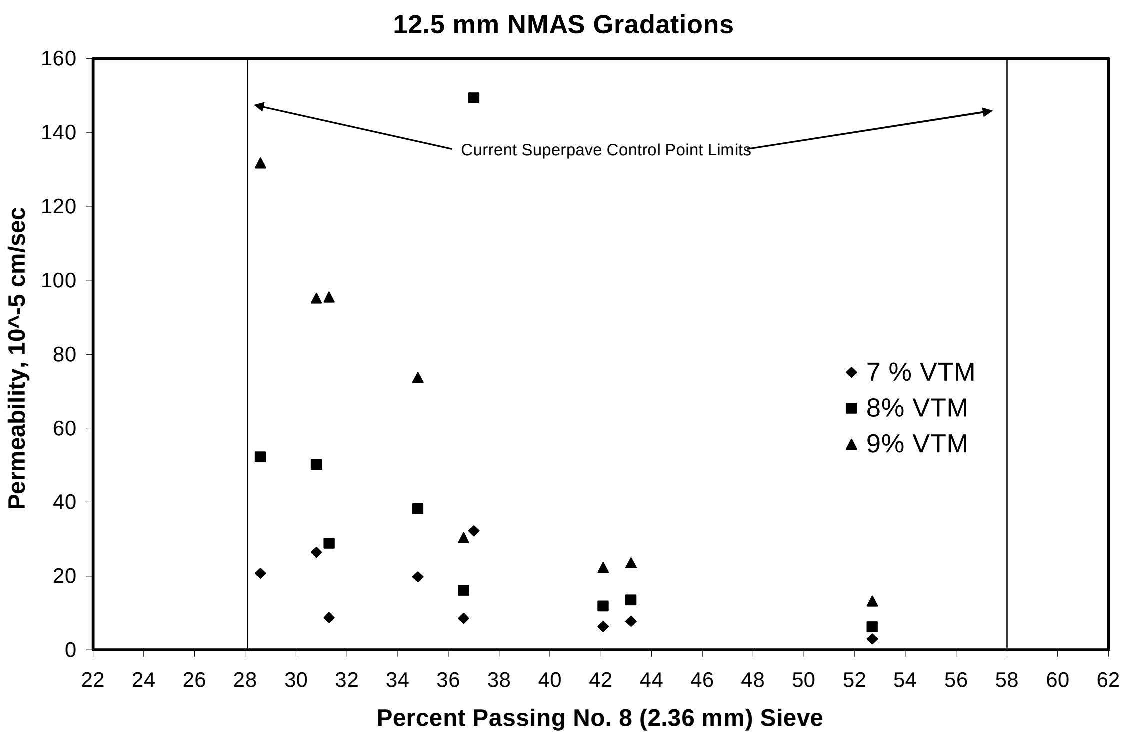

Figure 73: Relationship between Permeability and Percent Passing No. 8 Sieve - 12.5 mm NMAS

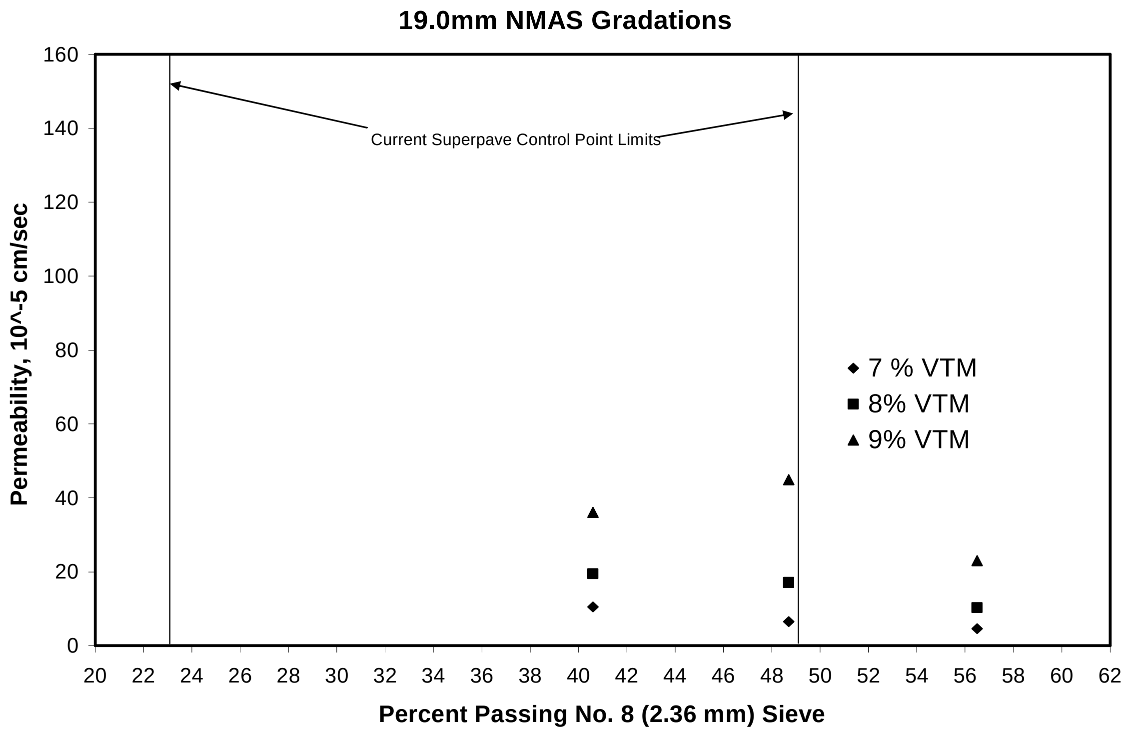

Figure 74: Relationship between Permeability and Percent Passing No. 8 Sieve - 19.0 mm NMAS

Figure 72 presents the calculated permeability values versus percent passing the No. 8 sieve for the lone 9.5 mm NMAS gradation size (KNHZ). This figure shows that permeability values are all relatively low; however, the permeability values begin to increase once the percent passing the No. 8 sieve is below approximately 40 percent. For the 9.5 mm NMAS gradation requirements, the Superpave control limits are at 32 and 68 percent. Based upon Figure 72, the potential for permeability at typical in-place air voids would increase for 9.5 mm NMAS mixes having gradations with the percent passing the No. 8 sieve below approximately 40 percent.

Figure 73 illustrates the relationship between permeability and percent passing the No. 8 sieve for 12.5 mm NMAS gradations. Three of the five mixes included within this experiment met a 12.5 mm NMAS including C24, VOK, and JAN. Because three airfields had a 12.5 mm NMAS, Figure 73 contains more data than did the 9.5 mm NMAS mixes or 19.0 mm NMAS mixes. Figure 73 shows that permeability values begin to increase once the gradation becomes coarser than approximately 36 percent on the No. 8 sieve. At 36 percent passing the No. 8 sieve, all permeability values are below 40×10^{-5} cm/sec. At percent passing the No. 8 sieve values below 36 percent, the permeability begins to increase. For 12.5 mm NMAS gradations, the Superpave control points are at 28 and 58 percent. Figure 73 suggests that the minimum percent passing the No. 8 sieve should be approximately 34 to 36 percent in order to minimize the potential for permeability problems in the field.

Figure 74 illustrates the relationship between permeability and the percent passing the No. 8 sieve for the lone 19.0 mm NMAS gradation size mixes (NTV). As shown on the figure, permeability values were all relatively low for these mixes. However, this is not unexpected (as discussed above), because all three of the gradations evaluated were very fine.

In summary, the data and discussion provided suggest that mixes having coarser gradations do have greater potential for being permeable. The data suggests that the lower control point for the highway version of the Superpave mix design system should be increased in order to minimize the potential for permeable pavements. This is important because durability is a major concern on airfield pavements. Pavements that are permeable have an increased potential for cracking, raveling and moisture damage. Based on the data, for HMA designed for airfield pavements using the Superpave mix design system, the lower control points should be increased by 5 percent on the No. 8 sieve.

Table 35 presents Superpave gradation requirements if the lower control point on the No. 8 sieve is increased by 5 percent. Figures 75 through 78 present these revised Superpave gradation control points compared to the current P-401 and UFGS-32 12 15 gradation requirements. As shown in these figures, by increasing the lower control point on the No. 8 sieve by 5 percent, the revised Superpave gradation requirements very closely match the current P-401 and UFGS-32 12 15 gradation requirements on the No. 8 sieve. Therefore, there is no reason to change the current airfield HMA gradation requirements when designing airfield HMA mixes using Superpave methods. However, the researchers have consolidated the Item P-401 and UFGS-32 12 15 gradation requirements in order to provide a single gradation band for each maximum aggregate size gradation. The recommended gradation requirements are presented in Table 36.

