Evaluation of the Superpave Gyratory Compaction Procedure

Final Report

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ABSTRACT

This report documents and presents the results of a study of the refinement of the Superpave gyratory compactor. The information includes the results of a literature search, gyratory compactor questionnaire results, laboratory testing, and conclusions and recommendations. Project work included developing mixture design procedures for gap graded and large stone mixtures, evaluating the possible reduction of the number of compaction levels (N_{design}), an evaluation of using the mixture's compaction temperature as the short term aging temperature in lieu of the standard aging temperature, developing compaction levels for mixtures below 100 mm in the pavement structure, an evaluation of the $N_{initial}$ and $N_{maximum}$ gyratory compaction requirements, an evaluation of the back-calculation of specimen densities from $N_{maximum}$, and a revision of AASHTO TP-4.

Various laboratory prepared and field obtained mixtures were evaluated in the Superpave gyratory compactor. The evaluation consisted of an extensive analysis of the volumetric and densification properties of the compacted mixtures.

The results of the study indicate that gap graded and large stone mixtures can be successfully designed in the gyratory compactor, the number of gyration levels can be reduced substantially from the present levels, the number of gyrations for mixtures placed below 100 mm in the pavement structure can be reduced from that required for surface mixtures, and changes to the $N_{initial}$ and $N_{maximum}$ gyratory compaction requirements should be made.

CHAPTER 1- INTRODUCTION AND RESEARCH APPROACH

1.1 BACKGROUND

The Strategic Highway Research Program (SHRP) on asphalt has been concluded and a number of new tests and new analysis procedures have been recommended for adoption. These new Superpave tests include asphalt binder tests, mineral aggregate tests, and mixture tests. One of the primary purposes of the SHRP program was to develop tests that would allow for evaluation of hot mix asphalt (HMA) on a more rational basis. It is believed that these new tests have partially accomplished the primary goals of SHRP, but additional research is needed in the future to finalize some of the results.

The Superpave gyratory compactor (SGC) was developed as a part of the SHRP study and has been recommended as an improved method for compaction and analysis of HMA in the mix design process. The purpose of this study was to provide the specific information needed to refine the Superpave gyratory compaction procedure. This new procedure, just like any other new procedure, will take a significant amount of research to develop the optimum test procedure to be used. This study has provided answers to many of the major questions and has identified additional studies needed to answer other questions discussed in this study.

1.2 RESEARCH PROBLEM STATEMENT

The Superpave gyratory compaction procedure, developed under the Strategic Highway Research Program (SHRP), is the method required for all Superpave mix designs. It must provide a density in the compacted laboratory specimen that closely approximates the ultimate density of the HMA mixture obtained in the pavement when subjected to traffic loads and climatic conditions; so that an appropriate optimum asphalt content can be selected during the laboratory compaction process.

There are three controls (vertical pressure, gyratory angle, and gyration rate [rpm]) on the Superpave gyratory compactor that must be set to estimate the ultimate field density. These control parameters were established by SHRP researchers and are not the subject of this research project. The number of gyrations (N) also has a significant effect on the laboratory density. SHRP has established, based on a limited set of data, a procedure for determining N_{design} for dense graded mixtures; which is based on the design high air temperature of the paving location and the traffic level in terms of equivalent single axle loads (ESALs). The current Superpave mixture design specification *(37)* allows for 28 possible values of N_{design} (seven levels of ESALs and four levels of temperature). This is thought to be excessive and confusing; therefore it is believed that these N_{design} levels should be reduced to a lower number, such as four to six levels. Currently, there is a research effort sponsored by the Federal Highway Administration (FHWA) aimed at verifying the N_{design} compaction table for dense-graded mixtures.

Other factors that will affect the gyratory compactor results include aggregate properties, such as gradation, absorption, hardness, etc.; and the short-term aging of mixtures. Additionally, non-dense-graded asphalt mixtures, such as gap-graded and large stone mixtures, which were not included in the SHRP program, need to be evaluated using the Superpave gyratory compactor to establish the proper compaction levels. Also not evaluated in the initial SHRP research was whether the levels of N_{design} should be reduced for binder and base mixtures.

1.3 OBJECTIVES

The objectives of this research were to (1) recommend revisions to the Superpave gyratory compaction procedure (AASHTO TP4) and related mixture preparation requirements, and (2) recommend Superpave gyratory compaction procedure(s) for gap-graded and large stone mixtures.

1.4 SCOPE

This project was undertaken by the National Center for Asphalt Technology (NCAT) in partnership with the Asphalt Institute and Heritage Research Group. NCAT was responsible for the overall conduct of the project and specifically for tasks 1, 2, 4, 5, 6C, 7, and 8. The Asphalt Institute was responsible for tasks 3 and 6A and Heritage Research Group was responsible for task 6B.

The study involved a literature search, a Superpave gyratory compactor user survey, and extensive laboratory testing by the research team. The laboratory testing evaluated the effect of varying short term aging temperature on mixture volumetrics, the effect of depth of the mixture on the required number of gyrations, the consolidation of the N_{design} compaction matrix, and an evaluation of the $N_{initial}$ and $N_{maximum}$ requirements. The laboratory testing also resulted in compaction procedures for gap graded mixes and large stone mixes.

CHAPTER 2 LITERATURE REVIEW AND GYRATORY COMPACTOR USER SURVEY

2.1 TASK 1: LITERATURE REVIEW

Appropriate literature has been reviewed and included as a part of this report. Literature pertaining to the gyratory compactor, short term aging of mixtures, gyratory compaction of large stone mixtures and gap graded mixtures was reviewed, with the emphasis of the reviewing process being on the gyratory compactor. A summary of each publication reviewed is provided below in chronological order.

1. Ortolani, L. and Sandberg, H. A. Jr., "The Gyratory-Shear Method of Molding Asphaltic Concrete Test Specimens; Its Development and Correlation with Field Compaction Methods. A Texas Highway Department Standard Procedure." Journal of the Association of Asphalt Paving Technologists, Volume 21, 1952, pp 280-297.

The 1939 Texas Highway Department research program to develop methods to design and control bituminous mixtures is described in this report. Further, the first step in the design was to develop a method of molding hot mix asphalt (HMA) specimens. In the past, methods of control and design had relied on the mixture's appearance and feel or the experience of the designer. Various machines and equipment were evaluated as possible means of molding or compacting HMA specimens. Among these were the hydraulic jack and press assembly, the standard Proctor Soil Compaction Machine, the Public Roads Administration Vibratory Machine, and a pneumatic roller machine. However, the machine chosen for compaction at the conclusion of the study was the Gyratory Molding machine. The major problem with the other machines evaluated was that a shear force was imparted to the surface of the HMA specimen, but not evenly distributed throughout the specimen. The Gyratory Molding machine imparted the shear force throughout the specimen by holding the top and bottom surfaces parallel and rigid.

The standard specimen size in the Gyratory Molding machine was 50 mm in height and 100 mm in diameter.

2. Bonnot, J., "Asphalt Aggregate Mixtures." Transportation Research Record 1096, TRB, Washington D. C., (1986), pp 42-51.

The paper focuses on a number of elements which define the French HMA mix design technology. These elements include the study of compaction characteristics with the gyratory shear compacting press (PCG). The PCG applies a vertical or compacting pressure which closely resembles that of rubber tired compactors in the field. Along with the vertical pressure, a kneading action is applied to the asphalt mixture. Compaction takes place in a 160 mm diameter mold, with a constant angle of inclination of one degree, with a vertical pressure of 600 kPa. The rotational speed is set at six revolutions per minute (RPM). Specimen height after compaction is approximately 150 mm. A schematic of the PCG compaction procedure is shown in Figure 2.1. The temperature of the specimen is regulated throughout the compaction process. Two values are

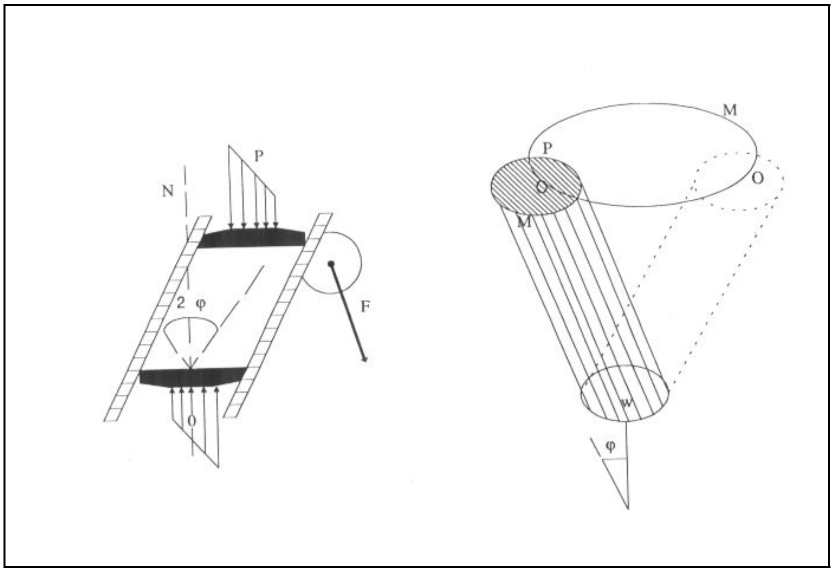

Figure 2.1 Schematic of the PCG Compaction Procedure

measured during compaction; the specimen height and the inclination force (F) required to maintain the one degree angle.

Numerous correlation studies have been performed between the void content given by the PCG and the number of passes of a rubber tired compactor in the field. For HMA with thicknesses between 30 and 120 mm, the number of revolutions corresponding to the number of passes of the rubber tired compactor is given by the following formula:

$$N_g = 0.0625\ e\ N_p$$

where

N_g = number of PCG press revolutions,
e = thickness of the asphalt pavement in millimeters,
N_p = number of rubber tired compactor passes.

The use of this equation allows the user to calculate the number of gyrations necessary to achieve a laboratory compacted void content equal to that of the pavement in the field immediately after compaction. For example, if a pavement is intended to have a thickness of 100 mm with 16 passes of a rubber tired compactor, then the reference number of gyrations is 100. Very often the number of gyrations is taken to be the pavement thickness. This approach assumes that the number of passes of the rubber tired compactor will be approximately 16. However, when vibratory compactors are used, the equation now becomes the following:

$$N_g = k\ e\ N_p$$

where k is a factor depending essentially on the nature of the compaction increasing with compactor efficiency. An assumption is sometime made that the vibrating drum has a linear static load of 3.5 kN/mm and is subjected to the action of a cam wheel of about 10 tons and a frequency of 25 to 30 Hz. This gives a k factor of approximately 0.25, which is substituted into the above equation.

3. Kandhal, P. S., Cross, S. A., and Brown, E. R., "Evaluation of Heavy Duty Asphalt Pavements for Rutting", Proceedings of the Seventh International Conference on Asphalt Pavements, Volume Four, 1992, pp 64-78.

Thirty-four large stone pavements in Pennsylvania were evaluated to determine the factors which lead to the premature rutting. The pavements consisted of both binder and wearing courses with varying aggregate sizes. A total of five cores were taken from each of the sites and the in-place properties of the mixture determined. The cores were then broken down and re-compacted using three different methods: the Gyratory testing machine (GTM) at 827 kPa vertical pressure, 1 degree angle, and 300 revolutions; the rotating base, slanted foot, mechanical Marshall compactor (75 blows per side); and the conventional, static base Marshall compactor (75 blows per side). After compacting, the volumetric properties were determined and compared to the core properties. All of the pavements evaluated had been designed using the Marshall method with varying numbers of blows per side. Results from the study indicate that the rotating base/slanted foot Marshall compactor yielded the highest density for both the wearing and binder courses, with the GTM providing the lowest density for the wearing course. However, as the maximum aggregate size increased (25-38 mm maximum size), the GTM provided greater densification of the mixes.

4. Vail, P. L., "The Trial of Three Large Stone Mix Pavements", Proceedings of the Association of Asphalt Paving Technologists, Volume 62, 1993, pp 693-707.

The Australian Asphalt Pavement Association (AAPA) designed 3 large stone pavements for heavy duty applications within Australia. These pavements were dense graded mixtures containing nominal maximum aggregate sizes from 28 to 40 mm. The Gyratory Testing Machine (GTM) was used for the design of two of the mixtures. With the first mixture, the GTM was set at a 2 degree angle and a vertical pressure of 690 kPa. The air voids, stability index, and compactibility index of compacted samples were determined after 30, 60, and 90 revolutions. Ninety revolutions were chosen as the design number of gyrations for the mixture.

The second mixture to utilize the GTM was designed by the Percentage Refusal Density method. The goal was to produce a mixture with a minimum of 3 percent air voids at a compaction level equivalent to a 75 blow Marshall design. Specimens were compacted using the GTM with a 2 degree angle and a 690 kPa vertical pressure at 30, 60, 90, 120, and 150 revolutions. Optimum asphalt content was determined to be 3.2 percent at 60 revolutions. Conclusions reached were that large stone mixtures can be designed effectively with the both the Marshall hammer and the GTM. The pavements described in this paper have provided good performance for several years in very heavy duty, high traffic applications.

5. Sosnovske, D. A., AbWahab, Y., and Bell, C. A., "Role of Asphalt and Aggregate in the Aging of Bituminous Mixtures." Transportation Research Record 1386, TRB, Washington D. C., (1993), pp 10-21.

Short and long term aging procedures for asphalt mixtures were evaluated in this study. Eight asphalt binders and four aggregates were used, encompassing a broad range of properties. All mixtures, with the exception of three specimens, were subjected to short term aging at 135°C for 4 hours before compaction in the California kneading compactor. These three specimens were mixed and immediately compacted in the kneading compactor. Four long term aging procedures were then evaluated: (1) low pressure oxidation (LPO) at 60°C , (2) LPO at 85°C , (3) long term oven aging (LTOA) at 85°C for 5 days and (4) LTOA at 100°C for 2 days. After each of the long term aging procedures, the compacted specimens were evaluated by resilient modulus, dynamic modulus, and tensile strength testing. LPO is an aging procedure in which short term aged specimens are place in a triaxial pressure cell and subjected to a confining pressure while oxygen is passed through them. This process typically takes place in a water bath at the conditioning temperature (60 or 85°C) for a period of five days. LTOA is another method of simulating the long term aging of asphalt mixtures. Specimens which have been short term aged are placed in a forced-draft oven at a temperature of 85°C for five days, or 100°C for two days. All long term aging procedures involved conditioning the compacted asphalt specimen.

Conclusions reached were (1) the aging of asphalt mixtures is influenced by both the asphalt and the aggregate, (2) the amount of asphalt aging is possibly related to the degree of adhesion between the aggregate and the asphalt binder (i.e. a stronger bond yields lower aging of the asphalt), (3) the LPO long term aging procedure produces the most mixture aging and the least variability of testing, and (4) the short term aging procedure does not enable prediction of long term aging.

6. Bell, C. A., AbWahab, Y., Cristi, M. E., and Sosnovske, D., "Selection of Laboratory Aging Procedures for Asphalt-Aggregate Mixtures." Strategic Highway Research Program (SHRP) Report No. A-383, National Research Council, Washington, D. C. 1994.

This report presents the results of a preliminary study to evaluate the aging methods for asphalt-aggregate mixtures. Specifically, the following short term and long term aging methods were evaluated in the study: Short term aging (forced draft oven and extended mixing) and Long term aging (forced draft oven, pressure oxidation, and triaxial cell aging). Two asphalt binders and two aggregates (crushed granite and chert gravel) were used, resulting in a total of four mixtures to be tested.

The short term aging evaluation consisted of aging specimens in a forced draft oven for durations of 0, 6, and 15 hours at temperature of 135°C or 163°C. The aged mixture was then compacted in the California kneading compactor to air void levels of 4 and 8 percent. The extended mixing time consisted of using a modified rolling thin film oven test. Samples were aged for 0, 10, 120, and 360 minutes at either 135°C or 163°C, and then compacted in the kneading compactor to 4 and 8 percent air voids.

The long term oven aging method consisted of using a forced draft oven to age compacted specimens for 0, 2, and 7 days at 107°C. Pressure oxidation used oxygen and compressed air. Compacted samples were exposed to pressures of 690 or 2070 kPa at 25°C or 60°C for durations of 0, 2, or 7 days. The triaxial cell procedure consisted of confining a compacted sample and then passing oxygen through the sample.

Evaluation tests for the samples consisted of resilient modulus, dynamic modulus, and tensile tests. Rheology tests on the recovered binder were planned, but were not run to any significant extent in the study.

Some of the conclusions and recommendations from the study are given below.

1. Short term aging and extended mixing result in a fourfold increase in mixture resilient modulus.
2. Extending mixing produces a more uniform aged mixture; however, short term aging is more efficient during production. Short term aging at 135°C for 4 hours appears to be the most appropriate method.
3. LTOA at 107°C is too harsh and unrealistic; therefore a temperature of 85°C is recommended.

7. Bell, C. A., Wieder, A. J., Fellin, M. J., "Laboratory Aging of Asphalt-Aggregate Mixtures: Field Validation. " Strategic Highway Research Program (SHRP) Report No. A-390, National Research Council, Washington, D. C. 1994.

This reports documents the results of the field validation of the short term and long term aging procedures described in SHRP Report A-383. The validation consisted of a preliminary and an expanded field validation. The preliminary validation sites, a total of four, were selected from Oregon Department of Transportation (ODOT) projects. This preliminary validation was targeted at determining a common time period to short term age the mixes which best represented the aging occurring in the construction process. Expanded sites, a total of 11, were located in Arizona (2), California (2), France (3), Georgia (1), Michigan (1), Minnesota (1) and Wisconsin (1). All of the expanded sites, with the exception of the France sites, were either AAMAS or SPS pavements. The expanded validation was designed to determine the validity of both the short term aging and the long term aging procedures to simulate field aging. Additionally, seven sites in the state of Washington were chosen for supplementary sites for field validation. The supplemental validation was simply an extension of the expanded validation.

Some recommendations reached from the study include the following:

1. Additional sites should be selected to better determine the effectiveness of the short term aging period of 4 hours. However, it appears that 4 hours at 135C is a good (although conservative for some sites) procedure for short term aging.
2. Possibly evaluate the low pressure oxidation (LPO) test at varied temperatures.
3. More parameters need to be studied to provide a more accurate determination of the effectiveness of the aging procedures. These would include traffic, field temperature, rainfall, pavement age, and laboratory aging time.

8. Cominsky, R., Leahy, R. B., Leahy, and Harrigan, E. T., "Level One Mix Design: Materials Selection, Compaction, and Conditioning." Strategic Highway Research Program Report No. A-408, National Research Council, Washington, D. C., 1994.

This report provides a detailed background and overview of the Superpave mix design system as it was developed. Specifically, it provides a detailed description of how the Superpave gyratory compactor was selected for use in mix design and quality control work in the Superpave system. After considerable research and effort, SHRP researchers selected to use a gyratory compactor with very similar operating protocols as the French (LCPC) gyratory compactor. A summary of how the existing compaction parameters were developed is provided in the following:

Gyrations Per Minute (Rotational Speed)

The French gyratory compactor operates at a rotational speed of 6 revolutions per minute (rpm). SHRP researchers desired to speed up the rotational speed, provided the volumetric properties of mixtures were not adversely affected. An experiment was conducted using one aggregate and one asphalt binder to determine if the mixture volumetrics (optimum asphalt content, air voids, VMA, and VFA) were affected between rotational speeds of 6, 15, and 30 rpm. Statistical analysis showed that the volumetric properties between the rotational speeds were not statistically different, and a speed of 30 rpm was selected for gyratory compactor operation.

Gyratory Compactor Comparison

Next, an experiment was conducted to determine if the specification of angle of gyration, rotational speed (rpm), and vertical pressure were sufficient to produce similar compactors. The experiment compared the modified Texas gyratory compactor, the SHRP gyratory compactor, and to a lesser extent, the U. S. Army Corps of Engineers gyratory compactor (USACOE). The SHRP gyratory compactor was manufactured by the Rainhart Company. The factors in the experiment consisted of the following:

Aggregate Blends:	Four blends were selected ranging from 9.5 to 25 mm nominal maximum size. These blends had been previously designed and produced.
Specimen Sizes:	Two specimen sizes were evaluated; 150 mm and 100 mm. 100 mm specimens were not possible with the modified Texas gyratory.
Asphalt Contents:	One asphalt cement was used (AC-20) with three contents; optimum, optimum plus 1 percent, and optimum minus 1 percent.

Compaction parameters, angle of gyration (1 degree), vertical pressure (600 kPa), and rotational speed (30 rpm); were selected and held constant for all compactors. All specimens were short term aged at 135°C for 4 hours. A major difference in the compactors used in the experiment was that the USACOE gyratory compactor operates with a variable angle of gyration, while the other two gyratory compactors have a fixed angle. Therefore, in lieu of a complete evaluation, a limited evaluation of the USACOE gyratory versus the other two gyratory compactors was accomplished with a single mix at three asphalt contents.

Conclusions reached from the experiment were as follows:

1. Specifying the angle of gyration, rotational speed, and vertical pressure alone is not sufficient to produce similar compactors.
2. The angle of gyration should have a angle of gyration of 1.00 ± 0.02 degrees.
3. Based on the limited evaluation, the USACOE gyratory compactor, operated at 1 degree, does not produce similar results as the SHRP gyratory compactor. This is attributed to the variable angle of the USACOE.

9. Blackenship, P.B., Mahboub, K. C., and Huber, G. A., "Rational Method of Laboratory Compaction of Hot-Mix Asphalt." Transportation Research Record 1454, TRB, National Research Council, Washington D.C., (1994), pp. 144-153.

The experimental approach, results, and conclusions from the initial N_{design} experiment are provided in this article. The N_{design} experiment was undertaken to determine the number of gyrations (N_{design}) required to represent the various traffic levels in differing geographical locations and climates. In accomplishing this task two gyration levels were evaluated; one was $N_{construction}$ which represents the initial laydown compaction level $C_{construction}$ and the other was N_{design} representing the compaction in the wheel path of the pavement under applied traffic C_{design}. For the experiment the value of $C_{construction}$ was unknown for many of the pavements and was assumed to be 92 percent of G_{mm}. The original experiment was to require 27 pavement sites with 54 mixtures. This was to provide three climates (hot, warm, and cool), three traffic levels (low, medium, and high), and two pavement layers (upper and lower). However, it was later decided to evaluate only pavements which had been in service for over 12 years. This resulted in the number of evaluated pavements being reduced to 18, with 15 being available for final evaluation. An assumption was made that all the mixtures were designed to have approximately 3 to 5 percent air voids in the laboratory and air voids in place of 7 to 9 percent immediately after construction

The aged asphalt was extracted from 305 mm cores taken from the various pavements and the aggregate re-mixed with an unaged AC-20 asphalt cement. The mixed specimens were then aged for 4 hours at 135°C and compacted to 230 gyrations using the SHRP gyratory compactor. Mixtures with 19.0 mm and less nominal maximum aggregate sizes were prepared using the 100 mm compaction mold while the 150 mm mold was used for mixtures with nominal aggregate sizes greater than 19.0 mm. All mixtures evaluated in the study had a fine gradation.

Analysis of the testing results provided a method of choosing N_{design} for a desired traffic level and an average 7-day high temperature. The authors suggested that the results and conclusions from the experiment were acceptable but more research needed to be completed to increase the precision of N_{design}.

10. Button, J. W., Little, D. N., Jagadam, V., and Pendleton, O. J., "Correlation of Selected Laboratory Compaction Methods with Field Compaction." Transportation Research Record 1454, TRB, National Research Council, Washington, D.C., (1994), pp. 193-201.

This study documents the results of an effort to determine the laboratory compaction device which most closely simulates the actual field compaction. In the study, extensive work was done to compare specimens made with the Texas gyratory compactor and the Exxon rolling wheel compaction with field obtained cores. Also evaluated on a more limited basis, were the rotating base Marshall hammer and the Elf linear kneading compactor. The Texas gyratory compactor used a three degree gyration angle, a vertical pressure ranging between 50 and 150 psi, and a leveling pressure of 2500 psi.

Mixtures comprised of different aggregates and asphalt binders were chosen for evaluation from five pavement sites. Approximately 30 cores from each site were taken and their in-place density determined. Samples of the aggregate and asphalt binder which were used in the construction of the pavement sites were obtained and the mixtures combined in the laboratory. The combined mixtures were then compacted in the various laboratory compaction devices.

Average air voids content for the pavement cores ranged from 3 to 8 percent, therefore, laboratory samples were compacted to that range of air voids in the laboratory. Six laboratory tests were performed on the compacted samples and the field cores. These tests included indirect tension at 25°C, resilient modulus at 0°C and 25°C, Marshall stability, Hveem stability, and uniaxial repetitive compressive creep followed by compression to failure. Some conclusions reached from the laboratory testing were as follows:

1. The Texas gyratory compactor most often (73 percent of the tests) produced specimens similar to pavement cores. The Marshall rotating base compactor had the least (50 percent of the tests) probability of producing similar specimens.
2. For producing small specimens with specific air void contents, the gyratory method of compaction was much more convenient and faster than the other compaction methods.
3. Based upon the findings in this study, the Texas gyratory compactor was recommended to SHRP for use in the preparation of laboratory test specimens.

11. Anderson, M.R., Bosley, R. D. and Creamer, P. A., "Quality Management of HMA Construction Using Superpave Equipment: A Case Study." Transportation Research Record 1513, TRB, Washington, D.C., (1995), pp. 18-24.

This paper provides the results from a case study in which the Superpave Gyratory Compactor was used for field quality control testing for an intermediate course mixture on an interstate highway in Lexington, Kentucky. The project was initiated in order to determine the ability of the SGC to detect subtle changes in the properties of material components.

The testing consisted of a laboratory verification of the states’s mixture design with both the SGC and the Marshall hammer. The optimum asphalt content for the mixture was determined to be 4.5 percent by the Kentucky Department of Highways. For laboratory evaluation in the SGC the gradation was blended with 4.0, 4.5, 5.0, and 5.5 percent asphalt content. For the Marshall hammer (75 blow) evaluation the gradation was only blended with 4.5 percent asphalt.

SGC specimens were prepared and aged at 135°C for a period of 3.5 hours, after which they were transferred to a 160°C oven for 30 minutes to reach the desired compaction temperature. Gyratory specimens were then compacted to $N_{maximum}$ of 204 gyrations. Marshall specimens were prepared and placed in the compaction mold. The mold was then placed in a 143°C oven for 1.5 hours to reach compaction temperature. Comparison of the volumetrics for the SGC and the

Marshall hammer specimens at 4.5 percent asphalt indicated that compaction with the SGC yielded lower air voids and VMA and higher VFA than did the Marshall hammer prepared specimens.

The second portion of the study consisted of a field study to determine the sensitivity of the SGC to material property changes. Field samples were obtained and compacted using the SGC and the Marshall hammer. Results indicated that the SGC appeared to be extremely sensitive to changes in asphalt content. For field samples, the average difference in air voids of two SGC compacted specimens was 0.3 percent compared to 0.6 percent for three Marshall specimens. This reduced variability is most likely a result of the increased sample size of the SGC.

12. Harman, T.P, D'Angelo, J., and Bukowski, J. R., "Evaluation of Superpave Gyratory Compactor in the Field Management of Asphalt Mixes: Four Simulation Studies." Transportation Research Record 1513, TRB, Washington D.C., (1995), pp 1-8.

This was a study undertaken by the FHWA to determine the effectiveness of the SGC in the field management of asphalt mixtures. In a side study of the project the Marshall hammer was compared to the SGC for possible use as a supplement for field control. Results of the four studies indicate that the SGC can be used as an effective tool for the field verification of laboratory designed HMA mixtures. However, in all cases, it was determined that the Marshall hammer compacts specimens in a much different manner than does the SGC; therefore it was determined that supplemental compactors should not be used for field quality control of HMA.

13. Hafez, I. H. and Witczak, M. W., "Comparison of Marshall and Superpave Level I Mix Design for Asphalt Mixes." Transportation Research Record 1492, TRB, Washington, D.C., (1995), pp 161-165.

This research project was an effort in which mix designs were performed for 20 different mixtures using both the Marshall procedure and the Superpave gyratory compactor Level I procedure. The mixtures were classified into five groups as follows: conventional mixtures, wet process asphalt rubber (manufacturer preblended), dry process asphalt rubber, polymer modified, and wet process asphalt rubber (plant blended). All mixtures in the study had the same aggregate type, source and gradation (Maryland State Highway Administration dense aggregate gradation with nominal maximum size of 12.5 mm), with the exception of two mixtures. These two mixtures were Plus Ride No. 12 and No. 16 open graded mixtures with nominal maximum sizes of 12.5 mm and 19.0 mm, respectively.

Optimum asphalt contents for all mixtures in the study were then determined by the Marshall 75 blow procedure and the Superpave Level I procedure. The Marshall procedure consisted of preparing three replicates at 1.0 percent asphalt content increments in order to cover a VTM range of 3.0 to 5.0 percent. The Superpave design consisted of compacting 100 mm diameter specimens at three different N_{design} values corresponding to a traffic level less than 10 million ESALs and design air temperatures of $\leq$34°C, 37-39°C, and 43-44°C. The N_{design} values corresponding to these parameters are 67, 96, and 119 gyrations. In addition to determining the optimum asphalt content at 4.0 percent VTM, the asphalt content was selected to provide both 3.0

and 5.0 percent VTM for comparison to the Marshall procedure.

Conclusions drawn from the study were as follows:

- The Superpave gyratory Level I mix design procedure cannot be used to design dry-process asphalt rubber mixtures. Specimens in this category experienced a degree of swelling, resulting in a volume change, after compaction which made the calculation of a corrected density at N_{design} to be in error.
- All other mixtures evaluated can be accurately designed and evaluated using the Superpave gyratory Level I procedure.
- As the compactive effort, $N_{design,}$ for the Superpave gyratory compactor is decreased from 119 to 67 gyrations an increase of approximately 1.0 percent asphalt content is experienced for all mixtures evaluated.
- For a given level of compaction with the Superpave gyratory compactor there was no consistent trends between the results from the Superpave procedure and the Marshall procedure.

14. Jones, C. Correspondence (letter), Couch Construction, Inc. Research and Development Study No. 95-2, 1995.

In response to widespread industry difficulties achieving specified volumetric properties using the Superpave Level 1 design procedure with limestone and crushed gravel aggregates, Couch Construction, Inc., initiated a study to evaluate granite aggregate in similar gradations. The study evaluated five mixtures, all of which were comprised of Georgia granite aggregate. The mixtures consisted of a Georgia Department of Transportation "E" mix, a 12.5 mm nominal maximum size Superpave mixture (well graded under the restricted zone), a 12.5 mm nominal maximum size Superpave mixture (gap graded under the restricted zone), a 12.5 mm nominal maximum size Superpave mixture (gap graded under the restricted zone with 20 percent natural sand), and a 12.5 mm nominal maximum size Stone Matrix Asphalt (SMA) mixture. The SMA mixture was used in the study for comparison purposes. All mixtures had 1 percent hydrated lime, with the exception of the SMA, which contained 5.5 percent lime and a stabilizing fiber. An AC-20 asphalt cement was used in the preparation of all mixtures. Superpave mix designs were performed for each of the five gradations using the Troxler gyratory compactor at an N_{design} of 109 gyrations. The mix design results indicated that all of the mixtures with the exception of the Georgia "E" mixture met Superpave volumetric and compaction parameters. The "E" mixture had a density at $N_{initial}$ greater than the 89 percent of the TMD which is the maximum allowed under the Superpave system. A major concern stated by the author was of the substantially low optimum asphalt content of the granite mixtures, when compared to limestone and crushed gravel mixtures in the past.

15. Sousa, J. B., Way, G., Harvey, J. T., and Hines, M., "Comparison of Mix Design Concepts." Transportation Research Record 1492, TRB, National Research Council, Washington, D.C., (1995), pp. 151-160.

The Arizona Department of Transportation evaluated mixtures using the Marshall, Superpave Level I, and a performance based procedure developed under SHRP-A003A. The laboratory designed mixture was placed on Interstate 17 near Phoenix, in November 1993. The primary goal of the study was to evaluate the new HMA component requirements set forth under the Superpave system. The material used in the study consisted of a PG 70-10 asphalt binder, along with a partially crushed river gravel (coarse aggregate had 90 percent with two or more fractured faces , (with the fine aggregate being manufactured). All mixtures were designed with 1 percent Portland cement to reduce the moisture susceptibility of the mixture. The mix design gradation conformed to a fine 19.0 nominal maximum size Superpave gradation, however, during production the aggregate source was changed. This resulted in the field gradation being coarser and passing through the Superpave restricted zone.

An optimum asphalt content for the mixture of 4.2 percent was determined using the performance based mix design approach from SHRP A003. The designed mixture was produced and placed in two 1 mile test sections on Interstate 17. Samples of the field mix were taken to the laboratory for evaluation using the Marshall and the Superpave Level I design procedures.

Results of the 75-blow Marshall testing showed stabilities of 5,044 and 3,760 lbs for the field mix and cores, respectively; both of which are well above the Arizona DOT's minimum requirement of 3000 lbs. Field samples were also compacted in the Superpave gyratory compactor at a compaction level of $N_{initial}$ (9), N_{design} (135), and $N_{maximum}$ (220). Volumetric determinations indicated that the field mixture would not meet the requirements for a Superpave Level I design. In particular, the air void content was too high (7.6 % and 6.3 %, with and without parafilm, respectively) and the VFA was too low (53.3 %). Because of the differences, the volumetric properties from the gyratory compactor were normalized to determine what optimum asphalt content would provide satisfactory volumetric results. An estimated optimum asphalt content of 5.2 percent was chosen, samples compacted, and their volumetrics determined. The results showed that the mixture marginally failed the VMA and the G_{mm} $N_{initial}$ requirements.

Field cores from this project were also evaluated in the Hamburg wheel tracking device at 55°C. Analysis of the results, against correlated field performance, indicated that the mixtures could be classified as "good" pavements which would last approximately 10 to 15 years in service.

Inspections of the pavements in July 1994 showed an average rut depth of 1.5 mm over the project. This provided an indication of the "good" performance of the mixture, since the majority of pavement failures with regards to rutting in Arizona have occurred after the first summer in service.

A further evaluation was undertaken to determine which laboratory compaction device yielded the best correlation with field compaction. Laboratory compaction devices evaluated consisted of the rolling wheel compactor, the kneading compactor, the Texas gyratory compactor, the Marshall hammer, and three SHRP gyratory compactors. The results indicated that the rolling wheel compactor compacted specimens which best correlated against field cores based upon their permanent deformation resistance in the repeated simple shear at constant height test (RSST-CH).

16. D'Angelo, J. A., Paugh, C., Harman, T. P., and Bukowski, J., "Comparison of the Superpave Gyratory Compactor to Marshall for Field Quality Control." Proceedings of the Association of Asphalt Paving Technologists, Volume 64, 1995, pp. 611-635.

Five different asphalt mixes, produced at five different asphalt plants, were compared using the Superpave Level I and the Marshall compaction procedures. Samples of the five mixtures were taken and compacted in both the Superpave gyratory compactor and the Marshall hammer. The quality control ability of the gyratory compactor and Marshall hammer were determined through an analysis of the volumetric properties. The results of the analysis indicate that samples compacted with the gyratory compactor had slightly less variability in air voids than did the Marshall samples. Based on air voids alone, the gyratory compactor and the Marshall hammer could both be expected to perform well in quality control applications. However, the voids in mineral aggregate (VMA), distinguishes the two compaction devices to a greater extent. The results show that for every mixture tested, the gyratory compactor samples had lower VMA than did the Marshall samples. For three of the five mixtures, the VMA of the gyratory and Marshall compacted samples tended to decrease with an increase in asphalt content. The other two mixtures showed that as the asphalt content increased, the VMA decreased for the gyratory samples, but increased for the Marshall samples. This indicates that the asphalt contents are on the low and high sides of the VMA curve for the gyratory and the Marshall hammer, respectively. The general trend of lower VMA with the gyratory indicates that the compaction effort obtained with the gyratory compactor is greater than with the Marshall hammer for the mixtures evaluated. The overall conclusion of the study was that the gyratory compactor identified the plant production variability more so than did the Marshall hammer.

17. Brown, E.R., Hanson, D.I., and Mallick, R., "Evaluation of SHRP Gyratory Compaction of HMA." Transportation Research Record 1543, TRB, Washington D.C., (1996), pp 145-150.

Specimens compacted in the Superpave gyratory compactor at different gyration levels were compared with the density of in-place cores obtained from pavement test sections at various levels of cumulative traffic. Project work consisted of obtaining cores from six test pavements (2 in Alabama, 1 in the states of Idaho, South Carolina, New Mexico, and Wisconsin) with different levels of traffic. The cores were taken immediately after construction, after one year in service, and after two years in service. The air void content and the density of the cores were then established. Two sets of specimens were then compacted using the SGC. One set of specimens consisted of original plant produced material which was reheated and then compacted (This set is referred to as compacted-reheated). The other set consisted of using the aggregate and asphalt cement which was used in the mixture (This set is referred to as laboratory prepared).

Results from the study provide the following conclusions:

- The gyrations required to achieve the one and two year in-place density were below 100 for all mixtures evaluated.
- For similar gyration levels, the density of compacted reheated specimens and laboratory prepared specimens varied about one percent on average.

- The N_{design} gyration level may be too high for low traffic volume roadways. This will be further evaluated in the future after the three year in-place density is recorded.
- The values of voids at $N_{initial}$ and N_{max} were lower than the specified values based upon the laboratory data obtained from the project.

18. McGennis, R.B., Anderson, M. R., Perdomo, D., and Turner, P., "Issues Pertaining to Use of Superpave Gyratory Compactor." Transportation Research Record 1543, TRB, Washington D.C., (1996), pp 139-144.

This project was undertaken to evaluate the effect of various compaction parameters on the results of the Superpave Gyratory Compactor (SGC). Parameters included mold diameter, short-term aging time, and compaction temperature. A study was also performed to determine if changing any of the parameters affected the AASHTO T-283 moisture susceptibility results. In order to determine the variability of mixtures with regards to the above compaction parameters, specimens were compacted in three gyratory compactors: the Troxler SGC, Pine SGC, and the modified Texas SGC. A fourth compactor, the Rainhart SGC, was used in a compactor comparison portion of the study.

Mold Diameter

For the mold diameter comparison, five 19.0 mm and two 12.5 mm nominal maximum size aggregate structures were used. The gradations, six total, ranged from gap graded to intermediate, with all being below the restricted zone. The design asphalt content for each of these mixtures was established to provide 4 percent VTM at an N_{design} of 172. Specimens were prepared at optimum asphalt content, optimum plus 0.5 percent, and optimum minus 0.5 percent for each of the seven mixtures. Next, specimens were compacted, at the design asphalt content, in 150 mm and 100 mm gyratory molds. For the experiment the volumetric properties of the mixtures were compared at gyration levels of 10, 100, 150, and 250 gyrations. Specimen bulk gravities from the two mold sizes were then compared. The results from the experiment indicate that for 56 percent of the comparisons there was a significant difference between the 150 mm and the 100 mm diameter specimens. Also, within a particular nominal maximum size, mold size affects the densification of coarser mixes more often than it affects that of the intermediate mixes. However, the amount to which mold size affected the volumetric properties of the mixture was not affected by nominal aggregate size or by asphalt content.

Compaction Temperature

In an effort to evaluate the effect of compaction temperature on specimen volumetrics, two asphalt binders (PG 64-28-unmodified and a PG 76-28-polymer modified) were blended with one designed gap graded aggregate gradation and compacted at a range of temperatures. Specimens were prepared at the design asphalt content of 4.7 percent and short term aged at 135°C for 4 hours. After the aging time the specimens were placed in an oven at the specified compaction temperatures. The compaction temperatures used in the study were 120°C, 135°C, 150°C, 165°C, and 180°C. Afterwards, two specimens were compacted in each compactor at each compaction temperature. The results indicated that the variation in compaction temperature did not substantially affect the volumetric properties of the unmodified binder (PG 64-28) mixtures;

however, the volumetric properties of the modified binder (PG 76-28) were significantly affected by the variation of compaction temperatures.

Moisture Susceptibility

A testing procedure was designed to evaluate the effect of specimen size (150 mm versus 100 mm diameter), short term aging (4 hr at 135°C versus 16 hr at 60°C), compaction method (SGC versus Marshall hammer), and specimen size measured by ratio of diameter to thickness (d/t = 1.6 versus d/t = 3.0). The mixture evaluated in the study was identified by the Kentucky Department of Highways as a mixture with known stripping susceptibility. Results of the study indicated that the SGC yielded higher TSR values than did the Marshall hammer; however, the SGC did correctly identify the Kentucky mixture as moisture susceptible. TSR values for the Kentucky mixture ranged from 0.60 to 0.74 for all combinations of mold diameter, short term aging procedure, and compaction method. Current Superpave specifications require a minimum of 0.80.

Short Term Aging

In order to evaluate the effect of varying short term aging times a mixture with the same asphalt binder type and content and the same aggregate gradation was mixed in the laboratory and aged at 135°C for periods from 0, 0.5, 1.0, 2, and 4 hours. Three specimens at each aging temperature were then compacted in the gyratory compactor to 204 gyrations and their bulk specific gravities were determined. The bulk specific gravity for each sample was then compared to the theoretical maximum density (TMD) which was determined from an average of two specimens at each aging temperature. Results from the study indicate the specimen volumetric properties were affected by the varying aging times. The general trend was as the aging time increases, the G_{mb} decreased and the G_{mm} increased.

Gyratory Compactor Comparison

The testing for the comparison of the gyratory compactors consisted of preparing specimens at the trial asphalt content for each of six aggregate blends. The gyratory compactors evaluated were the Pine SGC, the Troxler SGC, the modified Texas gyratory, and the Rainhart gyratory. Specimens were prepared to determine any differences which exist in the percentage of G_{mm} at $N_{initial}$ (10 gyrations), at N_{design} (100 gyrations), and at $N_{maximum}$ (152 gyrations). The compaction slopes of the different mixtures were also analyzed for differences. The results indicated that there were significant differences between the four gyratory compactors evaluated. The modified Texas and the Pine SGC produced mixtures having lower air voids and lower optimum asphalt contents than did the Rainhart and the Troxler compacted mixtures. In addition the modified Texas and the Pine SGC yielded flatter compaction slopes than did the Rainhart and the Troxler SGC. This indicates that the modified Texas and the Pine SGC did not evaluate the design aggregate structure to be as strong as the Rainhart and the Troxler SGC.

19. Aschenbrener, T., "Comparison of 100 and 150 mm Specimens in the Superpave Gyratory Compactor." Internal Report Summary for Colorado Department of Transportation, 1996.