Table 35: Gradation Control Points for Airfield Superpave Mixes

Sieve Size, inch (mm)	Nominal Maximum Aggregate Size – Control Points (Percent Passing)											
	37.5 mm		25.0mm		19.0mm		12.5mm		9.5mm		4.75mm	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
2.0 (50.0)	100	---	---	---	---	---	---	---	---	---	---	---
1.5 (37.5)	90	100	100	---	---	---	---	---	---	---	---	---
1.0 (25.0)	---	90	90	100	100	---	---	---	---	---	---	---
¾ (19.0)	---	---	---	90	90	100	100	---	---	---	---	---
1/8 (12.5)	---	---	---	---	---	90	90	100	100	---	100	---
3/8 (9.5)	---	---	---	---	---	---	---	90	90	100	95	100
No. 4 (4.75)	---	---	---	---	---	---	---	---	---	90	90	100
No. 8 (2.36)	20	41	24	45	28	49	33	58	37	67	---	---
No. 16 (1.18)	---	---	---	---	---	---	---	---	---	---	30	60
No. 200(0.075)	0	6	1	7	2	8	2	10	2	10	6	12

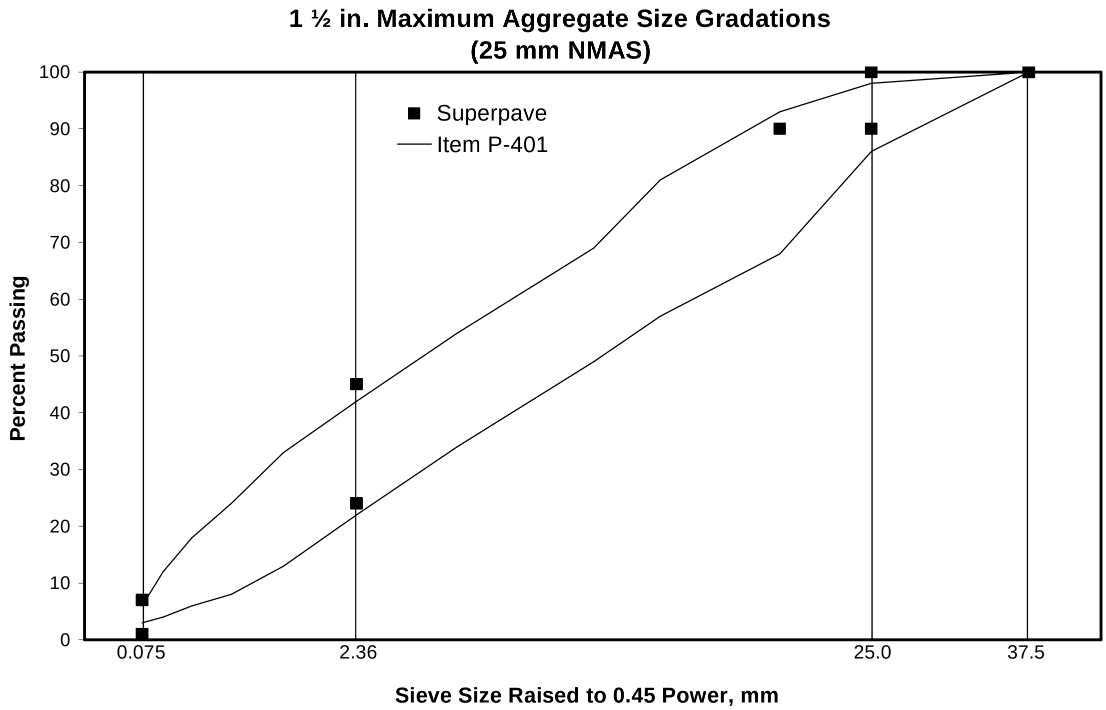

Figure 75: Revised Gradations for 1.5 in. Max. Aggregate Size Gradations

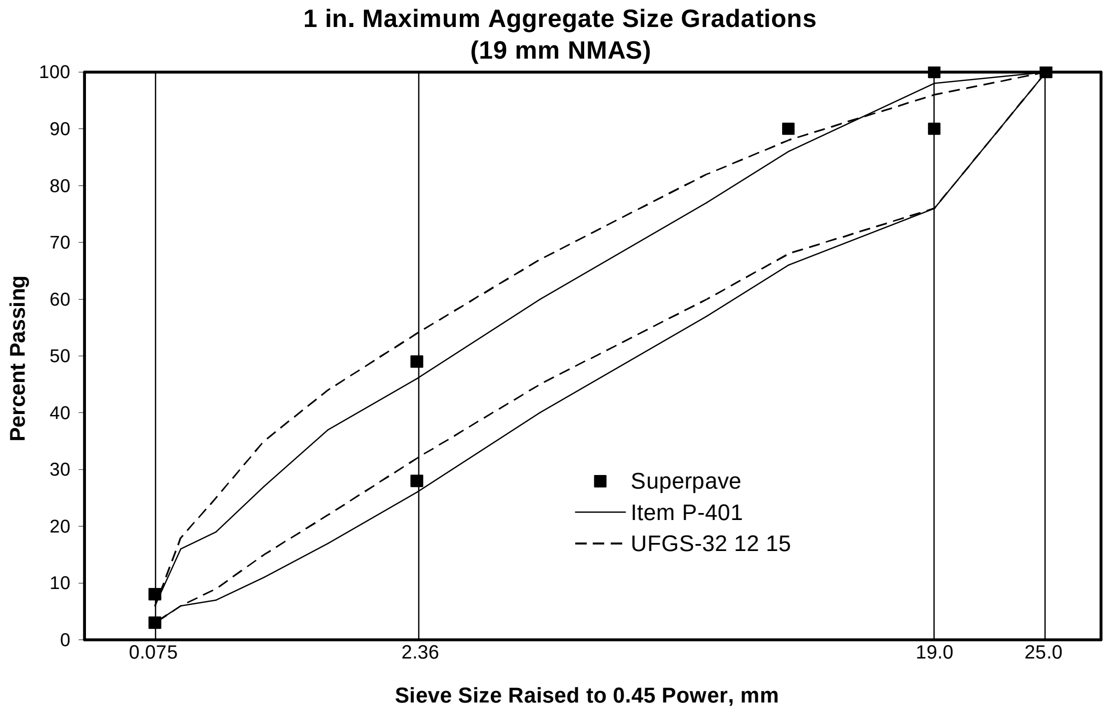

Figure 76: Revised Gradations for 1 in. Max. Aggregate Size Gradations

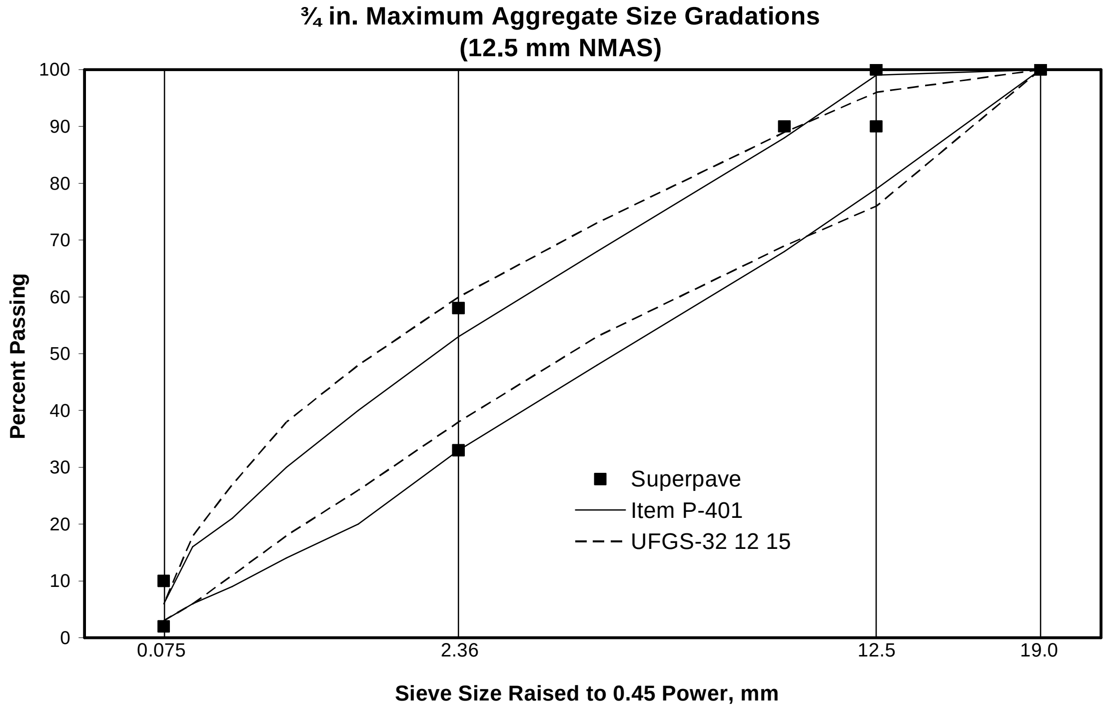

Figure 77: Revised Gradations for 3/4 in. Max. Aggregate Size Gradations

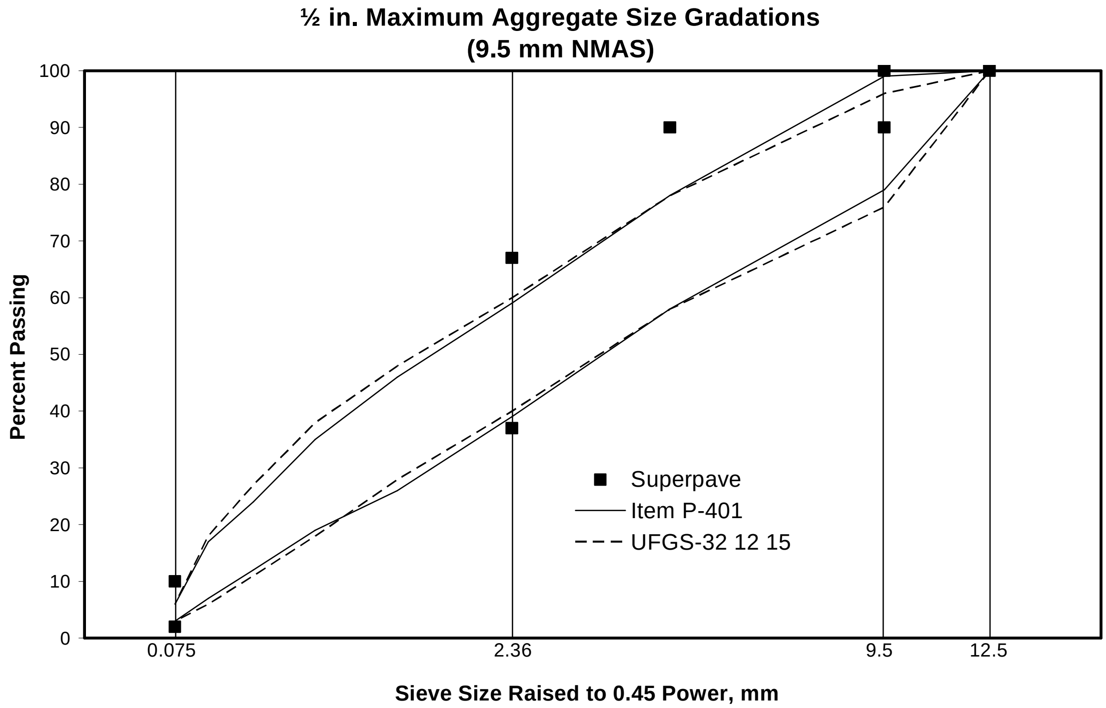

Figure 78: Revised Gradations for 1/2 in. Max. Aggregate Size Gradations

Table 36: Recommended Gradation Requirements for Superpave Designed Airfield HMA

Sieve Size	Percentage by Weight Passing Sieves			
U.S. (mm)	1½"max	1" max	3/4" max	½" max
1-1/2 (37.5)	100	---	---	---
1 (25.0)	86-98	100	---	---
¾ (19.0)	68-93	76-97	100	---
½ (12.5)	57-81	67-87	77-98	100
3/8 (9.5)	49-69	58-80	68-89	77-98
No. 4 (4.75)	34-54	42-62	50-70	58-78
No. 8 (2.36)	22-42	29-48	35-55	40-60
No. 16 (1.18)	13-33	19-40	23-34	27-47
No. 30 (0.600)	8-24	12-30	16-34	18-36
No. 50 (0.300)	6-18	8-22	12-28	11-25
No. 100 (0.150)	4-12	6-17	7-20	6-18
No. 200 (0.075)	3-6	3-6	3-6	3-6

MATERIAL REQUIREMENTS

Materials used in the design of dense-graded HMA include coarse aggregates, fine aggregates, asphalt binder, and other materials that may be required to meet the mix design specifications. No specific research was conducted as part of this study to evaluate the influence of material properties on HMA performance. However, a significant effort was made within Chapter 3 of this report to compare the historical airfield mix design methods with the highway version of the Superpave mix design system. These comparisons suggested that the aggregate quality characteristics were actually similar between the historical airfield mix design methods and the Superpave mix design system. For coarse aggregates, all three mix design methods had criteria for aggregate angularity, and shape. The primary difference was that the two airfield mix design methods had criteria for coarse aggregate toughness, soundness and cleanliness. The Superpave mix design system allows individual agencies (or states) to develop the criteria for these three aggregate quality characteristics.

Fine aggregate quality characteristics were also similar. All three mix design methods have requirements for the angularity and cleanliness of the fine aggregates. Item

P-401 does have requirements for toughness and soundness, while UFGS-32 12 15 and Superpave do not. Table 37 presents a summary of all of the aggregate characteristics specified in the three mix design methods along with the test methods used to specify these characteristics.

Table 37: Summary of Aggregate Quality Characteristics

Agg. Qual. Characteristic	P-401		UFGS-32-12-15		Superpave	
	Coarse Agg.	Fine Agg.	Coarse Agg.	Fine Agg.	Coarse Agg.	Fine Agg.
Angularity	Fractured Faces	Max. % Nat. Sand	Fractured Faces	Max. % Nat. Sand & Uncomp. Voids	Fractured Faces	Uncomp. Voids
Shape	Flat, Elongated & Flat and Elongated	---	Flat and Elongated	---	Flat and Elongated	---
Toughness	LA Abrasion	LA Abrasion, parent agg.	LA Abrasion	---	Ind. Agency	---
Soundness	Sulfate	Sulfate, parent agg.	Sulfate	---	Ind. Agency	---
Cleanliness	Deleterious Materials	Atterberg Limits	Deleterious Materials	Sand Equivalency	Ind. Agency	Sand Equivalency