The Colorado Department of Transportation conducted this study to determine whether differences exist between the volumetric results of 100 and 150 mm diameter specimens. The project consisted of obtaining samples of forty-four mixes used in Colorado and compacting them in the gyratory compactor in both the 100 and 150 mm diameter molds. The samples were compacted to $N_{maximum}$ and their bulk specific gravity was determined for comparison. Results from the project indicated that the average bulk specific gravity of the 100 and 150 mm diameter specimens was 2.347 and 2.336, respectively. Statistical analysis of the results indicated that the two specimen diameters were statistically different. However, this difference results in about 0.4% air voids and approximately 0.2% asphalt content, both of which are not large when compared with typical laboratory repeatability. The conclusion reached by the Colorado DOT was to consider using the 100 mm diameter specimens for use in the Superpave mixture design process.

20. Brown, E.R. et al, Designing Stone Matrix Asphalt Mixtures-Volume II Research Results (Interim Report), Transportation Research Record, TRB, Washington, D.C. 1996.

This research study was designed to develop mixture design procedures for SMA using both the Marshall hammer and the Superpave gyratory compactor. For the study a total of eight different aggregate types were evaluated. The aggregates were chosen to provide a range of particle shapes, absorption, surface textures, and abrasion values. Mixture designs were performed for the eight aggregates using 75, 100, and 125 gyrations of the SGC and 35, 50, and 75 blows of the Marshall hammer. It was determined that 100 gyrations of the SGC shows good correlation with the 50 blow design Marshall procedure and should be used for SMA mixture design.

21. Gowda, G. V., Hall, K. D., and Elliott, R. P., "Arkansas Experience with Crumb Rubber Modified Mixes Using Marshall and Strategic Highway Research Program Level 1 Design Methods." Transportation Research Record 1530, TRB, National Research Council, Washington, D. C., (1996), pp. 25-33.

In this study a total of seven mixes currently used by the Arkansas State Highway and Transportation Department were evaluated. The main objective was to compare the mix design parameters of unmodified and crumb rubber modified mixes using Marshall and Superpave Level 1 design procedures

The aggregate used in all seven mixes was a crushed limestone material, with all mixes having the same gradation. An AC-30 asphalt binder was used for all mixes. The main difference in the mixes was the concentration of crumb rubber and the process of the rubber addition (i.e. wet or dry process). The Marshall mix designs were accomplished using the 75 blow/side procedure for high traffic volume roadways. The Superpave Level 1 designs were performed using values of $N_{initial}$, N_{design}, and $N_{maximum}$ of 8, 96, and 152, respectively. A comparison of the two mix design procedures showed that the Superpave procedure yielded optimum asphalt contents which were between 0.8 and 1.3 percent lower than the Marshall procedure. In addition, none of the Superpave mixes met the minimum VMA criteria, while all of the Marshall mixes had adequate VMA.

22. Gowda, G. V., Hall, K. D., Meadors, A. L., and Elliott, R. P., "Critical Evaluation of Superpave Volumetric Mix Design Using Arkansas Surface Course Mixes." Proceedings of the Association of Asphalt Paving Technologists, Volume 66, 1997, pp. 250-276.

The state of Arkansas evaluated a total of eight typically used surface mixes using Superpave volumetric mixture design. The objectives were to determine the influence of varying levels of gyratory compaction on the volumetric properties of the asphalt mixtures and to study the variation in optimum asphalt content requirements relative to varying levels of traffic. The mixtures evaluated consisted of four surface course mixtures each with two asphalt binders (PG 64-22 and PG 76-22). The aggregates used in the study consisted three granite aggregates and one sandstone aggregate. All mixtures were designed using the gyratory compactor to a N_{max} of 288 gyrations, which is the highest compaction level currently specified. Afterwards, the bulk specific gravities of the mixes were back calculated based on the recorded specimen heights for the remaining 27 levels of N_{max}. The approach assumed that the back-calculation of specimen densities was linear in nature. The eight mixes were designed for 4 percent air voids, with the remaining volumetric properties verified at the design asphalt content. The percent compaction was also verified at the compaction levels of $N_{initial}$ and $N_{maximum}$.

Statistical analysis were performed to determine whether there were differences between the mix design parameters at design gyration levels differing by only one to two gyrations. The design levels of gyrations selected for comparison were as follows: (76,78), (82,83), (93,95), (95,96), (105,106) and (119,121). A second analysis was performed to determine if a statistical difference existed between the mean mix design properties of the eight mixes at seven traffic levels for the four temperature classifications. Observations made in the study was that for the eight mixes evaluated the reduction in optimum asphalt content (OAC), with increased gyratory compactive effort (N_{design} from 68 to 172 gyrations), ranged from 0.75 to 1.2 percent. For all mixes the OAC remained relatively constant at compaction levels greater than 90-110 gyrations. Voids in Mineral Aggregate (VMA) of the mixes also was reduced with increased gyrations up to approximately 90-110 gyrations, where it then remained constant. Conclusions reached from the study were given as follows:

1. The OAC and VMA requirements of the mixes tended to decrease with an increase in the design gyratory compactive effort to approximately 90-110 gyrations. At greater compactive efforts the parameters remain constant over the range of gyrations investigated.
2. Mix designs for design gyration levels (representative of exponential increases in traffic level) which differ by 1 to 2 gyrations are not statistically different.
3. The range of temperature environment selected in the Superpave mix design system are

narrow and do not produce mix designs which differ statistically.

23. Button, J. W., Fernando, E. G., Crockford, W. W., and Coree, B., "Design and Evaluation of Large Stone Asphalt Mixtures", Proceedings of the International Conference on Asphalt Pavements, Seattle, Washington. Volume 2, 1997, pp 335-339.

A new mix design procedure for large stone mixtures with maximum aggregate sizes ranging from 25 mm to 63 mm is presented in this study. The goal of the research was to provide a mixture design procedure that would allow for the applied load to be carried by the internal stone skeleton of the mixture. Further, the strength of the skeleton would be provided mainly by the largest size stockpile material used in the mixture. To evaluate this mix design method, specimens were tested in the Superpave shear tester with the repeated simple shear at constant height test procedure and with the uniaxial test procedure.

The preparation of specimens in the study was done by two methods: field coring and compaction in the Superpave gyratory compactor at an angle of 5 degrees. The researchers were unable to adequately compact samples at the Superpave recommended angle of 1.25 degrees. The increased stone on stone contact or harshness of the mixture was given as the probable reason for the inability of the gyratory compactor to prepare the specimens. AASHTO PP-3 (Practice for Preparing Hot Mix Asphalt (HMA) Specimens by Means of the Rolling Wheel Compactor) or a higher gyration angle in AASHTO TP-4 (Method of Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor) was recommended for the compaction of the large stone mixtures in this study.

24. McGennis, R. B., "Evaluation of Materials From Northeast Texas Using Superpave Mix Design Technology." Transportation Research Record 1583, TRB, National Research Council, Washington, D.C., (1997), pp. 98-105.

Asphalt overlay performance problems, in the form of disintegration, raveling, and stripping, in the state of Texas prompted an evaluation of the mixes by the South Central Superpave Center (SCSC). The cause of the pavement problems was identified as extreme moisture susceptibility of the asphalt mixtures, caused by local gravel aggregates. The evaluation by the SCSC was to determine whether the current Superpave specifications for mineral aggregates and mixture design would have identified the mixtures as being marginal in nature in other ways than moisture sensitivity.

A total of six aggregate materials were selected for the evaluation: (2) coarse gravels, (1) crushed fine aggregate from the same source as the coarse gravels, (2) local field sands of marginal quality, and (1) field sand of good quality. The study evaluated mixtures which generally conformed to the requirements of a Texas DOT Type C mix. This is a 19 mm nominal maximum size mixture with a fine dense graded gradation, and referred to as "sand mixes". In the "sand mixes" the coarse aggregate was held constant and the percentages of the three field sands were varied from 10 to 20 percent, thus yielding 6 total mixtures. For comparison purposes, mixtures consisting of coarse graded gradations of the crushed gravel only (no sand) were evaluated. Crushed limestone screenings and a high quality natural sand were also used in these comparison mixtures. Superpave mixture evaluation consisted of compacting specimens to levels of $N_{initial}$,

N_{design}, and $N_{maximum}$ of 8, 96, and 152 gyrations, respectively. The compaction data for all of the mixtures was normalized at 4 percent air voids. The analysis focused on evaluating the compaction slope, $\%G_{mm}$ at $N_{initial}$, voids in mineral aggregate (VMA) at N_{design}, and the dust proportion of the mixtures. Analysis of the testing results indicated that the sand mixtures all exhibited lower compaction slopes than the other coarse graded mixtures tested. Four of the six mixtures containing field sand failed the Superpave requirement of a minimum achieved density of 89 percent G_{mm} at $N_{initial}$, while all of the coarse graded mixtures met this requirement. For the most part, all mixtures tested met the minimum VMA requirement for a 19.0 mm nominal maximum size mixture. Three of the six sand mixes and all of the coarse graded comparison mixes failed to meet the dust to asphalt ratio requirement set forth under Superpave. Based upon these test results, it is obvious that the Superpave mixture design would have found 4 of the 6 sand mixes to be unacceptable. Also, none of the coarse graded comparison mixtures, as tested, would have met the Superpave requirements due to the dust to asphalt ratio being too low.

25. Takallou, H. B., Bahia, H. U., Perdomo, D., and Schwartz, R., "Use of Superpave Technology for Design and Construction of Rubberized Asphalt Mixtures." Transportation Research Record 1583, TRB, Washington, D.C., (1997), pp. 71-81.

The demonstration of the use of Superpave for the design and construction of rubber modified asphalt concrete (RUMAC) is provided in this study. The goal was to develop a mixture design procedure using Superpave volumetric design and the California Department of Transportation (CALTRANS) requirements for mixtures containing crumb rubber. The initial aggregate gradation was gap graded and was coarser than Superpave requirements on the 2.36 mm sieve. This initial gradation, when designed using the Superpave procedures at $N_{design} = 86$, yielded an optimum asphalt content of 8.3 percent. Past designs with similar gradations had shown around 7 percent to be the optimum asphalt content. Because of this, further research was performed to determine whether the increased asphalt content was a result of the Superpave design procedure. The research included evaluating the effect of varying the short term aging and compaction temperature and comparing the Superpave gyratory with the Marshall hammer, which was the compaction device used to design the previous RUMAC mixtures.

To evaluate the effect of temperature on the mixtures, specimens were compacted in the gyratory compactor at temperatures of 140°C and 146°C, and compared to the results of mixtures prepared at the specified compaction temperature of 138°C. The results showed that increasing the temperature had little effect on the densification rate or the volumetrics of the mixtures. An evaluation of the effect of the aging time was also performed. This consisted of preparing specimens at 7 percent asphalt content with and without the Superpave specified aging time of 4 hours at 135°C and compacting at 146°C. The results showed, unexpectedly, that the aged mixtures exhibited slightly lower air voids than unaged mixtures. Because of these results, temperature was determined not to be the major factor in the elevated optimum asphalt content determined with the Superpave procedure. The research then turned to comparing the gyratory with the Marshall hammer. Specimens were prepared using the 75 blow Marshall procedure at 6.0 and 7.0 percent asphalt content and compared to specimens compacted in the gyratory compactor at the same asphalt contents to N_{design} levels of 86 and 109 gyrations. The results showed that the Marshall specimens were less sensitive to increases in asphalt content than the gyratory compacted specimens. Also it was shown that neither the Marshall nor the gyratory compactor could achieve 4 percent air voids at 7 percent asphalt content, the content which many felt was correct based upon experience. Because of these test results, the gradation was changed from being very gap graded to one that followed the Superpave gradation requirements, mainly by raising the percent passing the 2.36 mm sieve. This mixture, when designed using the Superpave procedures, yielded an optimum asphalt content of 7.1 percent.

26. Jones, C. Correspondence (letter), Superfos Construction (U.S.), Inc. Research and Development Study No. 97-10, 1997.

Superfos Construction (U.S.) conducted a study to determine the effect of long and short term aging times on the theoretical maximum densities of hot mix asphalt mixtures. The research study was in response to variations being prevalent between lab mix design and field obtained samples. The main goal of the study was to determine whether the theoretical maximum density (TMD) obtained in the laboratory was representative of the TMD of plant produced material. The mixture evaluated consisted of crushed gravel, limestone screenings, natural sand, recycled asphalt pavement; with approximately 5.5 percent asphalt binder content. Samples of the produced mixture were taken at the asphalt plant slat conveyor and at the paver. Additionally, mixture components were obtained and the mixture reproduced in the laboratory. A total of 10 samples of each of the mixtures were evaluated. The evaluation consisted of aging two replicates of each mixture for 0, 1, 2, 3 and 4 hours. Afterwards, the TMD was determined for all of the mixtures. The results of the study indicated that the unaged lab specimens had TMDs which were closer to the actual produced mixtures (both plant and paver) than those lab specimens aged for 1-4 hours. This indicates that for this mixture and asphalt plant operation, no short term aging during the mix design procedure would provide TMDs which were closer to the actual field mixture than aging for the 2 or 4 hours at the standard 135°C.

27. Lukefahr, E., "Investigation of Correction Factors in Back-Calculation of Density Using the Superpave Gyratory Compactor." Preliminary Report Summary, Texas Department of Transportation, 1997.

The Texas Department of Transportation investigated the effects of using a correction factor to back calculate densities in the Superpave mix design procedure. A total of four mixtures were evaluated in the study: three-12.5 mm plant produced mixtures consisting of two limestone and one gravel mixtures; and also a 19.0 mm laboratory mixture used by the South Central Superpave Center at the University of Texas. For each mixture two specimens were compacted in the gyratory compactor at each of 80, 90, 100, 110, 140, 150, 170, 190, 220, and 288 gyrations. These levels of gyrations were referred to in the study as $N_{maximum}$. Specimen densities were then back calculated for 80, 90, and 100 gyrations for each of the above $N_{maximum}$ levels. Two of the four mixtures, the gravel and the laboratory mixture, exhibited errors between the measured and corrected densities of approximately 1.1 percent at high levels of $N_{maximum}$ including 190 and 220 gyrations. The other two limestone mixtures had errors of approximately 0.4 percent at the same gyration levels. Conclusions reached from the study were that the current back-calculation procedure yields design target asphalt contents within reasonable expectations, although better accuracy could be obtained by not back-calculating. It was also stated that for production control, a correlation of the measured bulk specific gravity at N_{design} should be required as a minimum; and that production correction factors may be sensitive to slight changes in the asphalt binder content. This could lead to multiple correction factors being used at the plant corresponding to the measured asphalt binder content.

28. Anderson, R. M., and Bahia, H. U., "Evaluation and Selection of Aggregate Gradations for Asphalt Mixtures Using Superpave." Transportation Research Record 1583, TRB, National Research Council, Washington, D.C., (1997), pp. 91-97.

This paper reports the results of a two phase study in which, (1) guidelines for achieving acceptable VMA requirements for Superpave mixtures are provided, and (2) the relationship between properties determined during the Superpave volumetric mix design procedure and material component properties. Only the second phase of the study will be reported in the following summary.

For the completion of the second phase of the study, four aggregate blends were evaluated. These blends are given as follows:

1. Coarse (below the restricted zone), S-shaped, all crushed granite aggregate
2. Fine (above the restricted zone), all crushed granite aggregate
3. Intermediate (through the restricted zone), all crushed granite aggregate
4. Coarse (below the restricted zone), S-shaped, slightly humped fine gradation with 20 percent natural sand.

All of the aggregate blends were mixed with a PG 64-22 asphalt binder and compacted in the Superpave gyratory compactor to an N_{design} of 109. All blends evaluated met all Superpave design requirements.

More specifically, the second phase of the study was interested in possibly correlating the slope of the gyratory compaction curve with mixture analysis testing results. These mixture analysis tests included the mixture stiffness and phase angle as determined by the frequency sweep test and the mixture strain and strain slope as determined by the repeated simple shear at constant height test. Both of these tests were performed with the Superpave Shear Tester (SST). It is thought that the slope of the densification curve provides an indication of the shear resistance of the asphalt mixture. Conclusions reached from the study were given as follows:

1. The slope of the compaction curve differentiates between different aggregate gradations. Finer gradations and those with rounded aggregates exhibited lower compaction slopes.
2. The idea that mixtures with lower compaction slopes (i.e. finer gradations with natural sand) had weaker aggregate structure was not shown by the Superpave mixture analysis testing.

29. McGennis, R. B., Perdomo, D., Kennedy, T. W., and Anderson, V. L., "Ruggedness Evaluation of AASHTO TP-4 The Superpave Gyratory Compactor." Proceedings of the Association of Asphalt Paving Technologists, Volume 66, 1997, pp. 277-311.

The results of the ruggedness evaluation of "AASHTO TP-4 - Standard Method of Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor". This evaluation was designed using the principles of "ASTM C 1067-87, Conducting a Ruggedness or Screening Program for Test Methods for Construction Materials". The main objectives of the experiment was to identify the factors in the test procedure that cause a significant source of variation in testing results, and to determine the controls necessary for these factors in the test specification. The experiment was constructed with seven main factors. ASTM C 1067-87 dictates that each of the main factors be evaluated at a high and low value. The factors and their values are provided in Table 2.1.

Table 2.1 Main Factors and Levels Evaluated in the AASHTO TP-4 Ruggedness Experiment

Main Factor	Low Level	High Level
Gyration Angle, degrees	1.22 - 1.24	1.26 - 1.28
Mold Loading Procedure	Transfer Bowl Method	Direct Loading Method
Compaction Procedure, kPa	582	618
Precompaction	None	10 Thrusts w/ Standard Rod
Compaction Temperature, °C	at 0.250 Pa-s viscosity	at 0.310 Pa-s viscosity
Specimen Height, mm	approximately 110 mm	approximately 120 mm
Aging Period at 135°C, hrs.	3.5	4.0

Gyration Angle

The current AASHTO TP-4 specified gyration angle is 1.25 ± 0.02 degrees. It would seem obvious that the low and high levels for analysis should be 1.23 - 1.25 degrees, and 1.25 - 1.27 degrees. However, it is extremely time consuming, with the compactors evaluated in this experiment, to set the angle to exactly 1.23 or 1.27 degrees. Therefore, the values presented in Table 2.1 were selected as the levels to allow for the possible variation occurring in the angle adjustment and setting.

Mold Loading Procedure

TP-4 does not specify a method of loading or "charging" the gyratory mold. Because no method is specified, it was anticipated that many different methods would be used for mixture loading. Therefore, two extreme cases were chosen for evaluation: the "gyro loader", which loads the mold in a single drop; and the scoop method, which loads the mold in many drops.

Compaction Pressure

TP-4 requires a vertical consolidation pressure of 600 kPa ± 3 percent (18 kPa). Therefore, the low and high levels were chosen to be 582 and 618 kPa, respectively.

Pre-compaction

There is no mention of pre-compaction or rodding of the mixture in TP-4. However, many technicians with Marshall experience are accustomed to rodding the mixture 25 times prior to compaction. Also, a significant amount of the SHRP research was accomplished by pre-compacting the mixture in the mold with ten thrust of a small scoop. To account for the fact that some operators of the gyratory compactor may pre-compact the mixture, levels of no pre-compaction and ten thrusts of a standard concrete slump rod were chosen for evaluation.

Compaction Temperature

TP-4 requires that mixtures be compacted with a temperature range resulting an asphalt binder viscosity between 0.250 and 0.310 Pa-s. For the binder used in the experiment, temperatures of 141°C and 146°C met this criteria and were selected as the low and high levels.

Specimen Height

A majority of the SHRP research was conducted on specimens with a nominal height of 115 mm. The initial tolerance on specimen height was ±1 mm. This tolerance was considered to restrictive for the experiment and a tolerance of ±5 mm was selected. Therefore, levels of specimen height, after compaction to N_{max}, were 110 and 120 mm.

Aging Period

TP-4 and AASHTO PP2 "Standard Practice for Short and Long Term Aging of Hot Mix Asphalt (HMA)" require that samples be aged for four hours at 135 ° C. After the 4 hours are complete, the sample is then placed into another oven for a variable amount of time, not to exceed 30 minutes, to reach compaction temperature. However, during SHRP, another procedure was utilized. This procedure incorporated the additional oven time into the four hour short term aging period. Therefore, the two levels of aging time selected were (1) to place the mixture in the aging oven for four hours plus a fixed 30 minutes at compaction temperature and (2) to place the mixture in the aging oven for 3.5 hours plus a fixed 30 minutes at compaction temperature.

In the experiment both the Troxler and the Pine gyratory compactors were utilized. A total of six laboratories were selected to participate in the experiment. Three laboratories used the Troxler gyratory compactor and three used the Pine compactor. However, one of the Troxler laboratories was unable to complete the study and the experiment continued with five labs.

The materials used in the experiment consisted of crushed limestone and crushed river gravel aggregates and a PG 64-22 asphalt binder. A total of four gradations were selected for evaluation and are given below:

Mix 1: S-shaped gradation (below the restricted zone), primarily comprised of crushed limestone

Mix 2: Same as Mix 1, but comprised of crushed gravel
Mix 3: Fine gradation (above the restricted zone), primarily comprised of crushed limestone
Mix 4: Same as Mix 3, but comprised of crushed gravel

All samples in the study were mixed at the Asphalt Institute and sent out to the participating laboratories for uniformity in the experiment. After extensive analysis of the experimental data, the following conclusions and recommendations were stated:

1. The compaction angle tolerance of ± 0.02 degrees is reasonable.
2. For mold loading, a transfer bowl is preferable, but is not necessary.
3. Pre-compaction using the standard rod did not significantly affect the results.
4. A specimen height of ±1 mm is too narrow.
5. For binders similar to the PG 64-22 used in the study, the 30 minute compaction temperature equilibrium period can be included in the four hour short term aging period.