Though the aggregate quality characteristics specified between the three mix design methods are similar, the specification values do differ. This is specifically true for the coarse aggregate angularity. Item P-401 includes two specification values for the percent fractured faces: one for HMA being designed to carry aircraft with gross weights greater than 60,000 lbs and one for those airfields that will carry lighter aircraft. As would be expected, the more stringent specification value is for the airfield pavements that will carry the heavier aircraft. The specification value for these airfields is a minimum of 70 percent of coarse aggregates have two or more fractured faces, while the minimum percent two or more fractured faces for the lighter aircraft is 50 percent. UFGS-32 12 15 contains only a single requirement for two or more fractured faces which is a minimum of 75 percent. Conversely, Table 4 previously showed that the specified aggregate requirements within the Superpave mix design method are based upon anticipated traffic and tend to be higher than the two historical airfield mix design methods. At very low highway traffic levels, the Superpave requirements are similar to the two airfield requirements in that the minimum percent of coarse aggregate particles with one fractured face is 55 and 70 percent. However, at medium traffic levels, the fractured face percentages increase to 85 percent with one fractured face and 85 percent with two or more fractured faces (85/80). This requirement is more stringent than either of the two airfield mix design methods. As traffic increases, the angularity specification values increase to 95/90 for high traffic and 100/100 and very high traffic.

Requirements for the fine aggregates are generally very similar between the three mix design methods. The primary difference is that the two historical airfield mix design methods have requirements for a maximum percentage of natural sand, while the Superpave method does not. Uncompacted voids in the fine aggregate is specified within the Superpave method. UFGS-32 12 15 actually specified both a maximum percentage of natural sand and the uncompacted voids in the fine aggregates.

The toughness and soundness aggregate quality characteristics are included within both airfield mix design methods, while within the Superpave method it is considered a source property in which individual agencies develop specification values. However, notes within both Item P-401 and UFGS-32 12 15 state the Engineer can allow aggregates that don't meet the toughness requirements if there is a history of the aggregate source performing well within pavements.

The final aggregate quality characteristic included within three mix design methods is cleanliness. Item P-401 utilizes Atterberg limits while UFGS-32 12 15 and Superpave utilize sand equivalency.

As stated previously, no specific research was conducted to evaluate the aggregate quality characteristics during this project. However, any mix design system needs aggregate quality requirements in order to provide a quality HMA. Therefore, recommendations were developed for aggregate quality. Table 38 presents the recommended aggregate specification values for the design of airfield HMA using the Superpave method. This table purposefully does not include requirements for toughness and soundness as these will be maintained within the recommended guide specification in the same method as currently specified.

Table 38: Aggregate Requirements for Airfield Superpave Design HMA

N_{design}	Min. % Fractured Faces*	Uncomp. Voids of Fine Agg., % Min.	Max. % Natural Sand	Max. % Flat and Elongated Particles (5:1)	Min. Sand Equivalency
50	85/80	40	20	10	40
65	95/90	45	15	10	40
80	95/95	45	15	10	50

Commercial mineral fillers added to an HMA are addressed similarly within both historical airfield mix design methods. Both state that any commercial mineral fillers should meet the requirements of ASTM D242. This requirement will not be changed.

The final material that needs selection is the asphalt binder. AAPTP Project 04-02 was specifically conducted to recommend asphalt binder properties for airfield pavements.

SELECTION OF OPTIMUM ASPHALT BINDER CONTENT

All three mix design methods rely on volumetric properties to select the optimum asphalt binder content during design. The volumetric properties of air voids, VMA, and VFA are included within all three design methods. Item P-401 and UFGS-32 12 15 both allow the designer to select the optimum asphalt binder content based upon a range of air voids, while the current Superpave mix design system requires selection of optimum asphalt content at 4.0 percent voids.