30. McGennis, R., "Evaluation of Various Superpave Gyratory Compactors." Summary from South Central Superpave Center Website (www.utexas.edu/research/superpave/articles/copreval.html), 1998.

Since the development of the Pine and the Troxler Superpave gyratory compactors in 1995, several other manufacturers have begun to develop their own gyratory compactors. Because of these new compactors, there was a need for a standard protocol for evaluating and judging the conformance of the compactors to the Federal Highway Administration's specification for the Evaluation of Superpave Gyratory Compactors. The compactor evaluation consists of a full factorial experiment in which the new gyratory compactor is compared with either the Pine or the Troxler compactor, which originally met the FHWA gyratory specification.

Three new gyratory compactors from Test Quip, Inc., Pine Instrument Company, and Interlaken Technology Corporation, were evaluated using the protocol. A total of four mixtures, given below, were used in the completion of the gyratory evaluation.

1. 12.5 mm coarse (below the restricted zone), crushed limestone aggregate
2. 19.0 mm coarse (below the restricted zone), crushed limestone and sandstone
3. 19.0 mm fine (above the restricted zone), crushed limestone and sandstone
4. 25.0 mm coarse (below the restricted zone), crushed limestone

A PG 64-22 asphalt binder was used for all of the project mixtures. The short term aging time and temperature was 4 hours and 135°C. Mixing temperature was 150°C, while the compaction of the mixtures was done at 135°C to reduce the variability in the mixtures. The Pine gyratory compactor was used as the reference compactor for the completion of this protocol. After specimens were compacted, three response variables, the G_{mb} and $N_{initial}$, N_{design} and $N_{maximum}$, were chosen for statistical analysis. To be considered to be comparable with the Pine gyratory compactor, the new gyratory compactors had to produce mixtures whose average G_{mb} values were within 0.010 of those obtained with the Pine.

The analysis of the results indicated that all three of the new compactors compared favorable with the Pine gyratory compactor. Conclusions reached from the evaluation were that the FHWA gyratory compactor protocol is a valid means of determining the acceptability of new gyratory compactors which are introduced to the market.

31. Vavrik, W. R., and Carpenter, S. H., "Calculating Voids at a Specified Number of Gyrations in the SUPERPAVE Gyratory Compactor." Paper presented at the 1998 Transportation Research Board Annual Meeting, Washington, D. C.

In response to inaccuracies, in both mix design and quality control testing, resulting from the back calculation of gyratory specimen density at N_{design} from densities obtained at $N_{maximum}$, the University of Illinois undertook a study to determine the extent of this problem.

At the present time the Superpave system uses a back calculation method in which the specimens density at N_{design} is determined through the use of the specimen height and a correction factor determined at $N_{maximum}$. This correction factor is distinct for each mixture designed and will vary with asphalt content, gradation, and compactive effort.

The mixture used for this evaluation was a 19.0 nominal maximum size dense graded mixture, with a gradation below the Superpave restricted zone and near the coarse control points. All aggregate consensus and source properties were met by the aggregate gradation. The procedure consisted of compacting one specimen to N_{design} and one to $N_{maximum}$. The density of the specimens compacted to N_{design} were then compared to those back-calculated from $N_{maximum}$. The results showed
air void differences between 0.5 to 1.5 percent for those samples.

Due to these discrepancies, the state of Illinois developed a method of determining the densification properties of a mixture based on analyzing all of the gyratory height data and the densification curve for a given mixture. In the procedure it is stated that the densification curve for a mixture is generally linear in nature up to the point of 96 percent of G_{mm} or 4 percent air voids. The majority of the back calculation error actually occurs as the void level drops below 4 percent air voids. The Illinois method utilizes a "locking point" concept. This "locking point" is referred to as the first of three consecutive gyrations producing the same specimen height. Generally, the densification rate of the mixture is nonlinear at any further gyration levels. The "locking point" concept was developed by Illinois to prevent the over compaction of their designed mixtures. The procedure determines the locking point of the mixture and stops compaction at that level, which will more adequately determine the specimen densities at prior levels of compaction. In a sense the "locking point" will actually become a version of the now specified $N_{maximum}$ gyration level. Values of 100, 75, and 50 gyrations were specified as typical values to provide 96 percent G_{mm} for high, medium, and low volume traffic pavements. The procedure consists of compacting specimens up to the "locking point" and then determining, by regression analysis, the number of gyrations to provide 96 percent of G_{mm}.

To test the procedure, a variety of mixtures used by the Illinois Department of Transportation on Superpave demonstration projects were evaluated. These mixtures varied in gradation, size, aggregate type, design compactive effort, and asphalt binder type (polymer and unmodified). The results of the evaluation showed that the Illinois method showed as a whole to be more accurate than the Superpave method. Also shown, was that smaller nominal maximum size mixtures yielded more consistent results than larger mixtures.

32. Mallick, R. B, Buchanan, M. S., Brown E. R. and Huner, M. H., "Evaluation of Superpave Gyratory Compaction of Hot Mix Asphalt." Paper presented at the 1998 Transportation Research Board Annual Meeting, Washington, D. C.

The objective of the study was to compare the correction factors obtained at different gyration levels during the compaction of HMA and to evaluate the change in the correction factor with gyrations. To complete the study, a traprock aggregate was used in two very different gradations: a stone matrix asphalt (SMA) gradation and a conventional well-graded or dense gradation. A PG 64-22 asphalt binder was used for all mixture evaluation. Mixtures were prepared at their respective optimum asphalt content in the Pine gyratory compactor at different gyration levels. The gyration levels used were as follows: Dense Mixture: 27, 46, 66, 85, 97, 109, 120 and 132 gyrations, and SMA: 40, 71, 101, 132, 153, 174, 194, and 215 gyrations. After the specimens were compacted, their bulk specific gravities and correction factors were determined. Next, specimens were compacted to the maximum level of gyrations used in the above procedure (i.e. 132 for the dense, and 215 for the SMA) and their bulk specific gravity and correction factors determined. The densities at lower levels of compaction were then back-calculated by using this correction factor at the highest gyration level and compared with those determined previously. The results of the analysis showed that the correction factor used to back-calculate specimen densities are not constant at different gyration levels for the mixtures evaluated. For the dense graded and SMA mixtures studied, the correction factor was found to decrease and become almost constant at higher gyration levels. As expected, the coarser mixture (SMA) exhibited the greatest difference between the back-calculated and the actual specimen densities. Also concluded was that at lower gyrations the densities of compacted specimens were greater than the densities which were back-calculated from a correction factor determined at a maximum level of gyrations. The recommendation from the study was to compact specimens to N_{design} in the volumetric mix design procedure. Volumetric properties could still be checked at $N_{initial}$ on the basis of a correction factor obtained at N_{design}. After the design is complete, specimens could be compacted to $N_{maximum}$ in order to check the $N_{maximum}$ requirement.

33. Habib, A., Hossain, M., Kaldate, R., and Fager, G. A., "Comparison of Superpave and Marshall Mixtures for Low Volume Roads/Shoulders." Paper presented at the 1998 Transportation Research Board Annual Meeting, Washington, D. C.

An evaluation of the Superpave and Marshall mix design procedures was performed to study the effectiveness of using a Superpave mix design in place of a Marshall mix design for a low volume roadway. The project chosen for the evaluation was a low volume section of State Route 177 in northeast Kansas. The research was needed because of the high percentage of low volume roadways present in the state of Kansas. The materials used in the study consisted of a crushed limestone coarse aggregate (CS-1) and coarse (SSG-1) and fine (SSG-2) river sands as the fine aggregates. A manufactured sand fine aggregate (MS-1) was also used for a small portion of the study. In the study, a total of five blends were evaluated. All blends were coarse graded 19.0

nominal maximum size and kept the same percentage of CS-1 (66 percent). The blends varied the amount of the SSG-1 aggregate from 5 to 20 percent, in increment of 5 percent. Table 2.2 shown the blends evaluated in the study. Fine aggregate angularity tests for the fine aggregate showed values of 39 and 40 for the SSG-1 and the SSG-2, respectively.

The Superpave mix designs were performed at the lowest level of traffic (0.3 million ESALs) and climate (< 39C) that is currently specified: $N_{initial}$ = 7, N_{design} = 68, and $N_{maximum}$ = 104. These designs were compared to Marshall 50 blow per side designs. The testing results indicated that all the Superpave mixtures met the requirements for VMA, $\%G_{mm}$ at $N_{initial}$, and $\%G_{mm}$ at $N_{maximum}$. All mixtures, with the exception of blend 1, failed the Superpave VFA requirement. Blends 3, 4, and 5 failed the dust to asphalt ratio on the low side. Evaluation of the compaction slopes of the compacted mixtures, showed that blend 1 had the steepest slope, which would indicate the strongest aggregate structure. Marshall mixture designs were also performed for each of the five blends. The optimum asphalt content for the Superpave and the Marshall mix designs was chosen to provide 4

Table 2.2 Aggregate Blends for Kansas 177 Evaluation

	Aggregate Percentages (%)				
Aggregate	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5
CS-1	66	66	66	66	66
SSG-1	8	5	10	15	20
SSG-2	21	29	24	19	14
MS-1	5	-	-	-	-

percent air voids in the mixture. An analysis of the mix design results showed that optimum asphalt contents determined in the Superpave designs were from 0.6 to 0.8 percent lower than for the 50 blow Marshall designs. Because the Superpave mixtures had lower optimum asphalt contents, there was concern about their durability, especially on relatively low volume roadways. This was further substantiated by the fact that four out of the five blends failed to meet the Superpave VFA criteria. These results indicate that the VFA criteria may not be applicable for low volume roadway mixture design. The authors stated a theory, which is speculated, that the current levels of N_{design} are approximately 20 percent too high for use. Therefore, a volumetric analysis of the previously conducted mixture design at 68 gyrations, was performed at a N_{design} level of 54 gyrations, which is 20 percent less than used initially. This resulted in new levels of N_{inital} and $N_{maximum}$ of 6 and 80, respectively. The analysis showed that the optimum asphalt content, VMA, and VFA values all increased, as expected. However, blends 2, 3, and 4 still failed the VFA criteria. Conclusions reached from the study were that Superpave mix designs yield lower optimum asphalt content than Marshall 50 blow designs for the same aggregate and gradation, river sands have the potential to be used for low volume roadways, Superpave VFA requirements for low volume traffic (< 0.3 million ESALs) appear to be too high, and the lowering of the N_{design} value yields increased optimum asphalt content.

34. Bahia, H. U., Frieme, T. P., Peterson, P. A., Russell, J. S., and Poehnelt, B., "Optimization of Constructibility and Resistance to Traffic: A New Design Approach for HMA Using the Superpave Compactor." Paper presented at the 1998 Annual Meeting of the Association of Asphalt Paving Technologists, Boston, Massachusetts.

This study was conducted in an effort to determine a method to utilize the gyratory compaction data to optimize the densification characteristics under construction and traffic. More specifically, the objectives were to develop a new concept to use densification energy indices to evaluate HMA performance under construction and in-service and to evaluate the effect of aggregate gradations and fine aggregate angularity used in Superpave on the densification characteristics of HMA. The idea used by the authors is that the gyratory compactor design allows the the calculation of the energy required to change the volume of an asphalt mixture. This energy or work represents the area under the densification curve.

The following variables were controlled in the study and are given as follows:

1. Aggregate: All aggregates used conformed to Superpave consensus property requirements.
2. Asphalt binder: A PG 58-28 binder was used for the entire study.
3. Traffic Level (ESALs): Traffic levels corresponding to Wisconsin Department of Transportation (WisDOT) high volume (HV) and medium volume (MV).
4. Asphalt binder content: Samples were mixed at three contents around the optimum asphalt content for each aggregate blend. One sample of each blend was mixed and compacted at 5 percent asphalt content to determine the densification variability at a constant asphalt content.
5. Compactive Effort: The HV mixes were compacted to N_{max} = 150 gyrations and the MV compacted to N_{max} = 129 gyrations. Two samples were compacted for each blend; one to N_{design} and the other to N_{max}.
6. Aggregate Gradation: A total of six blends were evaluated for both the HV and MV traffic levels in the study. These blends ranged from above the restricted zone to below the restricted zone.

The 12 mixtures were compacted and their compaction data obtained and used to calculate various volumetric and densification characteristics. These characteristics were divided into mixture volumetrics, densification rate indicators, and densification energy indices. An analysis of the volumetric properties of the mixtures showed the following:

1. Mixtures with higher $\%G_{mm}$ at $N_{initial}$ do not necessarily show higher $\%G_{mm}$ at N_{max}. In fact, the opposite seems to hold true.
2. Values of $\%G_{mm}$ at $N_{initial}$ are very close or beyond the maximum limit of 89 percent of G_{mm} for blends above and through the restricted zone for both the HV and the MV mixtures. Blends below the restricted zone are well below the 89 percent limit.
3. The $\%G_{mm}$ at N_{max} is close to the limit of 98 percent for all blends. Coarser mixtures are closer to the limit than the finer mixtures, and are equal to or closer than the blend through the restricted zone. This would indicate that coarser mixtures would offer less protection to densification beyond the two percent air void limit.

An analysis of the densification characteristics of the mixtures showed that the densification slopes between $N_{initial}$ and N_{design} are between 6.2 and 6.7 for the HV mixtures above the restricted zone, opposed to slopes of 8.1 to 9.8 for HV mixtures below the restricted zone. The same holds true, but with lower values, for the MV mixtures.

35. Sombre, R. D., Newcomb D. E., Chadbourn, R., and Voller, V., "Parameters to Define the Laboratory Compaction Temperature Range of Hot Mix Asphalt." Paper presented at the 1998 Annual Meeting of the Association of Asphalt Paving Technologists, Boston, Massachusetts.

The objective of this study was to define the temperature range in which the desired density of an asphalt mixture can be obtained. The gyratory compactor was used in the study to compact samples at different temperatures. The total work required to compact the samples was then determined through use of the sample height, sample size, and applied vertical pressure during compaction. The author recognized that the ability of an asphalt mixture to be compacted depends upon the cohesion and the angle of internal friction of the mixture.

In the study, both laboratory and field produced asphalt mixtures were evaluated. The mixtures consisted of six laboratory mixtures (three asphalt binders and two aggregate structures), and five different field mixtures (two dense graded and three coarse Superpave mixtures). The gyratory compactor used in the study was an Intensive Compaction Tester (ICT) produced by Invelop Oy of Finland. This compactor is very similar to the Superpave gyratory compactor, in that it operates on a shear compaction principle.

The laboratory work consisted of three asphalt binder grades (Pen 120/150, pen 85/100, and an AC-20) with two gradations (dense and gap graded or SMA). The aggregates consisted of a crushed granite coarse aggregate with a rounded pit-run gravel fine aggregate. For the laboratory mixtures, 1200 gram size samples were prepared and compacted in 100 mm molds. It was recognized that Superpave specifications require 150 mm diameter specimens; however, the laboratory air pressure was not adequate for the larger specimen preparation.

Samples of each of the five field mixtures were taken directly behind the paver and compacted in the ICT compactor, and compacted in the laboratory at temperatures between 70°C and 140°C, depending upon the temperature of the mixture at the time of sampling. The temperature of the field mixture was also determined throughout field compaction after every roller pass. The laboratory compacted field mixtures were 150 mm diameter specimens. This was possible due to an upgrade in the laboratory air pressure since the laboratory work was performed.

The next step was to determine the maximum shear stress and power for each compacted sample. The results of the study indicate that the shear stress in a laboratory sample increases with decreasing temperature and decreases with increased aggregate coarseness and angularity. Further, the shear stress of mixtures prepared with the same binder grade can vary greatly depending upon the type of polymer modifier used. Also, the power needed to compact field produced and laboratory prepared asphalt mixtures depends upon the aggregate type and gradation used, with the asphalt binder grade and temperature having little effect on the required power.

36. Anderson, M. R., Cominsky, R. J. and Killingsworth, B. M., "Sensitivity of Superpave Mixture Tests to Changes in Mixture Components." Paper presented at the 1998 Annual Meeting of the Association of Asphalt Paving Technologists, Boston, Massachusetts.

The purpose of this study was to determine if laboratory changes in mixture components results in significant mixture property (volumetric and mechanical) changes. To complete the study, the Superpave gyratory compactor was used to evaluate volumetric changes and the Superpave shear tester (SST) for the mechanical properties. The volumetric properties determined from the gyratory compactor included the percent air voids at N_{design}, the percentage of G_{mm} at $N_{initial}$ and $N_{maximum}$, and the densification slope. The experiment consisted of varying a number of parameters from one baseline asphalt mixture, a 19.0 nominal maximum size blend of crushed limestone and natural sand with a PG 64-22 asphalt binder. Specifically, two levels from the baseline values of each of the following were chosen for evaluation: asphalt binder content (± 0.5 percent), coarse aggregate gradation (± 6 percent), intermediate aggregate gradation (± 4 percent), fine aggregate gradation (± 2 percent), and ratio of natural sand to crushed sand (± 10 percent). Because of the large scale of the study, a 1/4 fractional factorial experiment was conducted. Specimens were compacted to $N_{maximum}$ (152 gyrations) in the SGC in accordance with AASHTO TP4 compaction protocol. All mixtures were aged for 4 hours at 135°C prior to compaction.

The results of the study indicate that the interaction of asphalt content and fine gradation had the most significant effect on the volumetric and densification properties. The main effect of coarse aggregate gradation, the main effect of asphalt content, the interaction of asphalt content and fine gradation, and the interaction of asphalt content and coarse gradation all significantly affected the % G_{mm} at $N_{initial}$. Also the densification slope was affected by the fine gradation, the intermediate gradation, the interaction of asphalt content and coarse gradation, and the interaction of asphalt content and fine gradation. It was further shown that asphalt content had an effect on all volumetric and densification properties with the exception of the densification slope.

2.1.1 Literature Review Summary

The following is a brief summary of the literature review as it pertains to items addressed in the research project. Among these items are the gyratory compactor, the short term aging procedure for Superpave mixtures, and the design of large stone and gap graded mixtures with the Superpave gyratory compactor.

2.1.1.1 Gyratory Compaction

Gyratory compaction of hot mix asphalt is not a new concept in the areas of mixture design and process control. The Texas Highway Department developed a method of gyratory compaction in 1939 after a research project designed to determine a better method of compacting hot mix asphalt samples. A gyratory molding machine was selected because it imparted constant shear forces throughout the asphalt mixture sample. The resulting specimen size used by the gyratory molding machine was 50 mm in height and 100 mm in diameter.

The Corps of Engineers' Gyratory Testing Machine (GTM) has also been used for the gyratory compaction and analysis of asphalt mixtures for many years. This device compacts samples using a gyratory kneading procedure. The vertical pressure, angle of gyration, and the number of gyrations can be varied depending upon the type and amount of expected traffic. Typical compaction parameters are a vertical pressure of 827 kPa, a one degree angle of gyration, and 300 revolutions. The GTM has the ability to measure the mixture's shear strength during compaction.

France has also used gyratory compaction for many years for the design of hot mix asphalt. Their concept very closely resembles the Superpave gyratory compactor. Compaction takes place in a 160 mm diameter mold with an angle of gyration of one degree and a vertical pressure of 600 kPa. The rotational speed is held constant at six revolutions per minute. The number of gyrations applied to the sample is determined through the use of an equation which is highly dependent upon the mixture thickness in the field and the type of field compaction.

The Superpave gyratory compactor which was developed under SHRP operates with a vertical consolidation or compaction pressure of 600 kPa, a rotational speed or gyration rate of 30 revolutions per minute, and a constant angle of gyration of 1.25 degrees. Both 100 mm and 150 mm diameter specimens can be prepared in the compactor; however, 150 mm diameter is specified in the provisional AASHTO specification TP-4 "Standard Method of Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the SHRP Gyratory Compactor". A benefit of using the larger specimen diameter is the ability to design mixtures up to 37.5 nominal maximum size and also reduces the density variability of the compacted specimens. However, some literature does suggest that the use of 100 mm diameter specimens could be justified in cases where the nominal maximum aggregate size of the mixture allows.

Results obtained from the initial N_{design} experiment were used to establish compaction levels for Superpave mixtures. A total of 28 levels (seven traffic levels and four high temperature levels) of N_{design} resulted from the study. For each level of N_{design}, values of $N_{initial}$ and $N_{maximum}$ were also established.

Generally, research has shown that for medium to high traffic levels, which result in higher levels of compaction or N_{design}, the gyratory compactor yields higher specimen densities and therefore lower optimum asphalt contents and voids in the mineral aggregate than does the Marshall hammer.
Another study indicated that the gyratory compactor better identified plant production variability than did the Marshall hammer.

Various reports, in which the in-place density of pavements throughout the country have been monitored over time and traffic, conclude that the current N_{design} levels for gyratory compaction are too severe or high for lower volume roadways. Additionally, research indicates that significant differences do not exist between N_{design} levels which differ by only one or two gyrations.

Currently the Superpave mix design procedure requires that specimens be compacted to $N_{maximum}$ and their volumetric properties back-calculated to N_{design} and $N_{initial}$. The literature suggests that this results in errors in the volumetric properties a vast majority of the time. The errors varied depending upon the coarseness and the asphalt content of the compacted mixture. Mixtures with a coarse gradation, such as a Superpave mixture below the restricted zone or a stone matrix asphalt (SMA), have higher errors than did fine graded mixtures.