A volumetric property in which there are differences between the two airfield mix design methods is VMA. Item P-401 requires 1 percent higher VMA for a given maximum aggregate size gradation compared to UFGS-32 12 15. The VMA requirements in the Superpave mix design method matches UFGS-32 12 15.

No specific research was conducted in order to select appropriate volumetric properties; therefore, similar to the aggregate properties, the researchers are recommending volumetric properties based upon experience. Table 39 presents the recommended volumetric criteria for designing airfield HMA using the Superpave gyratory compactor. Within this table, optimum asphalt content is selected based upon the same volumetric properties as outlined in all three mix design methods: air voids, VMA, and VFA. A single design air void content of 4 percent was selected. This air void content is consistent with the current Superpave mix design system. The recommended VMA values are consistent with the values currently specified within UFGS-32 12 15 and the Superpave mix design methods. Voids filled with asphalt values are based upon the VMA and air void criteria. Also contained within Table 39 is a volumetric requirement for the percent theoretical maximum density at the initial number of gyrations. The initial number of gyrations for the less than 100, 100 to 200 and greater than 200 psi tire pressure categories are 6, 7 and 7, respectively. The final specification values show within Table 39 are for dust-to-binder ratio. This is calculated by dividing the percent minus No. 200 from the gradation (percent by mass) by the effective asphalt binder content of the mix.

Table 39: Volumetric Properties For Selecting Optimum Asphalt Binder

Tire Pressure, psi	N_{design}	Required Relative Density, Percent of Theoretical Maximum Specific Gravity		Voids in the Mineral Aggregate (VMA), Percent Minimum Maximum Aggregate Size, mm				Voids Filled with Asphalt (VFA) Range, Percent	Dust-to-Binder Ratio Range
		$N_{initial}$	N_{design}	1 ½	1	¾	½		
<100	50	≤90.5	96.0	12.0	13.0	14.0	15.0	70-80	0.6-1.2
100 to 200	65	≤90.5	96.0	12.0	13.0	14.0	15.0	65-78	0.6-1.2
>200	80	≤89.0	96.0	12.0	13.0	14.0	15.0	65-75	0.6-1.2

PERFORMANCE TESTING

Again, no specific research was conducted within this study to develop an appropriate performance test for HMA mixes design in accordance with the airfield Superpave mix design method. Moisture susceptibility testing should be conducted in order to evaluate the potential for moisture damage in the designed mix. ASTM D4867 should be used for this testing. A minimum tensile strength ratio of 80 percent is recommended for samples prepared with the Superpave gyratory compactor.

SHADOW SPECIFICATIONS

The final task conducted as part of this research was to utilize the Superpave mix design method as a shadow specification at two airfield construction projects. For this testing, plant mix HMA was used to compact specimens utilizing the Marshall hammer and the Superpave gyratory compactor in order to compare the variability of plant produced mixtures using the two compaction methods.

Two ongoing airfield construction projects were included in this phase of work. The HMA from Case Study No 1 was designed using 75 blows per face of the Marshall hammer while the mixture from Case Study No. 2 was designed using 100 gyrations of the Superpave gyratory compactor.

For each of the airfields, the first step in evaluating the Superpave method within the shadow specification was to develop equivalent compactive efforts. In other words, the researchers determined the number of gyrations by the Superpave gyratory compactor that were equal to the appropriate Marshall blows per face. In both instances, the appropriate Marshall design compactive effort was 75 blows. Following the identification of the equivalent compactive efforts, specimens were compacted for numerous lots in order to evaluate the variability in mix properties using the two compaction methods.

Shadow Specification at Case Study No. 1

The equivalent gyration level for the Case Study No. 1 mix was 50 gyrations. A total of 28 sampling times were compared. For each of these plant produced samples, companion materials were compacted using 75 blows per face and 50 gyrations. Two specimens were compacted using the Superpave gyratory compactor and three specimens were compacted using the Marshall hammer. For each specimen compacted, the bulk specific gravity was determined and volumetric properties calculated.

Figure 79 presents the bulk specific gravity of the compacted specimens in the form of a control chart. Both the Superpave gyratory compactor and Marshall hammer data are depicted on this figure. This figure shows that the bulk specific gravity resulting from the two compaction method values tracked each other well. Also included on this figure are the pooled standard deviations for compaction of the samples. Recall that three samples were compacted with the Marshall hammer and two samples were compacted with the Superpave gyratory compactor. As shown by the pooled standard deviations, the variability in test results were similar between the two compactive efforts even though an extra sample was utilized when compacting with the Marshall hammer.

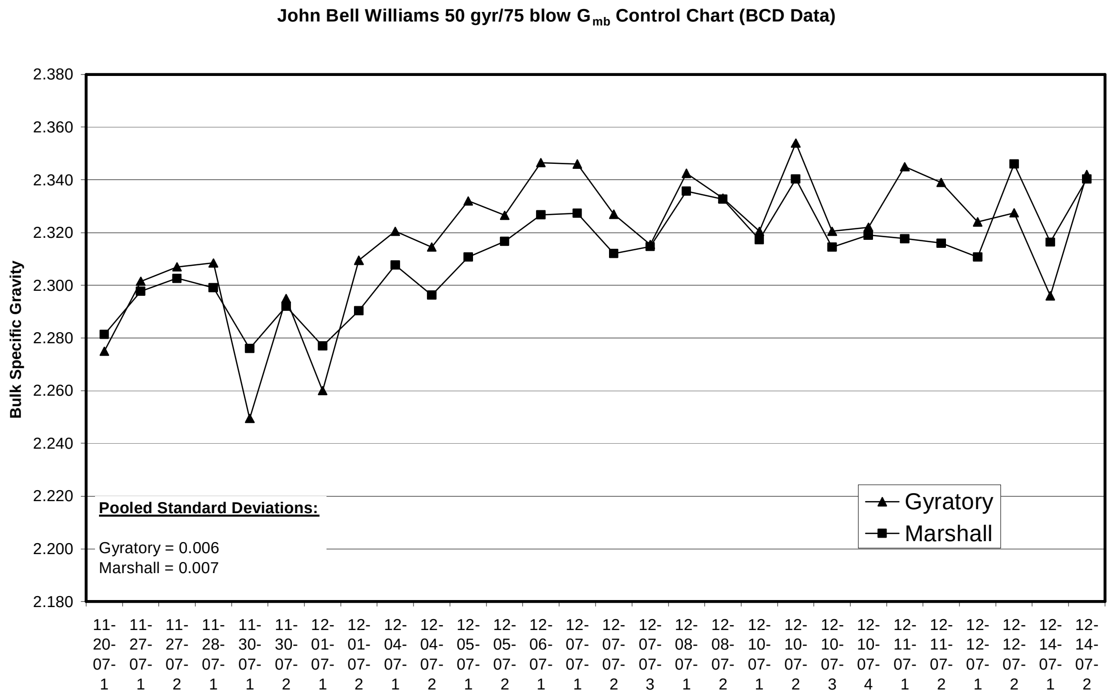

Figure 79: Bulk Specific Gravity Data for Companion Samples – Case Study No. 1

Figure 80 illustrates the air void contents of the samples in the form of a control chart. Also included on this figure are the allowable limits for air voids contained within Item P-401 which are 2 to 5 percent. Figure 80 shows that for the most part, the air void contents of the companion specimens tracked each other well, as would be expected since the bulk specific gravity values tracked well. There are instances where samples prepared with the Superpave gyratory compactor exceeded the allowable limits, while there are also instances where the samples compacted with the Marshall hammer exceeded the allowable limits.

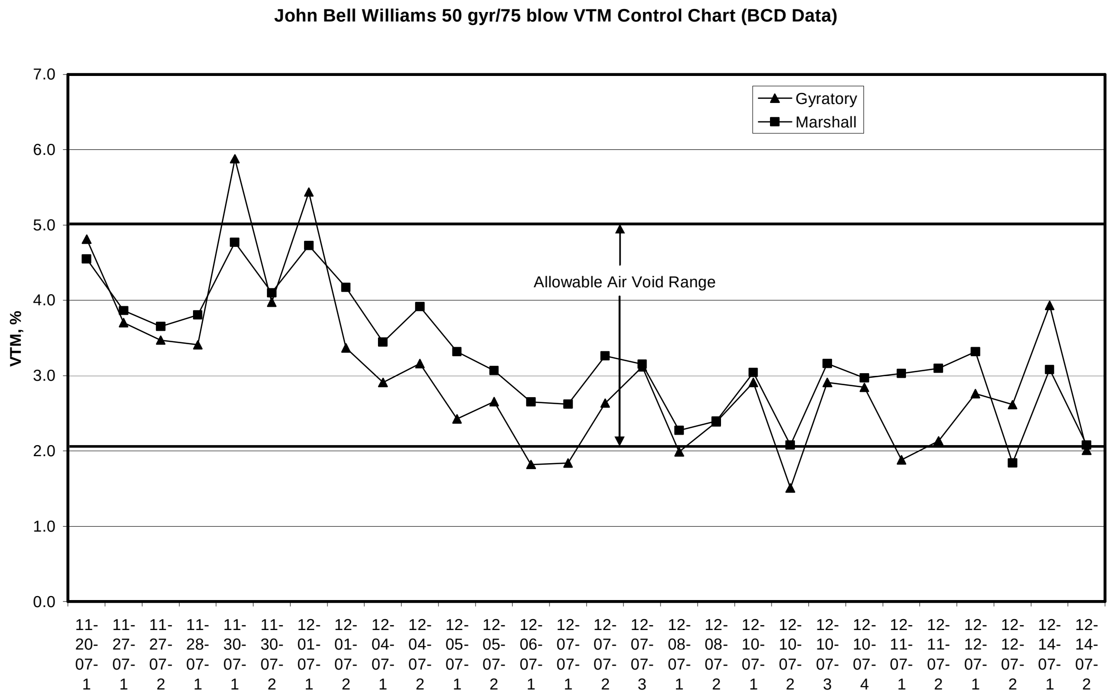

Figure 80: Control Chart for Air Voids – Case Study No. 1

Another important volumetric property is voids in mineral aggregate (VMA). Figure 81 presents the companion VMA data in the form of a control chart. Similar to the previous two figures, VMA values resulting from the two compactive efforts appear to track well. Also included on this figure is the minimum VMA requirement during design. The figure shows that all but a few of the data points fall below the minimum design VMA requirement. This is not unexpected as VMA is generally reduced going through the HMA plant during production. However, some of the VMA values are more than 2 percent below the minimum design value. Unfortunately, VMA alone can not provide an indication of whether the proper volume of asphalt binder is included within the mix, especially when air void contents vary during production.

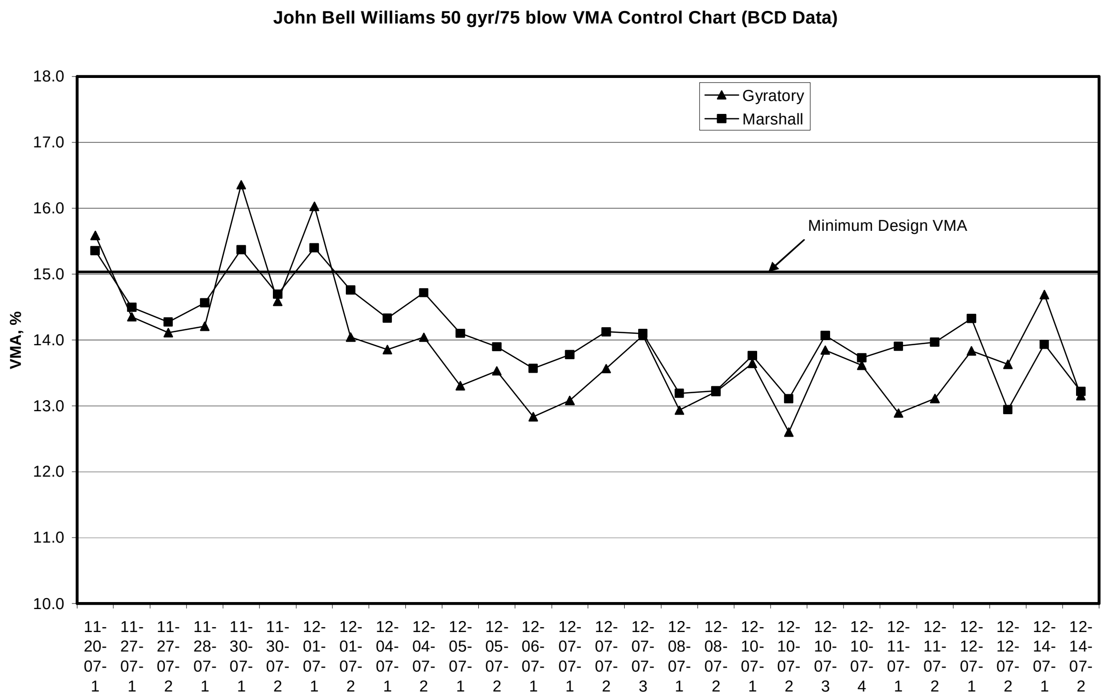

Figure 81: Control Chart for VMA – Case Study No. 1

In order to evaluate whether the appropriate volume of asphalt binder was being placed within the mixes, a control chart for the volume of effective asphalt (VEA) was developed. VEA is the numerical difference between VMA and VTM and defines the percent volume of effective asphalt binder within the mix. Figure 82 presents the VEA data for John Bell Williams Airport in the form of a control chart. Also included on this figure are the minimum VEA requirements. These values are based upon the minimum VMA requirement of 15 percent and allowable design air void content range. This figure shows that samples during the initial portion of production had slightly less volume of asphalt binder than desirable. Evaluating Figures 80 and 81, the air void contents were relatively high during this portion of production, while VMA values were near the minimum requirement. Interestingly, in the last portion of production, VMA values were well below the minimum design value but the VEA was within the design range. This was the result of air voids being toward the lower end of the allowable limits as shown in Figure 80. This suggests that VEA may be a better indicator of durability than VMA during production.

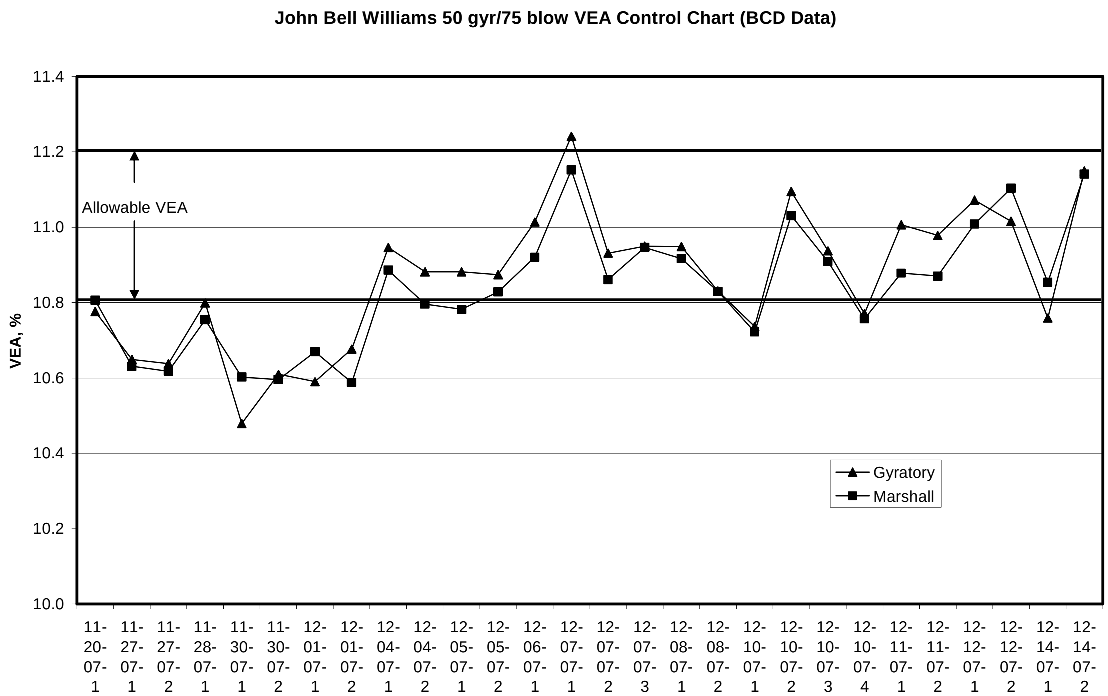

Figure 82: Control Chart for VEA – Case Study No. 1

Shadow Specification at Case Study No. 2

The equivalent gyration level (to 75 blows per face) for the Case Study No. 2 was 65 gyrations; however, samples were also compacted at 100 gyrations as this was the design compactive effort utilized during mix design. Therefore, samples were compacted at 65 and 100 gyrations using the Superpave gyratory compactor and 75 blows per face of the Marshall hammer.

Figure 83 presents the bulk specific gravity of the compacted specimens when compacted at 65 gyrations and 75 blows per face. These two compactive efforts are presented together because they were determined to provide similar compactive efforts for this particular HMA. This figure shows that the bulk specific gravities were comparable during construction. A single data point from October 30th does show slightly less compaction from the Superpave gyratory compactor. Also included within this figure are the pooled standard deviations for the bulk specific gravity samples. Again, the pooled standard deviations were very similar with the Superpave gyratory compactor sample showing slightly less variability.