The slope of the gyratory compaction curve is also believed to provide an indication of the properties of the compacted mixture. Research has shown that the slope differentiates between different aggregate gradations. Finer gradations with rounded aggregates exhibit lower compaction slopes. However, Superpave mixture analysis testing did not support the idea that mixtures with lower slopes had a weaker aggregate structure than did mixtures with higher slopes.

Literature suggests that many mixtures with fine gradations have difficulty meeting the density requirement at $N_{initial}$ of less than 89 percent of G_{mm}. Further, with few exceptions, compacted mixtures meet the density requirement of less than 98 percent of G_{mm} at $N_{maximum}$. Coarse graded mixtures tend to have higher densities at $N_{maximum}$ than do fine graded mixtures.

2.1.1.2 Gyratory Compaction of Gap Graded and Large Stone Mixtures

Research has shown that gap graded mixtures, such as stone matrix asphalt (SMA) can be designed using the Superpave gyratory compactor (SGC). Initial laboratory research from NCHRP 9-8 indicated that approximately 100 gyrations of the SGC shows good correlation with the 50 blow Marshall design and should be used for SMA mixture design. The results of this phase of the study were used to develop a laboratory mix design procedure for

However, recent research results, also from NCHRP 9-8, obtained from the field validation of the laboratory mix design procedure for SMA in eleven projects throughout the country has shown that approximately 80 gyrations better correlates with the 50 blow Marshall procedure.

Literature suggests that large stone mixtures (37.5 mm nominal maximum size) can be successfully compacted and designed in the gyratory testing machine and provide good field performance in very heavy duty, high traffic applications.

Other research suggests that the Superpave gyratory compactor was unable to adequately compact large stone mixtures at the recommended angle of gyration of 1.25 degrees. The increased stone on stone contact and the harshness of the mixture was given as the probable cause.

2.1.1.3 Short Term Aging

The literature pertaining to the short term aging of Superpave mixtures dealt with the development and validation of the short term aging temperature. Mixtures were compacted after being subjected to various temperatures, aging procedures (forced draft oven, pressure oxidation, and triaxial cell aging) and aging times. The compacted specimens were then evaluated in the laboratory using resilient modulus, dynamic modulus, and tensile strength testing. An aging temperature of 135C and an aging time of four hours were selected for the short term aging parameters for Superpave mixtures. These aging parameters were field validated in a number of projects with the results being that 135C for four hours was a good (although conservative for some sites) procedure for the short term aging of Superpave mixtures.

2.2 TASK 2: SURVEY OF USERS OF THE SUPERPAVE GYRATORY COMPACTOR

In order to determine any concerns or problems associated with the gyratory compactor procedure, a questionnaire was sent out to various users of the gyratory compactor. The questionnaire was sent to State DOTs, FHWA Superpave mix expert task group (ETG) members, and various contractors and consultants who have had experience with the gyratory compactor. The recipients of the questionnaire were at various stages of Superpave implementation and were able to provide a wide range of suggestions and comments which aided in the research study. A copy of the questionnaire as it was sent out is provided in Figure 2.2.

NCHRP 9-9 QUESTIONNAIRE

Person Responding:______________________ Phone Number:_________________________

About 9-9: Refinement of Superpave Gyratory Compactor
NCHRP 9-9 is a study to evaluate the gyratory compactor and the compaction process as they exist today. Items to be studied include the following:

- Effect of specimen size, aggregate maximum size, and gradation on mixture volumetrics
- Evaluation and development of Superpave compaction procedures for SMA, open graded friction course, large stone, and gap-graded mixtures,
- Evaluation of compaction parameters for certain dense graded mixtures which meets design requirements and perform badly in the field, or which do not meet design requirements and performs well in the field.
- Evaluation and possible revision of the AASHTO TP-4.

Questions:

The following is a list of questions whose answers will aid in the research study.

1. Do you have any suggestions that may change the gyratory compaction process to provide for better evaluation of asphalt mixtures?

   Yes_____ No_____ Please Comment:

2. Have you had any experience with SMA, large stone, open graded, or gap graded mixture designs using the Superpave gyratory compactor?

   Yes_____ No_____ If yes, please comment:

3. Have you had any experience with mixtures that fail to meet any of the Superpave gyratory compaction requirements (i.e. $N_{initial,}$ $N_{max,}$ VMA) but are known to provide good field performance?

   Yes_____ No_____ If yes, please comment:

4. Have you had any experience with mixtures that meet all the Superpave requirements, but are known to provide poor field performance?

   Yes_____ No_____ If yes, please comment:

5. Have you used the Superpave gyratory compactor to compact 100 mm diameter specimens or specimens with heights other than 115 mm?

   Yes_____ No_____ If yes, please comment:

6. Do you have any other comments, concerning the Superpave gyratory compaction procedure, that you believe will be useful to the research team in accomplishing the objectives of this project?

Figure 2.2 Gyratory Compactor User Questionnaire

Individual responses to each of the questions are provided in Appendix A. The following provides the question along with a brief summary of the responses.

Question 1: Do you have any suggestions that may change the gyratory compaction process to provide better evaluation of asphalt mixtures?

In reviewing the responses to Question 1 it seems evident that generally users of the gyratory compactor seem satisfied with the operation of the compactors. However, some concerns as stated below are prevalent throughout the HMA industry. The most prevalent concerns about the gyratory compaction process as determined through the questionnaire are given as follows:

1. There is a concern that the gyratory is providing more compactive effort than the traffic level warrants, especially for low volume roadways.
2. There is general opinion that there should be *one* specified procedure for the loading of the mixture into the gyratory molds. AASHTO TP4 *(46)* at the present time states "Place the mixture into the mold in one lift." There is concern that different loading procedures or techniques between laboratories may result in possible discrepancies in test results.
3. There were concerns that the back calculation of specimen density at N_{design} from the N_{max} density based upon a correction factor may be in error.
4. There is a desire to use 100 mm specimen size when the nominal aggregate size of the mixture permits.
5. The short term aging temperature of 135°C should be changed to the compaction temperature of the mixture being designed and the aging time be reduced from the four hour period. This would eliminate the need for an additional oven in the laboratory while at the same time providing substantial time savings.

Some additional items mentioned included concern over the current VMA requirements for Superpave mixtures, a need for a method of using the gyratory compaction slope to provide information about the mixtures, and a better method of determining the mixing and compaction temperatures of mixtures for both laboratory design and field process control.

Question 2: Have you had any experience with SMA, large stone, open graded, or gap graded mixtures using the Superpave gyratory compactor?

The responses to Question 2 indicate that the use of the SGC for mixture types other than dense graded mixtures is at the very initial stages. This is to be expected since the use of the different mixture types is not as widespread as dense mixtures and by the fact that the SGC is a relatively new piece of equipment with many questions about its' use to be determined. Of the 14 respondents, 8 indicated that the SGC has been or will be used in the near future to design or field verify SMA mixtures. However, a number of states, including Maryland and Georgia, frequently use the SGC to design and control SMA mixtures, without any major problems.

Question 3: Have you had any experience with mixtures that fail to meet any of the Superpave gyratory compaction requirements (i.e. $N_{initial}$, N_{max}, VMA) but are known to provide good field performance?

The most frequent response to Question 3 was that there are numerous mixtures (mostly Marshall designed) which have a very good performance history, but will not meet the VMA requirements of Superpave volumetric mixture design. The Superpave design procedure has the same VMA requirements as Marshall. The gyratory provides more compactive effort at medium to high N_{design} levels than the Marshall hammer resulting in VMA requirements being more difficult to achieve using the Superpave design system. Another response given was that some mixtures with gradations passing through the restricted zone have good field performance history.

Question 4: Have you had any experience with mixtures that meet all the Superpave requirements, but are known to provide poor field performance?

As thought prior to distributing the questionnaire, there are not an abundance of projects which meet all of the Superpave requirements and exhibit poor field performance. The few projects that were mentioned included projects where stripping and bleeding occurred. According to the respondents these problems may have been attributable to poor construction techniques.

Question 5: Have you used the Superpave gyratory compactor to compact 100 mm diameter specimens or specimens with heights other than 115 mm?

The responses to Question 5 indicate that several people have attempted to produce specimens, having heights other than 115 mm for tensile strength ratio (TSR) testing. Others have attempted to compact specimens to predetermined heights in order to evaluate the specimens in some type of laboratory wheel tracking device.

Question 6: Do you have any other comments, concerning the Superpave gyratory compaction procedure, that you believe will be useful to the research team in accomplishing the objectives of this project?

Upon review of the responses to Question 6 it is once again evident that many people have concern over the design requirements in the Superpave design system. One concern was that the compactive effort (N_{design}), the VMA requirements, and the $\%G_{mm}$ at $N_{initial}$ require very harsh mixtures. It is feared that these mixtures, while being adequate for high traffic roadways, are very conservative for the low traffic roadways. The fact that the SGC typically results in less asphalt content required for a mixture when compared to the Marshall procedure makes some people very uneasy about the stripping potential and durability of the mixture. One respondent stated that they added a small amount of asphalt cement to the designed mixture for purpose.

Another concern was a fear that the current N_{design} levels are too high for many lower volume roadways and should be adjusted accordingly.

CHAPTER 3 RESEARCH TEST PLANS

3.1 INTRODUCTION

This section of the report contains the test plans for each of the laboratory associated tasks for NCHRP 9-9. For each of the tasks, a brief description of the problem will be presented followed by a detailed test plan.

3.2 TASK 4: DEVELOPMENT OF SUPERPAVE GYRATORY COMPACTION PROCEDURES FOR GAP GRADED AND LARGE STONE MIXTURES

3.2.1 Test Plan

In development of the Superpave mix design method, very little work was done with mix types other than dense graded mixtures. Since the Superpave gyratory will be used for designing and controlling other mixture types, a specific compaction method is needed for each mix type.

Work with Stone Matrix Asphalt (SMA) mixtures, which are gap graded in nature, has shown that fewer gyrations than those required for dense graded mixtures, are needed to densify the mixture. For example, approximately 80 gyrations *(38)* have been shown to be sufficient for SMA for very high traffic volumes. After some desired density is obtained in the sample, additional gyrations tend to break down the aggregates resulting in a finer gradation and a denser mix. This increased density, for the most part, is the result of aggregate breakdown and not increased compactive effort.

Asphalt mixtures with gradations below the maximum density line and especially with relatively soft aggregates tend to break down excessively if over compacted. This breakdown is partially due to the stone on stone contact. It is essential then that excessive compaction of the mixture does not occur.

Ideally, a procedure should be developed that will allow the same compaction criteria for these more stony mixtures as for dense graded mixtures. One way to do that is to require the same number of gyrations for the different mixture types, but work has already shown that the number of gyrations should be different for dense graded mixtures than for these special mixture types. Another possible technique to be used is to look at the mixture's compaction rate. Once the rate of compaction drops below some selected value then the ultimate density has been obtained and compaction should be terminated. This technique is used for the LCPC gyratory, for the 150 mm Texas gyratory, and for the Corps of Engineers gyratory. Mixtures that compact quickly would require fewer gyrations and mixtures that compact more slowly would require more gyrations for compaction. In this study data was collected on the rate of compaction as well as number of gyrations necessary to provide proper compaction. If some specified rate of compaction can be selected as the cut off for compaction and if it appears to be satisfactory for all mixture types, then this approach is desirable over that of setting the number of gyrations for each mixture type. If the design number gyrations cannot be determined using the rate of compaction procedure, the number of gyrations for each mixture type will be recommended.

As stated earlier, work with SMA mixtures *(38)* has shown that approximately 80 gyratory revolutions are required for satisfactory compaction. The number of revolutions was established to provide a density approximately equal to the 50 blow Marshall which had been used as the standard for compaction of SMA samples. SMA mixtures designed with 50 blow Marshall and properly placed have provided good performance. So the feeling was that the 50-blow Marshall density was the correct density to use for mix design. Actually, the compactive effort should be selected to provide a density equal to that obtained in the field after several years of traffic.

Establishing the number of gyrations for the various mixture types will initially be done to provide a density equal to that presently being used for designing the particular mixture type if available. Attempts will be made to collect data that will compare the gyratory density and density of mixture in place after traffic. NCAT had been monitoring six dense graded projects that had been down for about 2-3 years. Aggregates and asphalt cements were saved from construction to be blended and compacted with the gyratory machine. The in-place density data showed the relationship between laboratory density for various gyrations and in-place density. *(17)* The plan for this task was to prepare the various mixture types and to compact them using the present Superpave compaction criteria for dense graded mixtures. The proposed list of aggregates used for developing compaction criteria for gap graded and large stone mixtures is shown in Table 3.1. The mixtures will be evaluated for volumetric properties at N_{design} and the $\%G_{mm}$

Table 3.1 Test Plan for Gap Graded and Large Stone Mixture Evaluation Using Dense Graded Superpave Compaction Criteria

	Superpave Gyratory Compaction Criteria: $N_{initial}$ (8), N_{design} (128), and $N_{maximum}$ (208)	
Mineral Aggregate	Gap Graded	Large Stone
Alabama Limestone	X	X
Georgia Granite	X	X
Nevada Gravel	X	N/A[1]
New York Gravel	X	N/A

Notes (1) Stone size for these aggregates does not permit large stone mixture evaluation.

at $N_{initial}$ and $N_{maximum}$. The gradations used for the gap graded and the large stone mixture evaluations are shown in Figure 3.1.

Evaluating $N_{initial}$, N_{design}, and N_{max} using the criteria for dense graded mixtures was a starting point for the gap graded and large stone mixtures being evaluated in this task. Six gap graded projects and four large stone projects that had been in service for at least 2-3 years were identified and samples were compacted in the laboratory using the Superpave gyratory compactor. The laboratory compacted density was then compared to the in-place density to establish the number of gyrations necessary for the different mix types to produce an equivalent density in the field for a particular amount of traffic or ESALs. Three methods were utilized in the task to determine the density of field mixtures in the Superpave gyratory compactor. The first method used for one of the gap graded projects was to obtain plant produced material (loose mix) which had been collected at

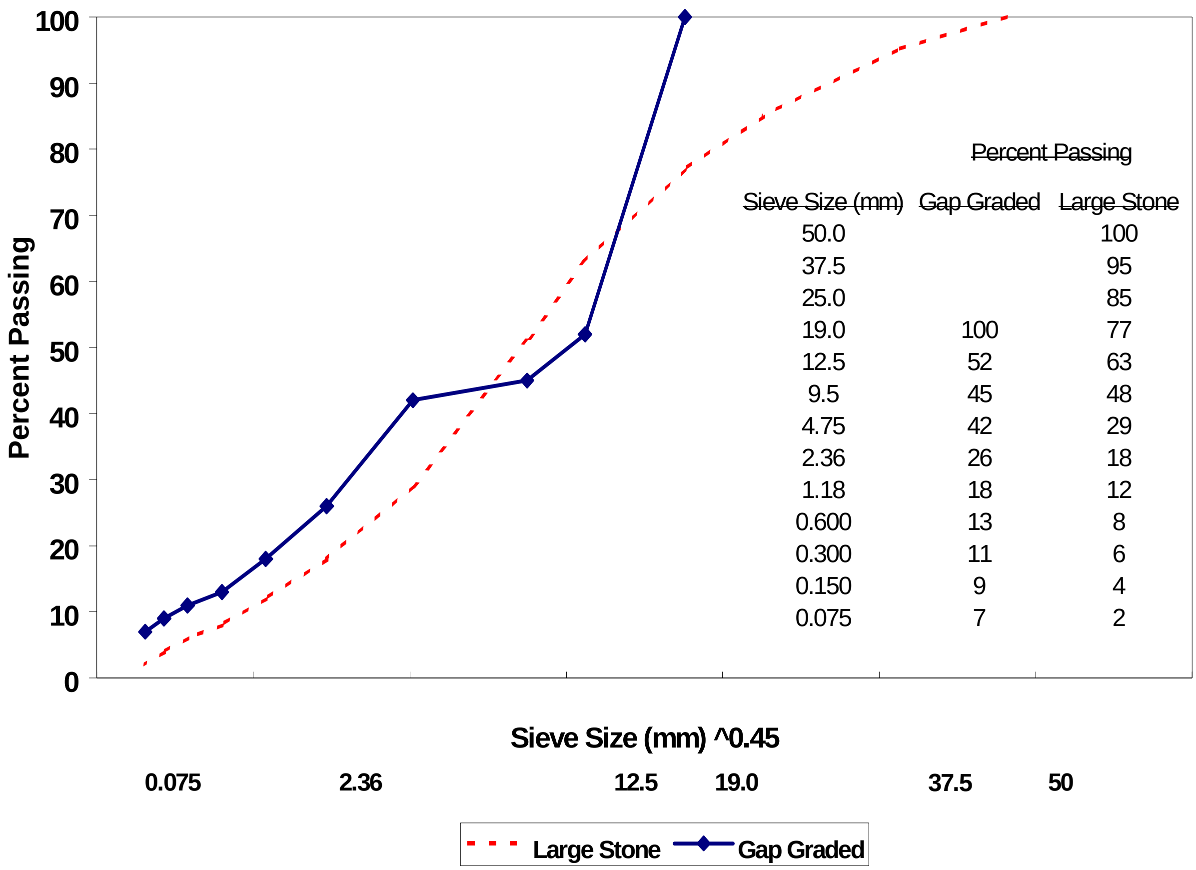

Figure 3.1 Task 4 Gap Graded and Large Stone Gradations

the time of construction. The obtained loose mix was then sampled and evaluated in the gyratory compactor. This method is the ideal method, but also the most difficult to obtain, since plant produced material (loose mix) is seldom available after two or three years from construction. The second method, used by the remaining five gap graded mixtures and three of the large stone mixtures, was to obtain samples of the mineral aggregates, asphalt binder, and other components, if present, which were used in the mixture. Mixtures were then combined according to the job mix formula and evaluated in the gyratory compactor. The third method used for one of the large stone mixtures was to obtain field cores and extract the asphalt binder from the aggregate. Next, the recovered aggregate was mixed with an asphalt binder having approximately the same grade or viscosity as the original asphalt binder, and the samples evaluated in the gyratory compactor.

Using one of the three methods discussed above to collect data from several projects provided information necessary to set N_{design} for the gap graded and large stone mixtures evaluated in this project. Based on these results, N_{design} requirements have been recommended for each mixture type.

It was believed that the gyratory compactor could be used to satisfactorily compact the special mixtures evaluated in this task. The number of gyrations might have to be adjusted for the various mixtures to produce the desired density. It was the goal of this study, however, to develop a unique method that could be used to compact all mixture types.

The research team would have liked to provide a method for special mixes that was consistent with the method for dense graded mixes. This means that the number of gyrations for various traffic levels and various climates would be the same as that for dense graded mixes.

Gap graded mixtures from Georgia (2), Maryland (2), Nebraska (1), and Wyoming (1) were selected for evaluation. Also, large stone mixtures from Missouri (1), Wyoming (1) and New Mexico (2) were evaluated. The projects evaluated consisted of a wide range of pavement age and loading (ESALs), which were beneficial in determining the appropriate compaction levels for the different mixture types. Specific information on the various field obtained mixtures can be found in Chapter 4.

3.3 TASK 5: PREPARE AN INTERIM REPORT

Approximately six months after the initiation of the project an interim report was provided to NCHRP documenting the results of the project work for Tasks 1-4.

3.4 TASK 6A: EVALUATION OF THE EFFECT OF VARYING SHORT TERM AGING TEMPERATURE ON MIXTURE VOLUMETRIC PROPERTIES

3.4.1 Test Plan

The principal research product of the Strategic Highway Research Program (SHRP) in the area of asphalt materials and construction was Superpave, a new asphalt mix design and analysis system incorporating criteria for materials selection, and volumetric proportioning of asphalt binders and aggregates to produce superior performing asphalt pavements (Superpave). As part of the Superpave mix design system, a different compaction procedure, Superpave gyratory compaction, was adopted to replace conventional compaction methods such as the Marshall method *(39)* and the Hveem method *(40)*. As described in the SHRP reports, the loose asphalt mixture was to be subjected to short term oven aging at 135˚C for four hours to simulate the aging of the in-place mixture during construction and in the first few years of the pavement's life *(8)*. This short term aging procedure was standardized as AASHTO PP2, *Short and Long-Term Aging of Hot Mix Asphalt (HMA). (41)*

One concern of asphalt technologists is the need for aging all loose asphalt mixture samples at a single temperature (135˚C) for a fixed period of time (4 hours) prior to compaction. Mixing and compaction temperatures of asphalt mixtures during production vary based on a number of factors with the stiffness of the asphalt binder largely responsible. Typically stiffer asphalt binders, such as PG 70-22 and PG 76-22 binders, are mixed in the hot-mix plant and compacted at higher temperatures than asphalt binders that are less stiff, such as PG 58-28 binders. Depending on the climate and traffic conditions, it is very possible that a single hot-mix asphalt plant could use each of these asphalt binder grades in different projects. For example, a mixture for use on a medium traffic volume roadway might require a PG 58-28 asphalt binder while another project mixture might be placed on an urban Interstate highway with sufficient traffic loading to require a stiffer high temperature grade such as a PG 70-28 asphalt binder. Although the two mixtures would likely be mixed, stored, and compacted at different temperatures during production, the laboratory aging procedure described in AASHTO PP2 would be unchanged.