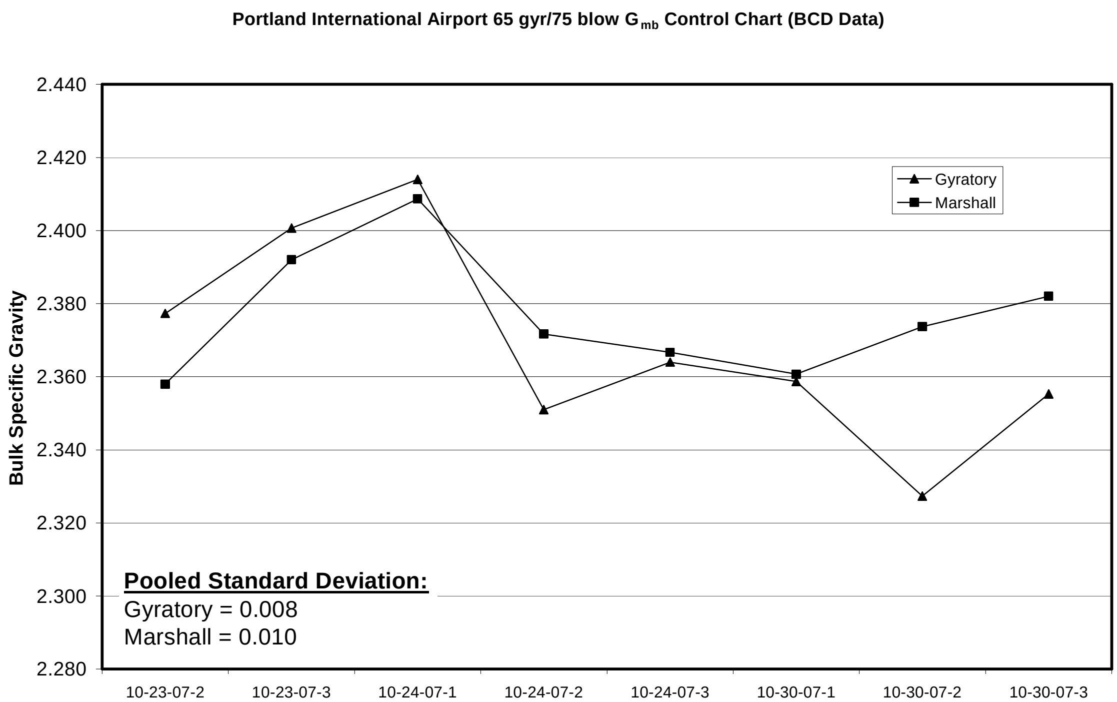

Figure 83: Control Chart for Bulk Specific Gravity (65 gyr/75 blows)- Case Study No. 2

Figures 84 through 86 present the VTM, VMA and VEA data in the form of control charts except for the single data point on October 30th. These figures also show that the two compactive efforts track each other well. No allowable limits are provided on these figures because the design compactive effort (100 gyrations) was different from both compactive efforts shown in the figures.