A further argument against the concept of a single aging temperature is related to the field quality control procedures of asphalt mixtures. Superpave did not address the issue of aging plant-produced asphalt mixtures, but SHRP A-407 *(42)* infers that the laboratory aging procedure described in AASHTO PP2 (4 hours, 135˚C) is used in the mix design process to "simulate field aging." Therefore, one can infer that it should not be necessary for plant-produced asphalt mixtures to be subjected to short-term oven aging prior to compaction using the Superpave gyratory compactor (SGC). This assumption was made by the NCHRP 9-7 research team for the duration of the research project. Consequently, aging of plant-produced asphalt mixtures was not addressed by the NCHRP 9-7 researchers (*43*).

The Superpave hypothesis can be stated as follows:

> *In the laboratory mix design process, all asphalt mixture specimens are subjected to a short-term oven aging procedure (4 hours, 135 ˚C) to simulate the aging that occurs when asphalt mixtures are produced in a hot-mix plant and compacted on the roadway. Therefore, plant-produced asphalt mixtures are not required to be subjected to short-term oven aging.*

The hypothesis is based on the assumption that the laboratory short-term oven aging procedure best simulates actual mixture aging for a variety of asphalt binders, and hot-mix asphalt plant types. If an asphalt mixture containing a PG 70-28 asphalt binder is mixed at 160˚C and stored in the surge silo for two hours, the asphalt binder absorption may be significantly different from the absorption that can occur during laboratory aging at 135˚C for four hours. This differential absorption can lead to changes in mixture volumetric and densification properties from the design to production, even though the as-produced mixture's component aggregate, gradation, asphalt binder, and asphalt content are exactly the same as the design components. By aging at a temperature that more closely simulates the asphalt mixture temperature during production, a better correlation can be achieved between laboratory and production mixture properties. Differences in mixture properties can more confidently be assigned to changes in material components rather than improper simulation of field conditions.

A final, pragmatic reason favoring the use of the asphalt mixture's compaction temperature as the aging temperature is that by aging at the standard temperature of 135˚C, mixture specimens may have to be transferred to an oven operating at a higher temperature for a short period of time to increase the asphalt mixture temperature to the appropriate compaction temperature range. This process will require an additional oven operating at a temperature greater than 135˚C. In a field laboratory, the addition of an oven will result in an increase in cost, an increase in specimen handling time, and a reduction in laboratory space.

Based on the preceding discussion, it would seem logical that the short-term oven aging temperature could be changed for volumetric mix design specimens from 135˚C to the laboratory mixture compaction temperature based on the temperature-viscosity relationship of the asphalt binder. This change has the following advantages:

- The short-term aging temperature would better represent the temperature at which an asphalt mixture is maintained during production.
- The aging temperature would better simulate the temperature at which asphalt binder absorption occurs during production, thereby allowing a better correlation between laboratory and field mixture properties (such as air voids, VMA, etc.).
- An additional oven would not be needed, thereby reducing sample handling time, while not reducing available lab space.

An experiment was required to verify that the recommended change in short-term oven aging can be made without significantly affecting mixture volumetric and densification properties. The test plan for this task is shown in Table 3.2 and Figure 3.2. As can be seen, the test plan consists of four levels of mineral aggregate type, two levels of aggregate gradation, four levels of asphalt binder type, and two levels of aging temperature. The test plan or experimental design is described in more detail in the following paragraphs.

Aggregate

Four aggregates with different physical properties were evaluated. The four aggregates include a Georgia granite, an Alabama limestone (low absorption), an Ohio limestone and a New York gravel (high absorption). Using these aggregates allowed a determination of the effect of different aggregate absorption values on the aging characteristics of the mixture.

Gradation

Two combined aggregate gradations were evaluated: a coarse graded 12.5 mm gradation, and a fine graded 12.5 mm gradation. The coarse gradation was developed below the restricted zone near the coarse control points for a 12.5 mm nominal maximum size mix. The fine gradation was developed near the restricted zone for a 12.5 mm nominal maximum size mix. One of the fine graded mixes passed through the restricted zone since this was the best gradation that could be obtained for the aggregate stockpiles. Due to variations in individual stockpile aggregates, the actual gradations vary slightly for each of the four aggregate types. Figures 3.3 - 3.6 illustrate the blend gradations for the four aggregates.

Asphalt Binder

Four binders were selected to represent a range of typically used asphalt binders: PG 52-28, PG 64-22, PG 76-22, and the SHRP Materials Reference Laboratory (MRL) asphalt binder identified as AAG-1 (PG 58-10). The PG 52-28 was selected to provide an asphalt binder with a compaction temperature less than 135˚C. The PG 64-22 was chosen to provide an asphalt binder with a compaction temperature greater than 135 ˚C.

The PG 76-22 was selected for two reasons. The first reason was to provide an asphalt binder with a higher compaction temperature than a PG 64-22 asphalt binder. The second reason was to evaluate a modified asphalt binder in the testing program. It was hypothesized that the polymer modification could affect the aging and asphalt absorption process.

The AAG-1 asphalt binder from the SHRP MRL was selected because it was one of the two asphalt binders most often used in mixture studies during SHRP (AAK-1 was the other asphalt binder often used). The AAG-1 asphalt binder is a California Valley asphalt which is very temperature-susceptible. It was hypothesized that a temperature-susceptible asphalt binder might indicate a different response to changes in aging temperature than the other asphalt binders selected for this study.

Aging Temperature

Two aging temperatures were selected: 135˚C (standard practice) and the median temperature from the mixture compaction temperature range. The selected aging temperatures were 128˚C, 131˚C, 145˚C, and 155˚C for the PG 52-28, AAG-1 (58-10), PG 64-22, and PG 76-22 asphalt binders.

Asphalt Binder Content

The design asphalt binder content was used as the asphalt content for all combinations of aggregate and gradation in the task. The design asphalt content was selected to provide 4 percent air voids using a N_{design} of 128 gyrations. The PG 64-22 asphalt binder was used in determining the design asphalt content.

Replicates

Three SGC specimens were compacted for each experimental cell. Specimens were mixed, aged, and compacted at temperatures specified in the experimental matrix. Compaction was stopped at N_{design} (128 gyrations). After the SGC specimens had cooled, the bulk specific gravity, G_{mb}, of the specimens was determined using AASHTO T166 (*44*). Two specimens were also prepared to determine the mixture's maximum theoretical specific gravity for each experimental cell. Following mixing and aging, the specimens were allowed to cool before determining the maximum theoretical specific gravity, G_{mm}, in accordance with AASHTO T209 *(45)*. The mixture G_{mb}, G_{mm}, and height data from the SGC were then used to determine the compaction characteristics of the specimen set.

Two other variables were considered as part of the experimental design: asphalt binder content and aging time. Asphalt binder content could be explored not only at the design asphalt content, but also at 0.5 percent less and 0.5 percent more than the design asphalt content. These asphalt binder contents are appropriate for field production. It is possible that different asphalt contents will result in different asphalt film thicknesses, thereby affecting asphalt absorption and aging. However, additional asphalt binder contents would increase the total number of cells from 64 to 192 while increasing the total number of specimens from 192 SGC specimens and 128 G_{mm} specimens to 576 SGC specimens and 384 G_{mm} specimens. Consequently, the asphalt content was established as the design asphalt binder content only.

Aging time could also be included as a controlled variable. A study performed by members of the FHWA Mixtures ETG *(46)* indicated no significant difference in mixture properties of

Table 3.2 Test Plan for Evaluation of the Effect of Varying Aging Temperature

		PG 52-28		SHRP MRL AAG-1 (58-10)		PG 64-22		PG 76-22 (Polymer Modified)	
Aggregate	Gradation[1]	135°C	Mixture[3] Compaction Temperature	135°C	Mixture Compaction Temperature	135°C	Mixture Compaction Temperature	135°C	Mixture Compaction Temperature
Alabama Limestone	Fine	X[2]	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X
Georgia Granite	Fine	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X
New York Gravel	Fine	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X
Ohio Limestone	Fine	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X

Notes:
(1) Fine and coarse indicate gradations above and below the restricted zone, respectively.
(2) Each cell represents three replicate compacted specimens and two TMDs.
(3) The mixture compaction temperature as determined from the temperature/viscosity plot or the manufacturer's recommendation, as is the case with certain polymer modified asphalt binders.

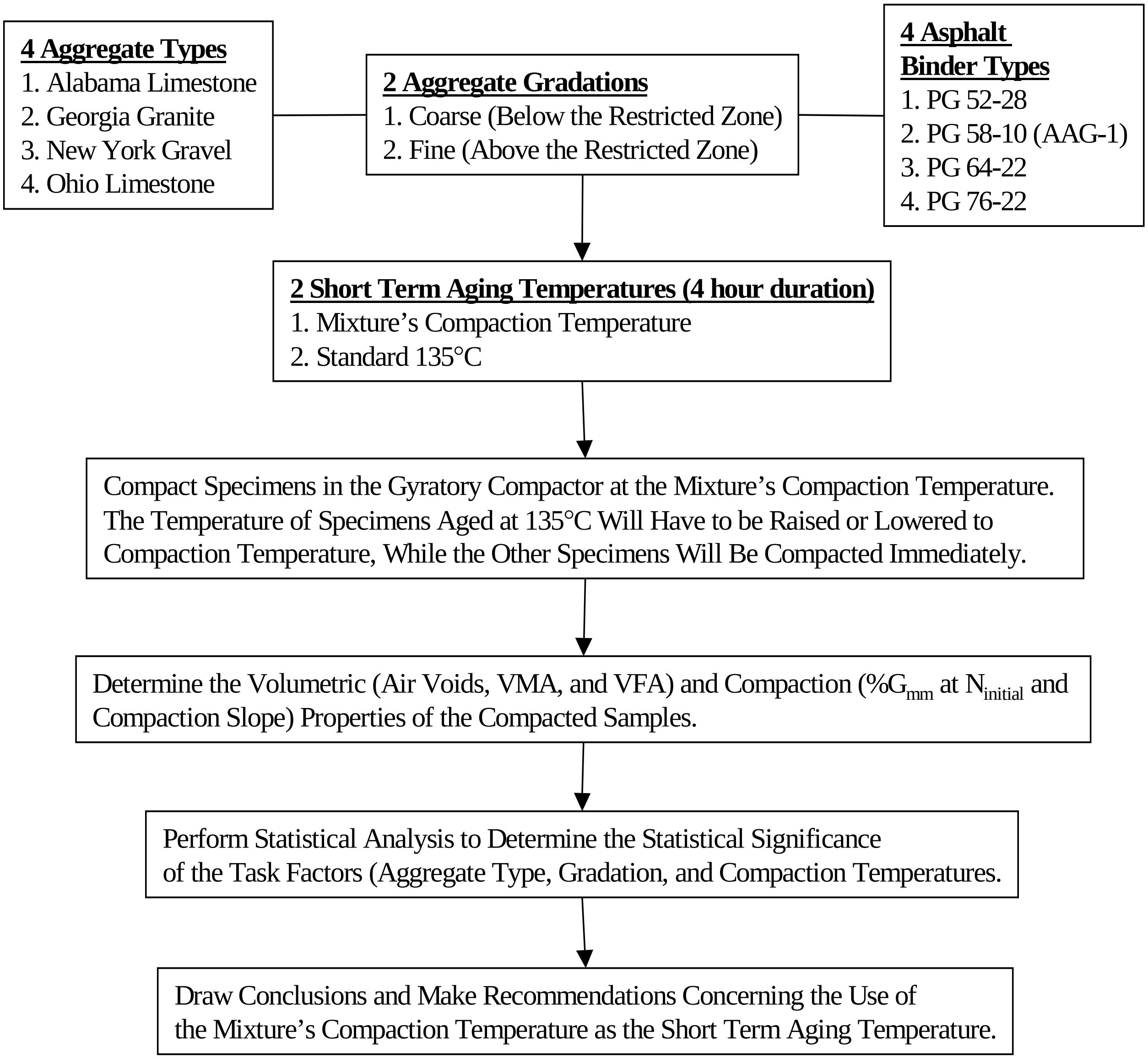

Figure 3.2 Test Plan for Task 6A

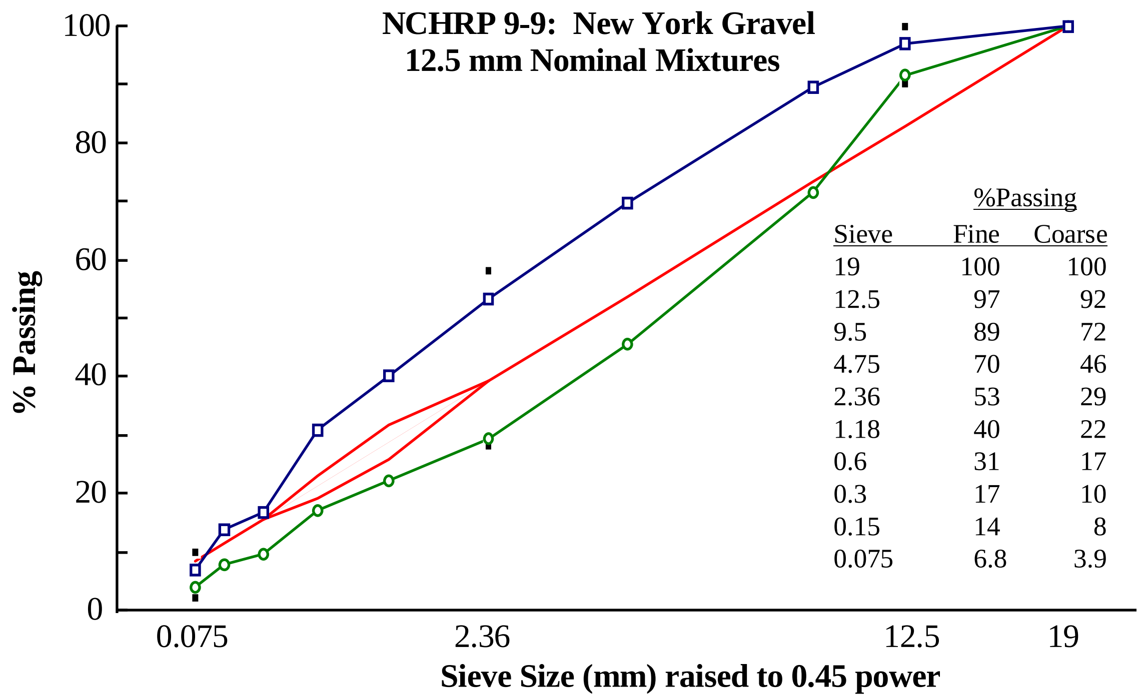

Figure 3.3 Task 6A Aggregate Gradations for 12.5 mm Nominal Maximum New York Gravel Mixtures

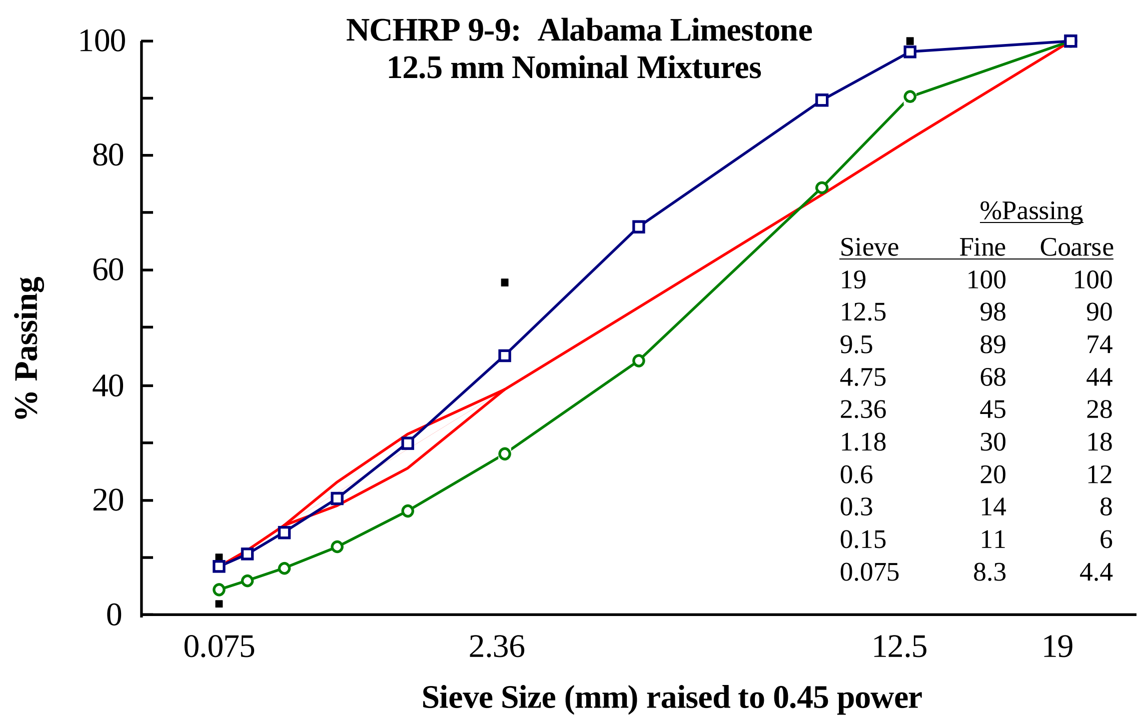

Figure 3.4 Task 6A Aggregate Gradations for 12.5 mm Nominal Maximum Alabama Limestone Mixtures

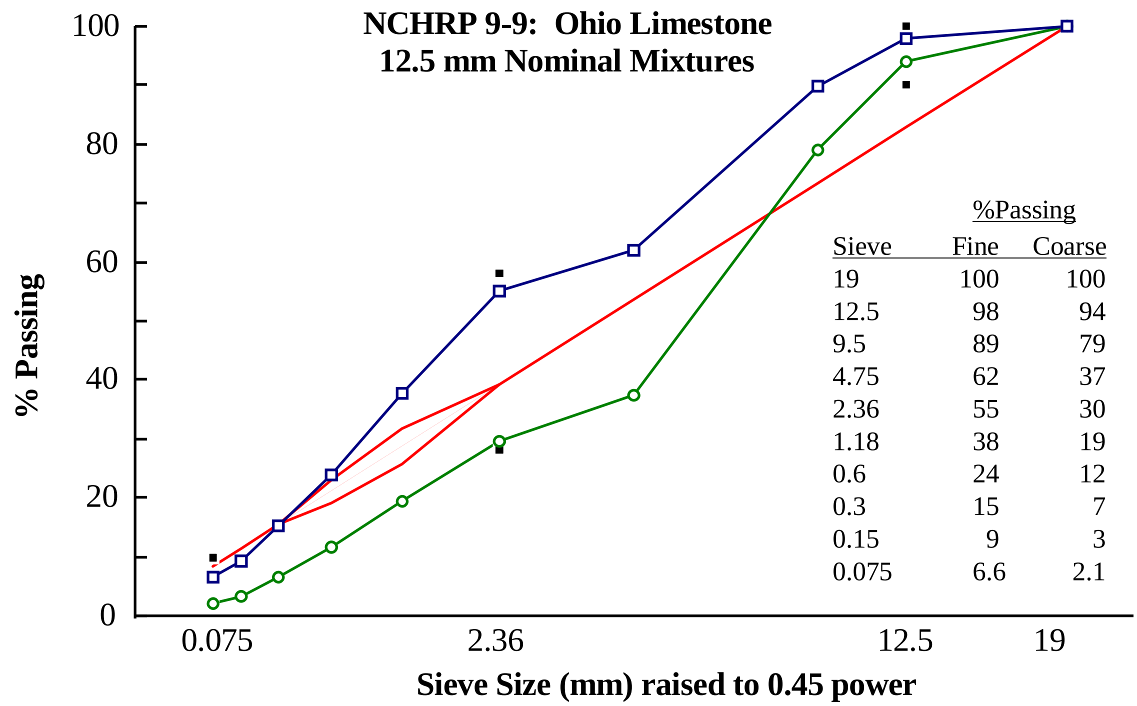

Figure 3.5 Task 6A Aggregate Gradations for 12.5 mm Nominal Maximum Ohio Limestone Mixtures

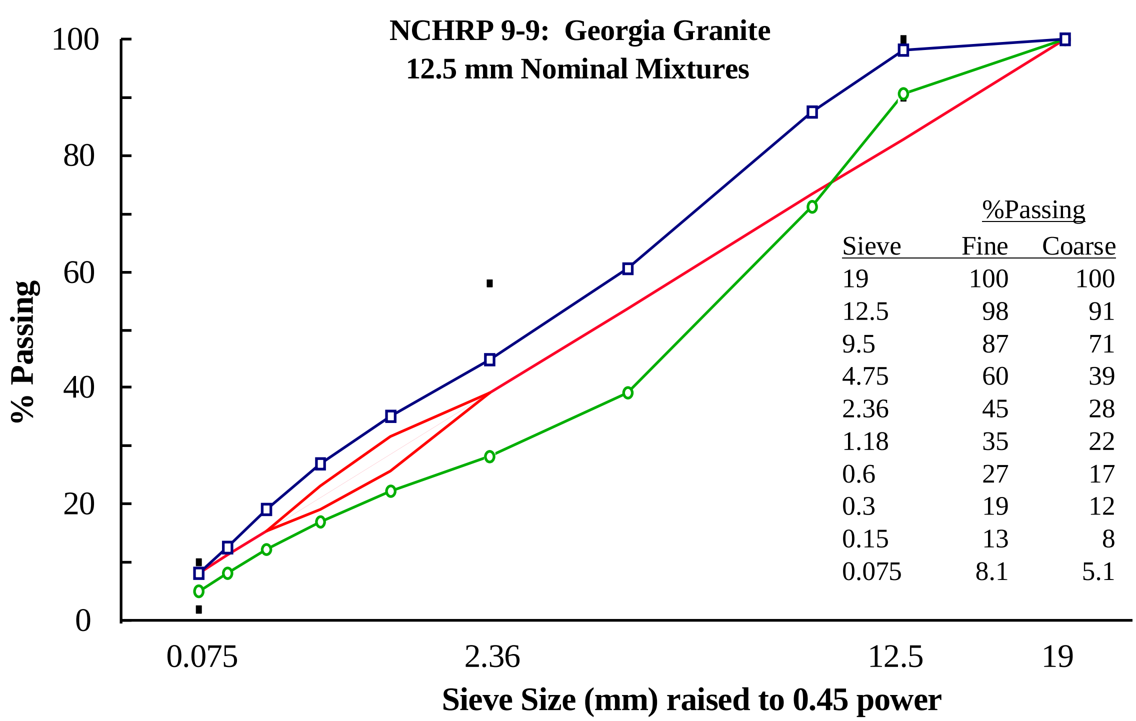

Figure 3.6 Task 6A Aggregate Gradations for 12.5 mm Nominal Maximum Georgia Granite Mixtures

specimens that were aged for 2 hours and 4 hours. However, if a different aging temperature is selected than 135˚C, it is possible that aging time could have an effect on mixture properties. If this variable were selected, two levels (2 and 4 hours) could have been evaluated. Much like the asphalt binder content variable, adding an additional aging time would substantially increase the total experiment. Rather than adding two levels of aging time to the experiment, an additional partial experiment was developed as a subset of the original experiment described in Table 3.2. This partial experiment included: one aggregate (GA Granite); two gradations (Fine and Coarse); two asphalt binders (PG 64-22 and PG 76-22); two aging temperatures (135˚C and the Compaction Temperature); two aging times (4 hours and 2 hours); and one asphalt binder content (Design). The resulting experimental matrix is indicated in Table 3.3.

The response variables for the experiments shown in Tables 3.3 and 3.4 are as follows:

- Percentage of air voids at N_{design}
- Percentage of voids in the mineral aggregate (VMA) at N_{design}
- Maximum theoretical specific gravity (G_{mm})
- Compaction at $N_{initial}$ (%G_{mm} at $N_{initial}$)
- Compaction slope (calculated from $N_{initial}$ to N_{design})

Table 3.3 Experiment to Evaluate the Effect of Short Term Aging Time and Temperature on Mixture Volumetric and Densification Properties

Asphalt Binder		PG 64-22				PG 76-22			
Aging Time, hrs.		2		4		2		4	
Aging Temp. ˚C		135	145	135	145	135	155	135	155
GA Granite	Fine	x	x	x	x	x	x	x	x
	Coarse	x	x	x	x	x	x	x	x

The hypotheses of the experiment described in Table 3.2 are as follows:

Null Hypothesis (H_o):
In the laboratory mix design process, performing short-term oven aging of asphalt mixtures at the mixture compaction temperature rather than 135 ℃ will not significantly affect the asphalt mixture volumetric and densification properties.

Alternate Hypothesis (H_a):
In the laboratory mix design process, performing short-term oven aging of asphalt mixtures at the mixture compaction temperature rather than 135 ℃ will significantly affect the asphalt mixture volumetric and densification properties.

The hypotheses of the experiment described in Table 3.3 are as follows:

Null Hypothesis (H_o):
In the laboratory mix design process, performing short-term oven aging of asphalt mixtures for two

hours at the mixture compaction temperature rather than four hours at 135 ℃ will not significantly affect the asphalt mixture volumetric and densification properties.

Alternate Hypothesis (H_a):
In the laboratory mix design process, performing short-term oven aging of asphalt mixtures for two hours at the mixture compaction temperature rather than four hours at 135 ℃ will significantly affect the asphalt mixture volumetric and densification properties.

3.5 TASK 6B: EVALUATION OF THE DEPTH OF MIXTURE ON THE REQUIRED NUMBER OF GYRATIONS

3.5.1 Test Plan

Superpave mix design is based upon volumetric properties of mixtures compacted in the Superpave Gyratory Compactor. The amount of compactive effort or number of gyrations is a function of the expected number of ESALs and expected high ambient temperatures. There is some thought that the number of gyrations should also be a function of depth below the surface of the pavement. For example, a mixture 100 mm below the pavement surface will probably not be compacted, under traffic, to the same density as an identical mixture at the surface. The depth of a mixture greatly affects the degree and rate of densification. Cooler temperatures and reduced vertical stresses at the lower levels will result in less densification and at a slower rate. The original experimental design for the SHRP study included depth. However, time restrictions required that depth be removed from the experiment. The results of the SHRP experiment, which include only mixtures in the top 100 mm of the pavement structure, formed the basis of the N_{design} compaction matrix, as given in AASHTO PP28. Therefore, using the current N_{design} compaction matrix results in extra compaction for mixtures below 100 mm in the pavement structure. Practically, the main effect is that a mixture low in asphalt content is designed, possibly leading to earlier durability problems.

Data is needed to establish the proper number of gyrations for the SGC as a function of depth below the surface. When considering this approach, one must keep in mind that many times the underlying layers are exposed to traffic for a significant amount of time prior to being overlaid. This exposure to high traffic levels, prior to the upper level mixtures being placed, could result in over compaction and lead to rutting.

Development of laboratory compaction criteria involves several steps as discussed below. The first step was to determine the expected vertical stress at various points below a pavement surface. At the pavement surface this was equal to the expected truck tire pressure of 827 kPa. The estimated stress at points below the surface was determined by using an existing acceptable model (such as DAMA, JULEA, etc.) to calculate the vertical compressive stress at various points below the pavement surface. A typical loading condition (i.e., a fully loaded 18 wheeler) was applied to the surface and the vertical stresses determined at various points below the pavement surface. After the vertical stress at the various points had been determined, mixtures were prepared and compacted in a Superpave gyratory compactor (SGC), in which the normal (vertical) pressure was varied. Each mixture evaluated was compacted with the SGC at a pressure equal to the expected truck tire pressure (827 kPa). This was done for a range of gyrations as shown in Figure

3.7. The density at N_{design} with the SGC was used to determine the number of gyrations with the SGC set at a vertical pressure of 600 kPa to provide equal density. Then the density at P1, P2, P3, and P4 for the same number of gyrations was plotted. This data was plotted as shown in Figure 3.8 so that the number of gyrations for the SGC for various pressures could be determined. The number of gyrations for the SGC for various depths below the surface was established based on Figure 3.8.

The test plan, shown in Table 3.4, included three aggregates, four pressures, and four gyration levels with the SGC. Mixture designs were performed for the three mixtures with the SGC at an intermediate design (N_{design}) level of 128 gyrations.

This study determined the effect of pressure on the densification of HMA. This was then correlated to the number of gyrations with the SGC. For example, at the surface of the pavement 96 gyrations may be required for proper densification, but at 100 mm below the surface less than 96 gyrations would be required. If this difference in the number of required gyrations was significant then the final result would be a table that shows the required number of gyrations for various depths below the pavement surface. Appropriate $N_{initial}$ and $N_{maximum}$ requirements would also be specified. There were not any changes made in the volumetric criteria. It was anticipated that there would be a set of criteria for the pavement surface and for 100 mm below the surface. Depending on the results, criteria for an increased number of depths would possibly be necessary.

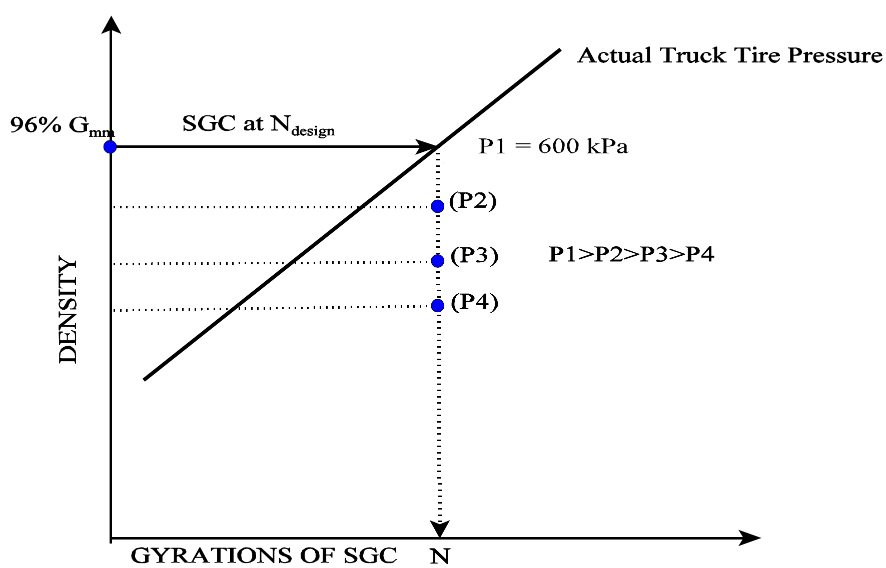

Figure 3.7 Effect of Normal Pressure on Density

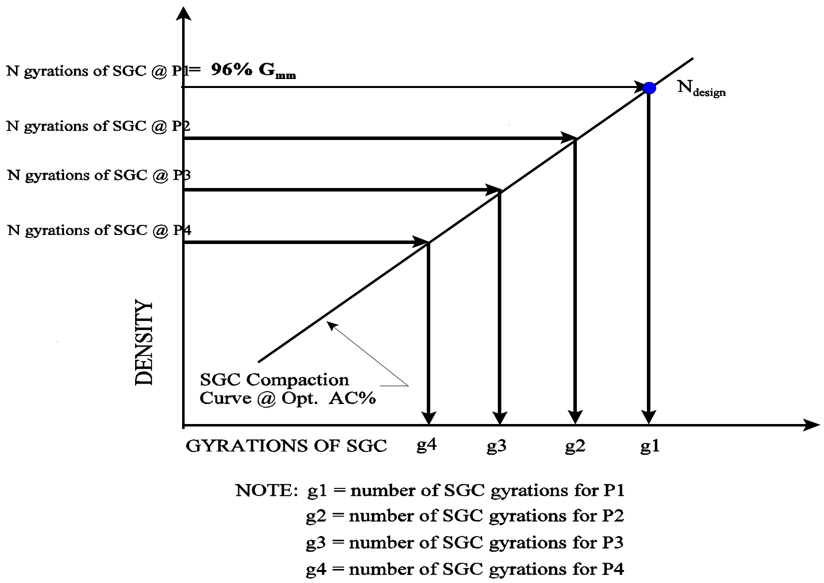

Figure 3.8 Correlation of Normal Pressure to Number of Gyrations

Table 3.4 Test Plan for Evaluating the Effect of Mixture Depth on the Required Number of SGC Gyrations

Compactor	Compaction Level	New York Gravel	Georgia Granite	Alabama Limestone
SGC (P1)	g1	X[1]	X	X
	g2	X	X	X
	g3	X	X	X
	g4	X	X	X
SGC (P2)	N	X	X	X
SGC (P3)	N	X	X	X
SGC (P4)	N	X	X	X
SGC	Intermediate Level	X	X	X

Notes:
(1) Each cell represents three replicate specimens.

3.6 TASK 6C: CONSOLIDATION OF THE N_{design} COMPACTION MATRIX AND EVALUATION OF THE $N_{maximum}$ REQUIREMENT

3.6.1 Test Plan

Currently, the Superpave mixture design specification, AASHTO PP28, provides a 7 * 4 N_{design} matrix for the gyratory compaction of HMA specimens. The matrix consists of seven levels of traffic ranging from less than 0.3 million ESALs to greater than 100 million ESALs. Four levels of temperature are considered ranging from less than 39°C to 43-44°C.

Many individuals and organizations have stated that the number of compaction levels should be reduced from the twenty-eight, to three to five levels. Some have also expressed the desire to possibly have low, medium, and high traffic compaction levels. In order to reduce or consolidate the compaction matrix it has to be determined that there is no significant difference between the volumetric properties of laboratory prepared mixtures at given compaction levels or levels of N_{design}. The test plan shown in Table 3.5 and Figure 3.9 was used to determine if a reduction or consolidation of the N_{design} matrix is justified.

In the experimental plan a total of four factors were varied and their effect determined. These factors included the levels of N_{design}, the type of mineral aggregate, the aggregate gradation, and the asphalt content. The levels of N_{design} evaluated consisted of the lowest (68) and highest (172) levels of N_{design} currently specified by PP28. Three levels of N_{design} (93, 113, and 139) were evaluated between these lowest and highest levels. In addition another level of gyration (40) was analyzed for low volume roads. It was anticipated that compacting mixtures at these values of N_{design} would allow interpolation of the specimen volumetric properties at intermediate N_{design} levels. The mineral aggregates consisted of a New York gravel, a Georgia granite, an Alabama limestone, and a Nevada gravel. The aggregate gradations used are shown in Table 3.6 and Figure 3.10, along with the Superpave control points and restricted zone information for a 12.5 mm nominal maximum size mixture. Mixtures were prepared for each combination of N_{design}, mineral aggregate type, and aggregate gradation at three asphalt contents. These three asphalt contents were chosen at 0.5 percent increments in order to bracket the optimum asphalt content for the mixture. The asphalt cement used for all mixtures was a PG 64-22 (unmodified). Samples were mixed at the specified mixing temperature and aged at 135°C for four hours. The temperature of the samples was then raised to compaction temperature and compacted to the specified N_{design} level in the Pine gyratory compactor. After compaction the response variables of air voids, voids in mineral aggregate, $\%G_{mm}$ at $N_{initial}$, and the compaction slope from $N_{initial}$ to N_{design} were determined for statistical analysis.

The $N_{maximum}$ specification limit of less than 98 % G_{mm} was also evaluated as shown in Table 3.7. This evaluation consisted of compacting duplicate specimens to $N_{maximum}$ levels of 104 (N_{design}= 68), 181 (N_{design} =113), and 288 (N_{design} =172). These specimens were compacted at the optimum asphalt content, which was determined in the first part of the task. The data was analyzed to determine whether the $N_{maximum}$ requirement is necessary for Superpave mixture design or analysis.

Table 3.5 Test Plan for the Evaluation of the N_{design} Compaction Matrix

		N_{design} = 40			N_{design} = 68			N_{design} = 93			N_{design} = 113			N_{design} = 139			N_{design} = 172		
Aggregate	Gradation[1]	Low AC (%)	Med AC (%)	High AC (%)	Low AC (%)	Med AC (%)	High AC (%)	Low AC (%)	Med AC (%)	High AC (%)	Low AC (%)	Med AC (%)	High AC (%)	Low AC (%)	Med AC (%)	High AC (%)	Low AC (%)	Med AC (%)	High AC (%)
New York Gravel	Fine	X[2]	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Georgia Granite	Fine	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Alabama Limestone	Fine	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Nevada Gravel	Fine	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Coarse	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X

Notes:

(1) Fine and coarse indicate gradations above and below the restricted zone, respectively.

(2) Each cell represents three replicate compacted specimens. Duplicate TMDs were run for the low, medium, and high AC% for each mixture type.

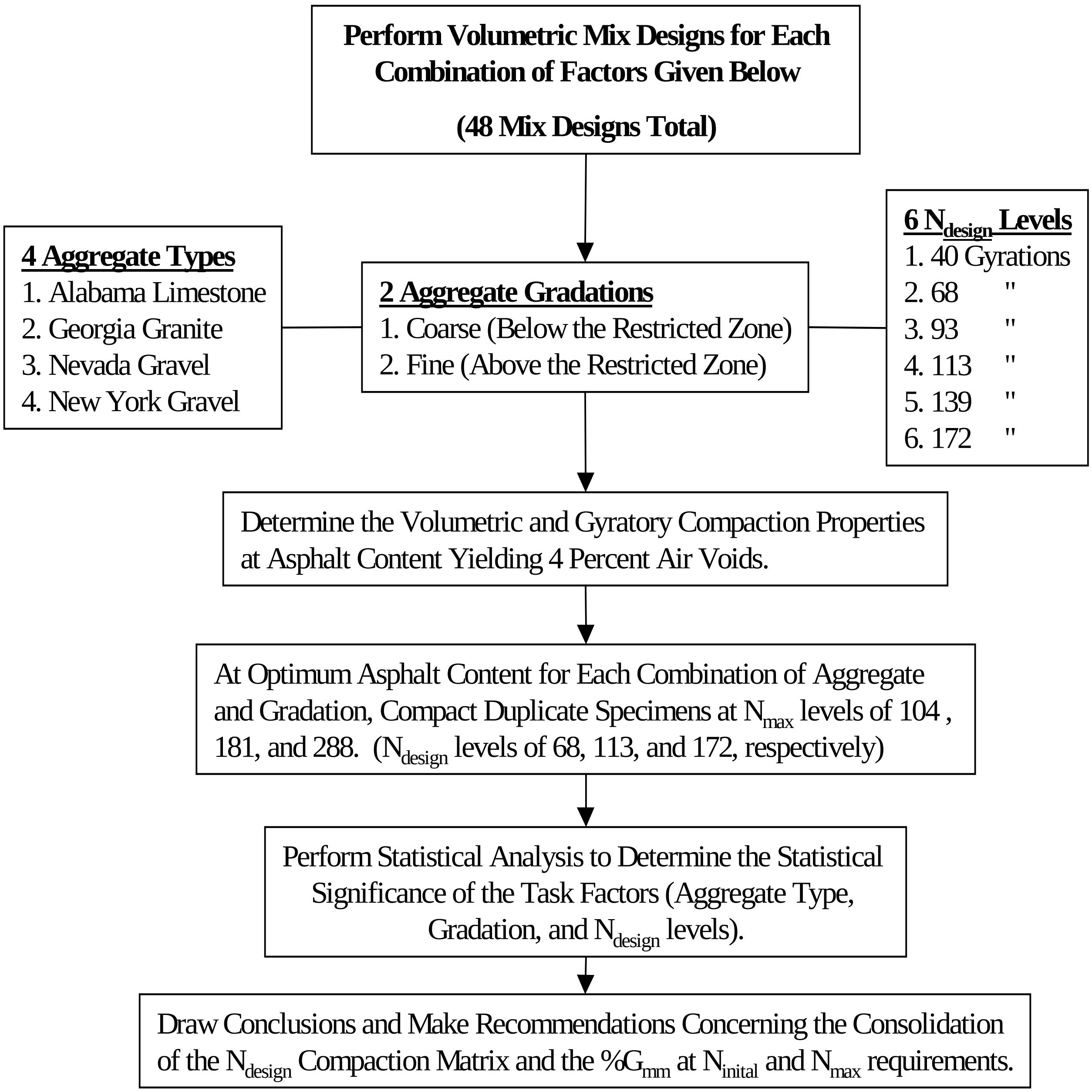

Figure 3.9 Test Plan for Task 6C

Table 3.6 Aggregate Gradations for Task 6C

	Percent Passing (%)					
	Gradations		Control Points		Restricted Zone	
Sieve Size (mm)	Fine	Coarse	Upper	Lower	Upper	Lower
19.0	100	100	100			
12.5	96	95	100	90		
9.5	85	83	90			
4.75	63	47				
2.36	48	32	58	28	39.1	39.1
1.18	37	21			31.6	25.6
0.6	28	15			23.1	19.1
0.3	19	11			15.5	15.5
0.150	10	8				
0.075	5	5	10	2		

Past research data has shown that the mixtures most likely to fail this requirement are the mixtures with aggregate gradations below the restricted zone. It is generally believed that these are the most rut resistant mixtures.

The requirements for $N_{initial}$ for low volume pavements were also evaluated from the data. Superpave contains the same criteria at $N_{initial}$ for all traffic levels. At high traffic levels the criterion is appropriate, but at low levels, where permanent deformation is of less concern, weaker aggregate skeletons would be appropriate and would result in more economical mixtures. Under the existing criteria, fine gradations could meet both the gradation and fine aggregate angularity requirements but would be eliminated by the requirement at $N_{initial}$. It appears that the specification requirement at $N_{initial}$ should be investigated.