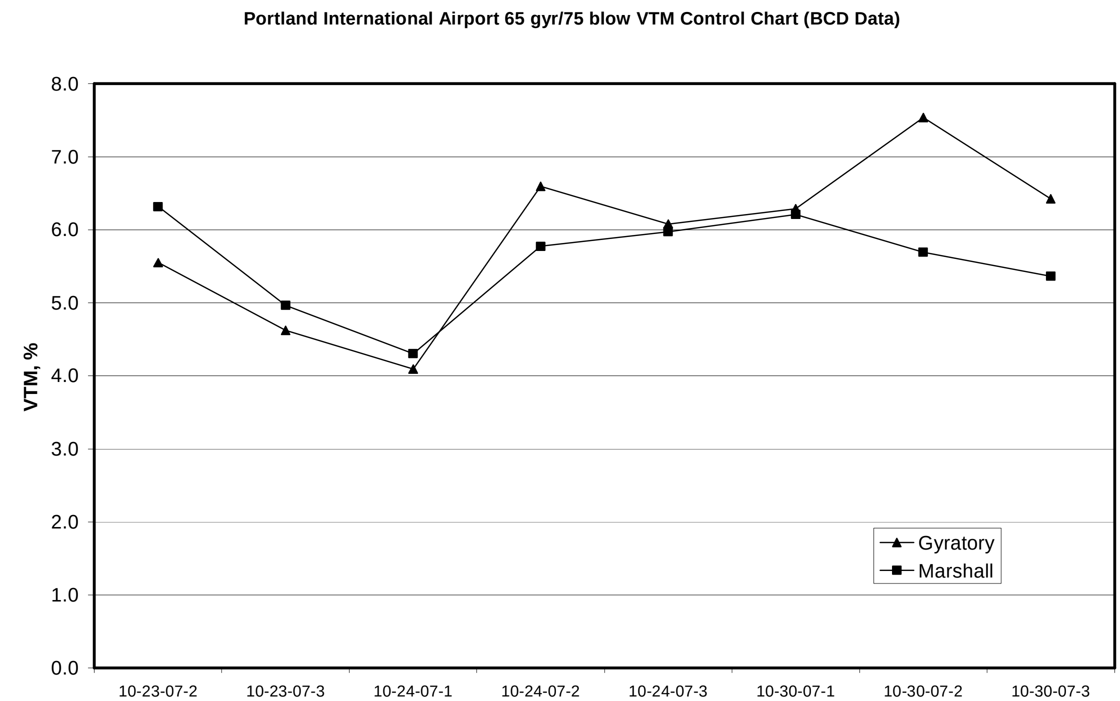

Figure 84: Control Chart for VTM (65 gyr/75 blows) – Case Study No. 2

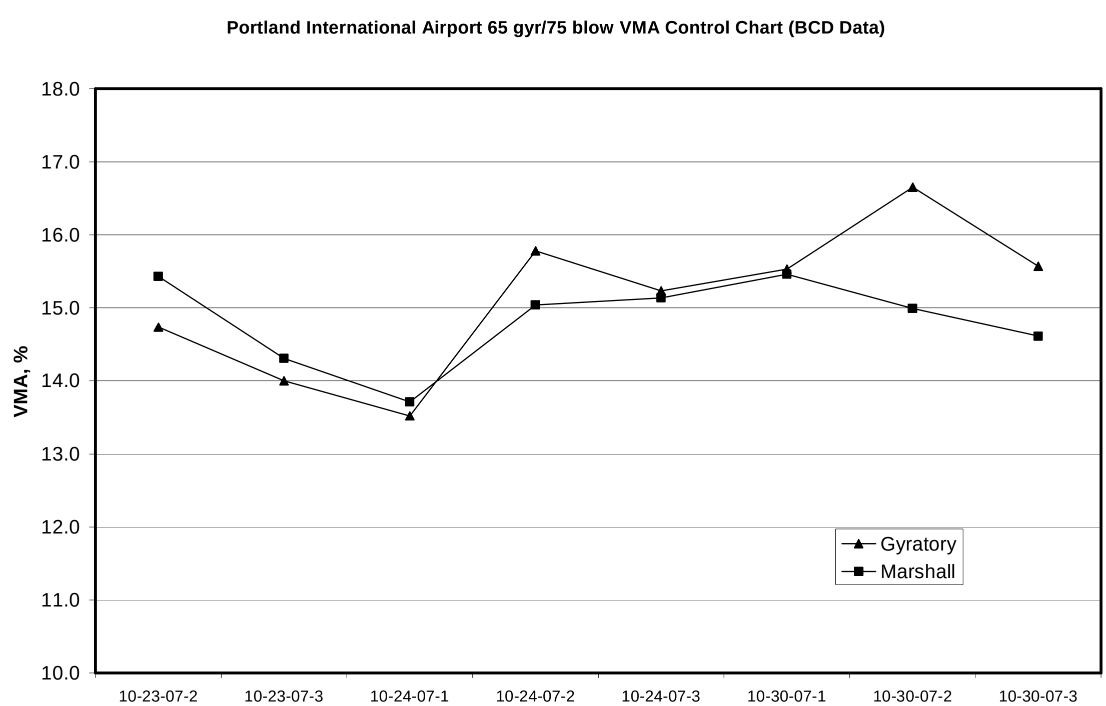

Figure 85: Control Chart for VMA (65 gyr/75 blows) – Case Study No. 2

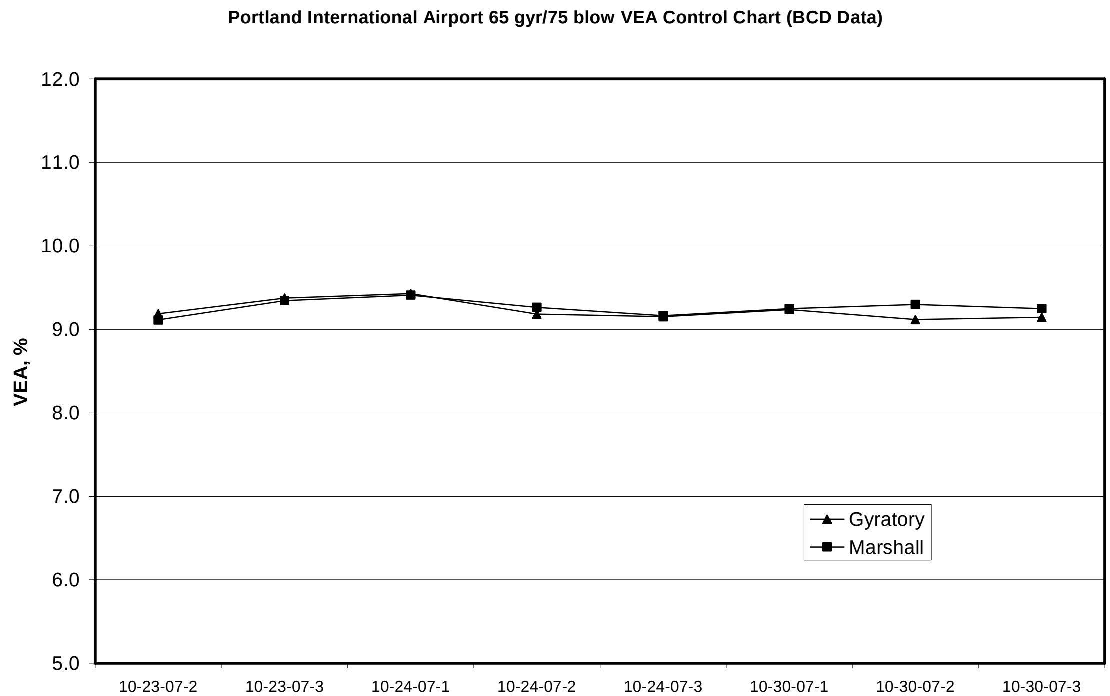

Figure 86: Control Chart for VEA (65 gyr/75 blows) – Case Study No. 2

Figures 87 through 89 present the data accumulated for the different sampling times when samples were compacted to 100 gyrations. Also included on these figures are the design values. All three figures suggest that the mixture was controlled closely to the design values during production. Air void contents ranged from approximately 3 to 5 percent. VMA values were near the minimum design values, with two data points having slightly lower VMA values than the design minimum. All of the VEA values plotted above the minimum design requirement.

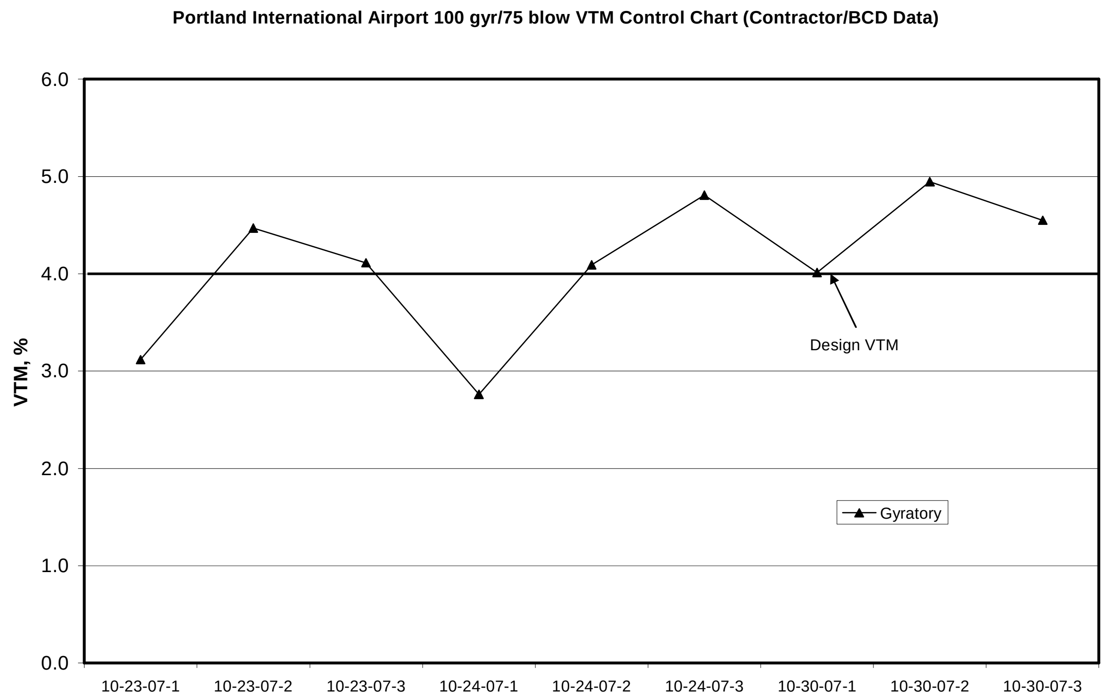

Figure 87: Control Chart for VTM (100 gyr) – Case Study No. 2

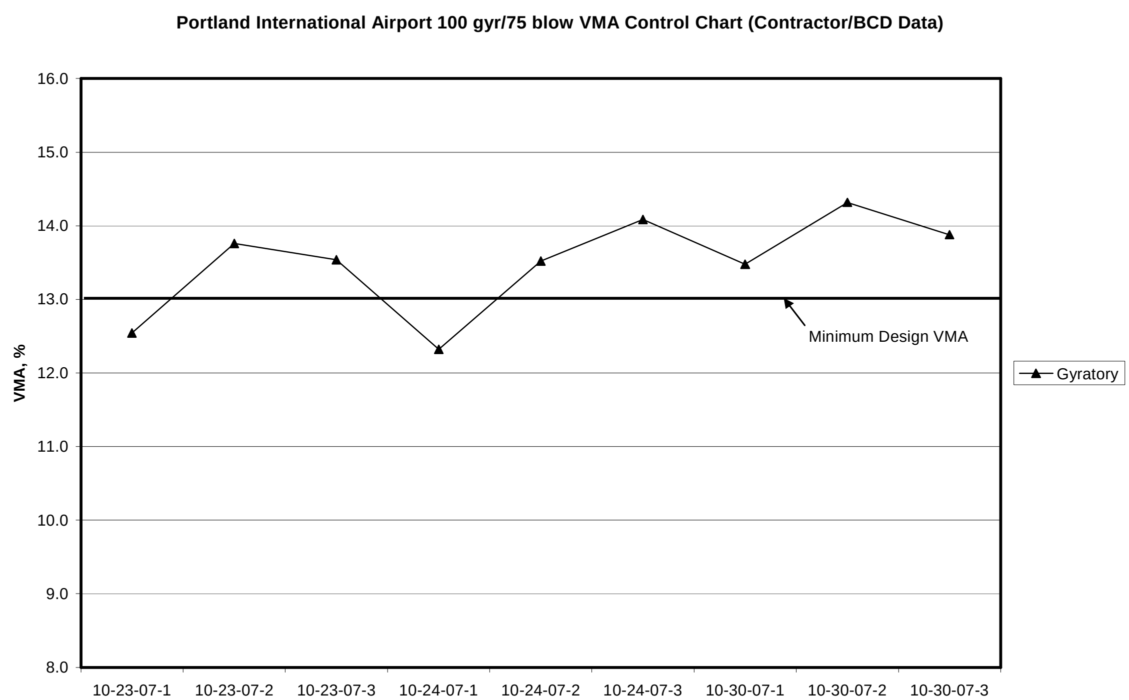

Figure 88: Control Chart for VMA (100 gyr) – Case Study No. 2

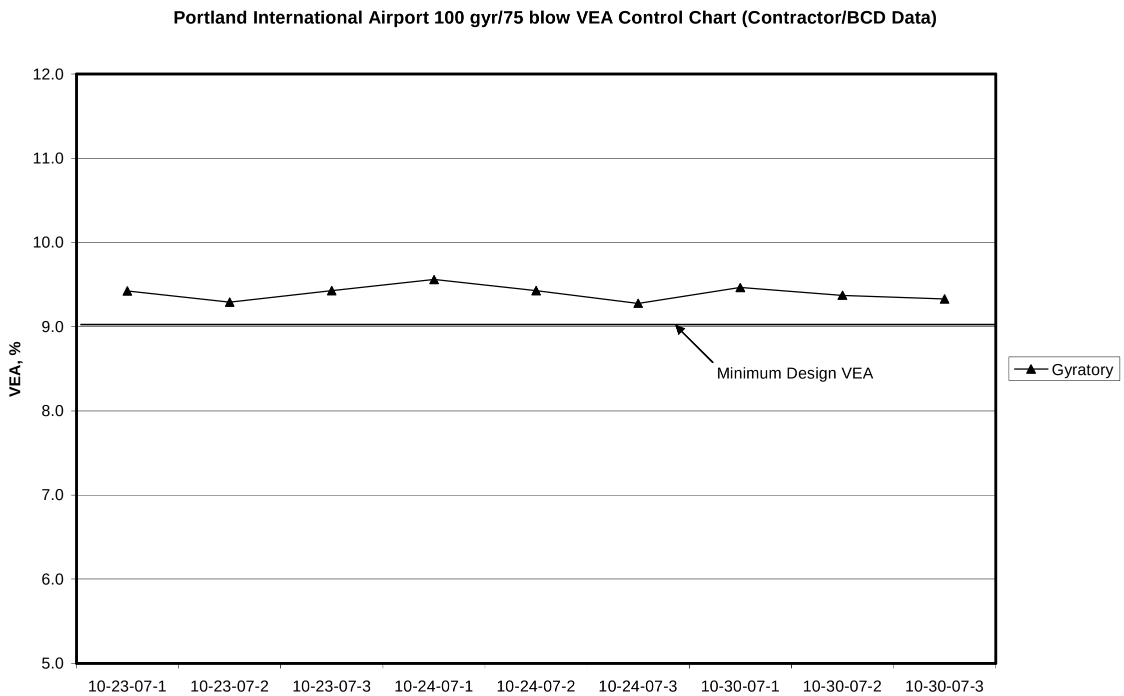

Figure 89: Control Chart for VEA (100 gyr) – Case Study No. 2

Summary

In summary, results from the shadow specification suggest that the control and assurance of HMA with the Superpave gyratory compactor is very similar to that of the Marshall hammer. When comparing the pooled standard deviations for replicate samples compacted using the Marshall hammer and Superpave gyratory compactor the values were very similar. However, the pooled standard deviations for the Superpave gyratory compactor were slightly lower than for the Marshall hammer for both projects.

One of the potential concerns with utilizing the Superpave mix design method for airfields is that there is no proof test for design or construction. Historically, the Marshall mix design method utilized stability and flow as performance indicators. No such test currently exists for the Superpave mix design system. During quality control and quality assurance (QC/QA), only the mixture properties are evaluated. Both air voids and VMA are routinely included in QC/QA programs when utilizing the Superpave mix design procedure. It is common practice by many DOTs to specify that air voids must be within some range and that VMA can not fall below some value. There are two different philosophies when specifying VMA during QC/QA operations. Some agencies specify that the VMA should not fall below the design VMA value while some may allow a 0.5 or 1.0 reduction in VMA due to breakdown of aggregates within the plant. However, it is well known that the air void content of plant produced HMA will vary. As air void contents vary, the VMA will also vary.

Based upon the data observed from the shadow specifications for both projects, a better method of controlling mix quality may be VEA instead of VMA. The primary reason that VEA may be a better indicator of mix quality than VMA is that VEA is a direct measure of what is important, the volume of effective asphalt binder within the mixture. Secondly, the VEA appears to be a much more consistent material property as it is the numerical difference between VMA and VTM. This is illustrated in Figures 80, 81 and 82 for the Case Study No. 1 data. During the first portion of production, VMA values are above the minimum design value; however, VTM values are at the upper limit. Based upon the VEA, there was insufficient asphalt binder within the mix during this initial portion of production. Because durability is vital to airfield pavements, this measure of mix quality seems appropriate.

CHAPTER 7
Conclusions, Recommendations and Implementation

The objective of this study was to adapt the Superpave gyratory compactor procedures for designing airfields HMA mixes. In order to accomplish this objective, HMA (in the form of cores) was obtained from ten airfield pavements located around the US. Materials from the original sources were also obtained for each of these ten airfields. These materials were used to conduct a laboratory study in order to adapt the Superpave mix design system for airfield pavements.

During the course of this study, the Superpave mix design system was expressed as a four step process. The first step is materials selection. Materials requiring selection include coarse aggregates, fine aggregates, mineral fillers, asphalt binder, and/or anti-stripping additives. Using the selected materials, a design gradation is obtained during the second step. The third step involves selecting the optimum asphalt binder content. This step involves the primary area where the Superpave mix design method had to be modified for the design of airfield HMA, which is the use of the Superpave gyratory compactor as the laboratory compactive effort. The final step of the mix design system is to evaluate the mixture for the potential for moisture damage.

Conclusions

Based upon the activities conducted within this research project, the following conclusions are provided:

- There are many similarities between the historical airfield mix designs, Item P-401 and UFGS-32 12 15, and the highways version of the Superpave mix design system.

- Aggregate quality characteristics required in the three mix design methods are similar in that angularity, shape, toughness, soundness and cleanliness are all included. Specified values differ slightly; however, the characteristics are consistent within the different mix design methods.
- There are differences between the gradation requirements contained in the three mix design methods. The UFGS-32 12 15 requirements generally allows the finest gradations while the Superpave mix design method for highways allows the coarsest.
- The primary difference between the three mix design methods is that the Superpave mix design procedure utilizes the Superpave gyratory compactor as the design compactive effort while the two historical airfield mix design methods utilize the Marshall hammer.
- Similar volumetric properties are required when utilizing the three mix design methods, though there are some differences in the required values. Most notably, the Superpave mix design procedure for highways requires a single air void content for selection of optimum asphalt binder content, 4.0 percent. The two historical airfield mix design procedures utilize a range of design air void contents in selection of optimum asphalt binder content.
- All three methods of designing HMA evaluate the moisture susceptibility of designed mixes using the tensile strength ratio method. Specified values are slightly different, however.
- For the ten airfields evaluated within this research, the Marshall hammer design compactive effort appeared to more accurately reflect the ultimate density of pavement layers than did the Superpave gyratory compactor. The data suggested that

design compactive efforts utilized for the different HMA pavement layers were too high when using the Superpave gyratory compactor.

- Based upon the obtained materials from the original sources for the ten airfields, 43 to 55 gyrations provide an equivalent compactive effort to 75 blows per face of the Marshall hammer, while 32 to 40 gyrations provides an equivalent compactive effort to 50 blows per face of the Marshall hammer.
- The repeated load permanent deformation test (Flow Number) test was utilized in order to identify the asphalt binder content at which the recreated HMA from each of the ten airfields would begin to exhibit high rutting potential. This asphalt binder content was then utilized to determine the gyration level that would result in 4 percent air voids. A design gyration level (N_{design}) was then estimated that would maximize the durability of HMA while minimizing rut potential. Estimated N_{design} values for the ten airfield HMA mixtures ranged from a low of 35 to a high of 75 gyrations.
- Estimated N_{design} values were more related to the design tire pressures than the maximum gross aircraft weights.
- Results of permeability testing conducted on HMA mixtures having varying gradations suggested that the gradation requirements contained within the highways version of the Superpave mix design allowed gradations that had potential for permeable pavement layers.
- Results from permeability testing suggested that the lower control points contained in the gradation requirements of the highways version of the Superpave mix design method should be increased by 5 percent passing.

- The Superpave gyratory compactor was utilized on two ongoing airfield construction projects as a shadow specification. Results of comparisons using the Marshall hammer and the Superpave gyratory compactor indicated that the variability in bulk specific gravity values were very similar even though three replicate samples were compacted with the Marshall hammer and two samples were compacted with the Superpave gyratory compactor.
- The evaluation of the Superpave gyratory compactor within the shadow specifications suggested that the percent volume of effective asphalt may be an effective tool for ensuring a sufficient volume of asphalt binder is included within the mixture during production.

Recommendations

Based upon the conclusions discussed above, a mix design procedure utilizing the Superpave gyratory compactor was developed and is recommended. Table 40 presents the recommended volumetric properties. This mix design method is presented in whole within Volume II of this report. It is recommended that this mix design method be evaluated by a number of persons that are experienced with both airfield pavement construction and the highways version of the Superpave mix design procedure. The recommended Superpave mix design method for airfields should be utilized to design HMA for several demonstration projects in order to verify that HMA mix designed using the recommended procedure can be properly produced. Currently, there are a number of research efforts for adapting the Superpave mix design method for airfields; researchers from these various efforts should meet in order to discuss the recommended procedure and share research results in an effort to improve the mix design procedure.

Table 40: Recommended Volumetric Properties For Selecting Optimum Asphalt Binder

Tire Pressure, psi	N_{design}	Required Relative Density, Percent of Theoretical Maximum Specific Gravity		Voids in the Mineral Aggregate (VMA), Percent Minimum Maximum Aggregate Size, mm				Voids Filled with Asphalt (VFA) Range, Percent	Dust-to-Binder Ratio Range
		$N_{initial}$	N_{design}	1 ½	1	¾	½		
<100	50	≤90.5	96.0	12.0	13.0	14.0	15.0	70-80	0.6-1.2
100 to 200	65	≤90.5	96.0	12.0	13.0	14.0	15.0	65-78	0.6-1.2
>200	80	≤89.0	96.0	12.0	13.0	14.0	15.0	65-75	0.6-1.2

Implementation

Specifying airfield pavement construction is much different than specifying highway pavement construction, especially civilian airfields. In most instances, local Civil Engineers are utilized by the airports to develop pavement construction specifications. The majority of implementation activities should be geared toward the education of these Civil Engineers and area FAA representatives on the intricacies of specifying HMA using the recommended Superpave mix design method for airfields. To aid in this education, Volume II of this report was developed. Volume II provides details and actions needed to design HMA for airfield pavements. Also included in Volume II is guidance on properly selecting HMA mixtures for different airfield pavement types. An important implementation activity would be to develop a training program for persons specifying airfield pavement construction that details the Superpave mix design procedure for airfields and provides guidance on mix selection. A uniform training program, whether presented on-line or in multiple locations, would greatly enhance the ability to implement Superpave for airfields.

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