The slope of the densification curve can be calculated between $N_{initial}$ and N_{design}. The density criteria at N_{design} and $N_{initial}$, when evaluated together, limit the minimum slope that is acceptable. It has been assumed that mixtures with steeper densification curves have stronger aggregate skeletons. Although the densification slope is not a Superpave criteria, it is interesting to consider the minimum slope which is allowed within each traffic category. Table 3.8 lists the initial and design gyrations and the calculated minimum slope for all traffic levels. Curiously, the slope is required to increase as the amount of traffic decreases. In other words, lower volume mixtures are required to have stronger aggregate skeletons than for higher volume mixtures. If the minimum densification curve slope for the highest traffic category is applied to other traffic levels, the allowable density at $N_{initial}$

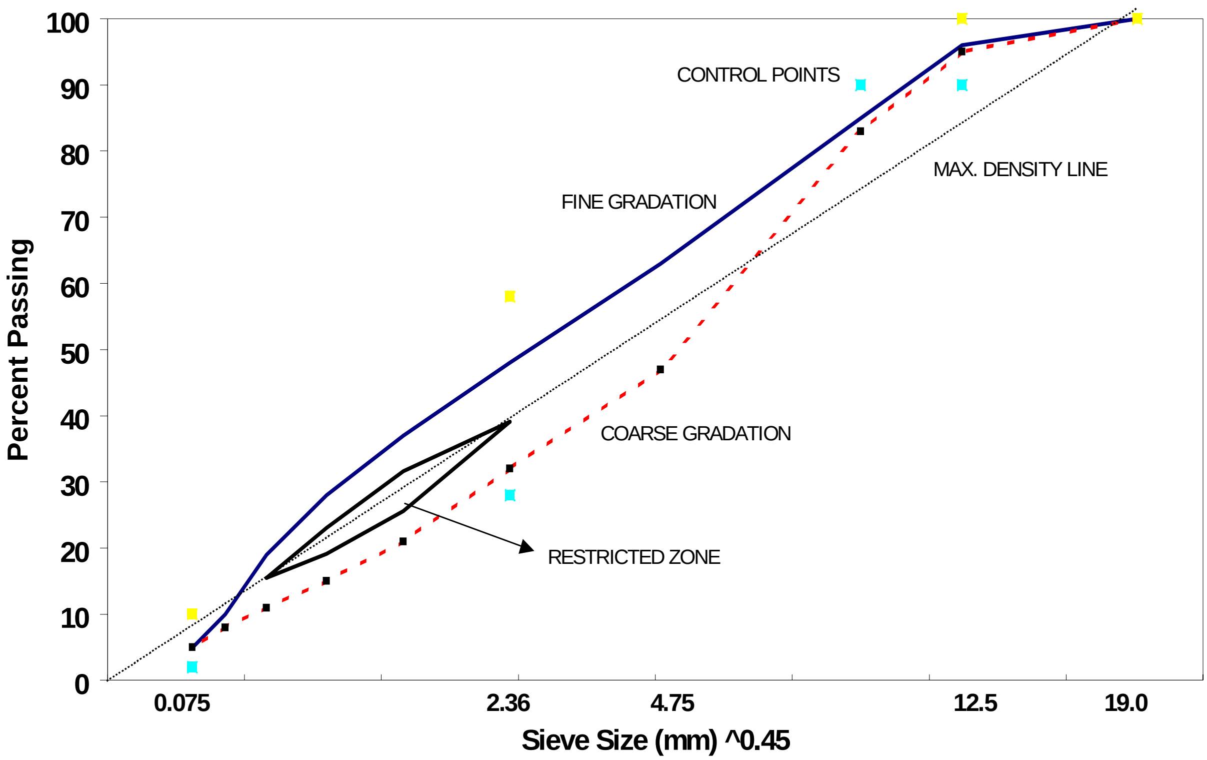

Figure 3.10 Task 6C 12.5 mm Nominal Maximum Size Gradations

is allowed to increase. For the lowest traffic level the maximum density allowed would increase from 89% to 90.2% of $G_{mm.}$ Based on these slopes, recommendations were made concerning the need for changes in the $N_{initial}$ requirements.

Table 3.7 Test Plan for the Evaluation of the $N_{maximum}$ Specification Requirement

		N_{max} = 104 N_{design} = 68	N_{max} = 181 N_{design} = 113	N_{max} = 288 N_{design} = 172
Aggregate	Gradation[1]	Optimum AC (%)	Optimum AC (%)	Optimum AC (%)
New York Gravel	Fine	X[1]	X	X
	Coarse	X	X	X
Georgia Granite	Fine	X	X	X
	Coarse	X	X	X
Alabama Limestone	Fine	X	X	X
	Coarse	X	X	X
Nevada Gravel	Fine	X	X	X
	Coarse	X	X	X

Notes: (1) Duplicate specimens compacted to $N_{maximum}$

Table 3.8 Gyrations and Minimum Compaction Slopes (<39°C)

Design ESALs (Millions)	$N_{initial}$	N_{design}	Slope for $N_{initial}$ Density = 89.0%	$N_{initial}$ Density to Provide Slope = 5.86
<0.3	7	68	7.09	90.2
<1.0	7	76	6.76	90.0
<3.0	7	86	6.43	89.6
<10.0	8	96	6.49	89.7
<30.0	8	109	6.17	89.4
<100.0	9	126	6.11	89.3
>100.0	9	142	5.86	89.0

3.7 TASK 7: PREPARE AND RECOMMEND DETAILED REVISIONS OF AASHTO TP-4

Revised test procedures for the Superpave gyratory compaction procedure *(37,41,47, 48)* have been submitted to AASHTO for publication. These revised standards will be reviewed for publication. This study produced a number of proposed changes to the present procedures.

CHAPTER 4 PRESENTATION OF RESULTS, ANALYSIS, AND DISCUSSION

4.1 INTRODUCTION

This section of the report contains the laboratory test results and the analysis and discussion of the results.

4.2 TASK 4: DEVELOPMENT OF SUPERPAVE GYRATORY COMPACTION PROCEDURES FOR GAP GRADED AND LARGE STONE MIXTURES

This task of the study involved evaluating the current Superpave compaction levels of $N_{initial}$, N_{design}, and $N_{maximum}$ for use with gap graded and large stone mixtures. Additionally, field obtained samples of gap graded and large stone mixtures were evaluated to determine a relationship or correlation between the number of gyrations in the Superpave gyratory compactor needed to match the in-place density of the mixtures and the traffic level (ESALs) applied to the mixtures.

4.2.1 Material Properties

Research material used for the completion of Task 4 consisted of four mineral aggregate types and one asphalt binder. The aggregates consisted of an Alabama limestone, a Georgia granite, a New York gravel, and a Nevada gravel. A PG 64-22 asphalt binder was used for all laboratory testing. The same mineral aggregates and asphalt binder were also used for the completion of Task 6C. Various physical properties of the coarse and fine aggregates are provided in Tables 4.1 and 4.2, respectively. The properties of the mineral aggregate greatly influence the field and laboratory performance of hot mix asphalt. The tables show that the four aggregates chosen for evaluation exhibit a wide range of physical properties, such as specific gravity, absorption, particle shape, angularity, hardness, and cleanliness. The absorption of the aggregates will significantly affect the optimum asphalt content of the designed mixture, along with its volumetric properties after compaction. Particle shape and angularity of the aggregates will influence the densification rate and thus the volumetric properties of the mixture. Additionally, the hardness of the aggregate can also influence volumetric properties, as a result of aggregate degradation during compaction. This degradation will result in the mixture gradation becoming finer, and possibly lowering the optimum asphalt content and voids in the mineral aggregate.

By using aggregates with distinctly different properties it was anticipated that a greater confidence could be obtained in the research results. All aggregates selected for evaluation in this project were materials which had been currently being used extensively in HMA in their respective states.

4.2.2 Gap Graded Testing Results

The evaluation and development of Superpave gyratory compaction procedures for gap graded mixtures consisted of two parts. Part I was the evaluation of gap graded mixtures using the existing compaction criteria for Superpave mixtures. Volumetric and compaction properties of the different mixtures were evaluated and compared to current Superpave design requirements for dense graded mixtures. Part II consisted of obtaining gap graded mixtures from the field and determining the number of gyrations to match the in-place density of obtained cores. From this data it was anticipated that a correlation could be developed between the design number of gyrations and traffic levels (ESALs).

4.2.2.1 Laboratory Testing Using Superpave N_{design} Levels

Gap graded mixtures were prepared in the laboratory and evaluated using the Superpave dense mixture compaction protocol. A mix design was conducted for each of aggregates shown in Table 3.1, with the exception of the Ohio Limestone, which was only used in Task 6A. Summary mix design information is provided in Appendix B. The aggregates were blended to meet the gradation for gap graded mixtures shown in Figure 3.1. Optimum asphalt contents were selected to provide 4 percent air voids at N_{design}, and the mixture's volumetric and compaction properties determined. These properties included VMA, VFA, compaction slope, $\%G_{mm}$ at $N_{initial}$, and $\%G_{mm}$ at $N_{maximum}$.

Table 4.1 Physical Properties of Coarse Aggregates

Property	Test Method	Aggregate Type				
		Alabama Limestone	Georgia Granite	Nevada Gravel	New York Gravel	Ohio Limestone
Bulk Specific Gravity	AASHTO T-85	2.752	2.688	2.553	2.556	2.565
Apparent Specific Gravity	AASHTO T-85	2.781	2.735	2.711	2.689	2.815
Absorption (%)	AASHTO T-85	0.4	0.7	2.3	1.8	3.5
Fractured Faces (%)[2]	N/A	100/100	100/100	99/96	74/52	100/100
Flat and Elongated 2:1, 3:1, 5:1	ASTM D4791 (Max - Min)	58,23,2	55,11,1	19,3,0	56,15,1	N/A,11,1
Soundness (%)[1]	AASHTO T-104	0.2	0.3	N/A	16.1	N/A
Los Angeles Abrasion (%)	AASHTO T-96	24	37	20	19	N/A

Notes: (1) Ten cycles with Magnesium Sulfate for the New York Gravel. Five cycles with Sodium Sulfate for all others. (2) 1 or more fractured faces/ 2 or more fractured faces

Table 4.2 Physical Properties of Fine Aggregates

Property	Test Method	Aggregate Type				
		Alabama Limestone	Georgia Granite	Nevada Gravel	New York Gravel	Ohio Limestone
Bulk Specific Gravity	AASHTO T-84	2.648	2.712	2.524	2.593	2.655
Apparent Specific Gravity	AASHTO T-84	2.742	2.736	2.713	2.695	2.800
Absorption (%)	AASHTO T-84	1.3	0.4	2.8	1.5	2.0
Angularity (%)	AASHTO T-33 (Method A)	44.6	49.4	46.7	46.9	49.2
Soundness (%)[1]	AASHTO T-104	0.2	0.3	N/A	23.5	N/A
Sand Equivalency (%)	AASHTO T-176	84	87	65	85	88

Notes: (1) Five cycles with Magnesium Sulfate for the New York Gravel. Five cycles with Sodium Sulfate for all others.

Gyratory compaction levels of $N_{initial}$ (9), N_{design} (128), and $N_{maximum}$ (208) gyrations were chosen for the design compactive effort. This compactive effort corresponds to an average design high air temperature of 41-42°C and a design ESAL count of 10-30 million. These levels of temperature and ESALs are somewhat medium range values, and therefore should provide a good evaluation of the mixtures.

In the mix design, specimens were compacted to N_{design}, not $N_{maximum}$, and their associated properties determined. Once the optimum asphalt content was determined, triplicate specimens were compacted to $N_{maximum}$ for further evaluation. Values of the determined volumetric and compaction properties of the mixtures are shown in Table 4.3. Optimum asphalt contents and VMAs given in Table 4.3 are low, which indicates the gradation may have to be adjusted to provide adequate VMA at this compactive effort. Mixtures of a gap graded nature, generally have asphalt contents which are substantially higher than those determined in this task.

The results indicate that all the mixtures evaluated had densities at $N_{initial}$ well below (86.7 percent was the highest) the maximum specified value of 89 percent. All specimens compacted to $N_{maximum}$ had densities less than the 98 percent maximum limit. Also interesting is the difference in the volumetric properties of the specimens compacted to N_{design} and those compacted to $N_{maximum}$ and back calculated for N_{design}. It is a known fact that error exists between the volumetric properties determined at N_{design} and those back calculated from $N_{maximum}$. For three of the four aggregate mixtures evaluated, there was an error in air voids at N_{design} ranging from 0.2 to 0.4 percent from the back-calculation procedure. This error results in the air voids of the sample being calculated to be higher than actual, which could result in a slightly higher optimum asphalt content for the mixture.

Table 4.3 Gap Graded Mixture Volumetric and Compaction Properties

	Volumetric/Compaction Properties						
Mixture	Optimum Asphalt Content	VMA	VFA	Slope	%G_{mm} at $N_{initial}$	%G_{mm} at N_{design}	%G_{mm} at N_{max}
New York Gravel	4.0	12.1[1]	66.2	8.434	86.2	95.9	-
		12.2[2]	65.2	8.211	85.9	95.7	97.1
Georgia Granite	4.0	12.5	68.3	8.086	86.7	96.0	-
		12.8	68.3	8.258	86.1	95.8	97.4
Alabama Limestone	3.3	11.0	63.8	9.000	85.8	96.0	-
		11.5	61.5	8.271	85.6	95.6	96.9
Nevada Gravel	4.3	11.0	63.8	8.662	86.1	96.0	-
		11.0	63.8	8.478	85.8	96.0	97.4

Notes: (1) Values from mix design conducted at N_{design} = 128 gyrations.
(2) Values back-calculated from specimens compacted to $N_{maximum}$ at optimum asphalt content.

The same back calculation error also exists at $N_{initial}$, although this obviously is not as critical as N_{design}. Compaction slopes also decreased for three of the four aggregates when specimens were compacted to $N_{maximum}$. This can be expected since, for most mixtures, the gyratory compaction or densification curve tends to break over or flatten at gyration levels beyond N_{design}, which would yield a flatter or lower slope value.

4.2.2.2 Field Mixture Evaluation and Testing

The field mixture evaluation part of the task consisted of obtaining samples of materials and cores from six gap graded mixture sections and determining the number of gyrations to match the in-place density of the pavements. The six sections chosen for evaluation were Interstate 80 in Wyoming, Interstate 680 in Nebraska, Interstates 75 and 85 in Georgia, and Interstate 70 and US 50 in Maryland. Summary information for these projects is shown in Table 4.4. As shown, these projects ranged from three to seven years in age and had a wide range of ESALs.

4.2.2.2.1 Wyoming Interstate 80

In 1994, the Wyoming Department of Transportation placed five gap graded mixture sections on Interstate 80, thirty miles east of Rock Springs. In these sections, the effectiveness of cellulose fiber and polymer modified asphalt binder were to be evaluated. One of these sections was chosen for evaluation in this task; a section with an AC-20 polymer modified binder with stabilizing fibers. The mineral aggregate used in the section consisted primarily of a limestone and siliceous river

gravel
material with a locally manufactured mineral filler. The asphalt binder consisted of an AC-20 modified with styrene butadiene (SB) elastomer. The stabilizing fiber used was cellulose fiber pellets. The thickness of the section was approximately 50 mm and was the surface mixture in the pavement structure.

Samples of loose mix obtained during the construction of the section were available for this project and were sent by the Wyoming DOT to NCAT for evaluation. Additionally, two cores (254 mm diameter) were obtained from the wheel paths of the section. For all the field projects evaluated in this task, cores were obtained from the wheel paths of the rightmost lane on a level tangent section of the roadway.

The first step in the evaluation was to determine the bulk specific gravity of the obtained cores. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the in-place density of the sections calculated. Values of average in-place density for the sections can be found in Table 4.4.

Next, the obtained loose mix was heated, extracted from the shipping containers, and quartered into approximately 4700 gram gyratory compactor samples. These samples were then placed in a forced draft oven to reach the specified compaction temperature. After the compaction temperature was achieved, duplicate specimens were compacted in the Pine gyratory compactor at each of 25, 50, 75, and 100 gyrations. Bulk specific gravities of the compacted samples were then determined and specimen density calculated. Values of specimen density at each gyration level can be found in Table 4.5 and are shown graphically in Figure 4.1. As seen, approximately 25 gyrations were required to achieve the average in-place density of the section after 2,520,000 applied ESALs. The average in-place air voids were low (2.6 percent) and the compacted samples had average air voids at 100 gyrations of approximately 0.0 percent.

Table 4.4 Gap Graded Field Mixture Information

	Gap Graded Field Mixture					
Sieve Size (mm)	Georgia Interstate-85	Georgia Interstate-75	Maryland US 50	Maryland Interstate-70	Wyoming Interstate-80	Nebraska Interstate-680
19.0	-	-	100	100	100	100
12.5	100	100	83	82	76	94
9.5	78	80	59	61	54	75
4.75	36	37	23	23	25	34
2.36	23	25	15	17	21	22
1.18	-	-	-	15	-	-
0.6	-	-	-	12	16	-
0.3	15	12	13	11	-	11.6
0.15	-	-	-	10	-	-
0.075	10	8	9.6	9	7.8	9.0
Asphalt Content (%)	5.70	5.80	6.30	6.30	5.75	5.80
Construction Date	September 1991	October 1992	November 1995	June 1995	August 1994	Summer 1995
Accumulated ESALs	14,000,000	8,000,000	1,630,000	1,730,000	2,520,000	800,000
Individual G_{mb} of Cores	2.364 2.382 2.348 2.352 2.375 2.385	2.296 2.320 2.282 2.306 2.342 2.391	2.568 2.562 2.556 2.567 2.573 2.579	2.557 2.569 2.578 2.555 2.567 2.572	2.353 2.374	2.284 2.370 2.344 2.357 2.277
Avg. G_{mb} of Cores	2.369	2.307	2.568	2.566	2.362	2.328
Avg. G_{mm} of Cores	2.442	2.423	2.650	2.634	2.425	2.427
Average In-Place Density, % G_{mm}	97.0	95.2	96.9	97.4	97.4	95.9

Table 4.5 Wyoming Gap Graded Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 80	25	2.329	2.396	97.2	97.4	25
	50	2.374		99.1		
	75	2.389		99.7		
	100	2.396		100.0		

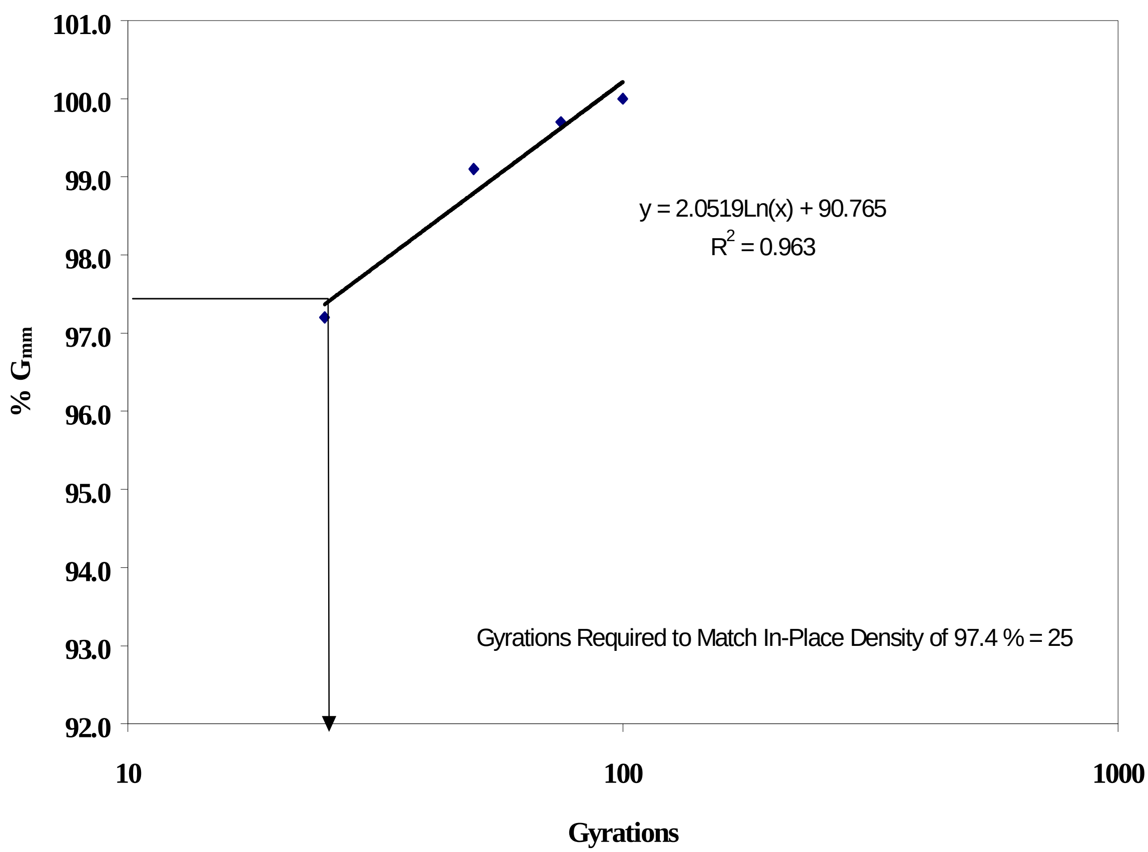

Figure 4.1 Density versus Gyrations for I-80, Wyoming Gap Graded Mixture

4.2.2.2.2 Nebraska Interstate 680

A gap graded section on Interstate 680 north of Omaha, Nebraska, was chosen for evaluation. This section was constructed in 1995, and had the lowest number of applied ESALs (800,000), as seen in Table 4.4.

Samples of loose mix were not available from this project. Therefore, samples of the mineral aggregate and asphalt binder used in the project were obtained and recombined in the laboratory according to the job mix formula. The mineral aggregate used in the project was a combination of limestone and quartzite, with a limestone dust used as the mineral filler. The asphalt binder was an AC-20 binder, with styrene butadiene (SB) elastomeric modification.

A total of five cores (100 mm diameter) was obtained from the project and their bulk specific gravities determined. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the in-place density was calculated. The average in-place density for the section can be found in Table 4.4.

The recombined aggregate blends were mixed with the modified AC-20 asphalt binder. These samples were then placed in a forced draft oven to reach the mixture's compaction temperature. After the compaction temperature was achieved, duplicate specimens were compacted in the Pine gyratory compactor at each of 25, 50, 75, 100, and 125 gyrations. Bulk specific gravities of the compacted samples were then determined and specimen density calculated. Values of specimen density at each gyration level can be found in Table 4.6 and also in Figure 4.2. Approximately 70 gyrations were needed to achieve the average in-place density of the sections after the application of 800,000 ESALs. This is surprising, since the traffic level (ESALs) of this section was the lowest of all mixtures evaluated. However, blending samples in the laboratory does not accurately match the mixture actually produced at the hot mix plant.

Table 4.6 Nebraska Gap Graded Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 680	25	2.229	2.410	92.5	95.9	70
	50	2.280		94.6		
	75	2.314		96.0		
	100	2.342		97.2		
	125	2.359		97.9		

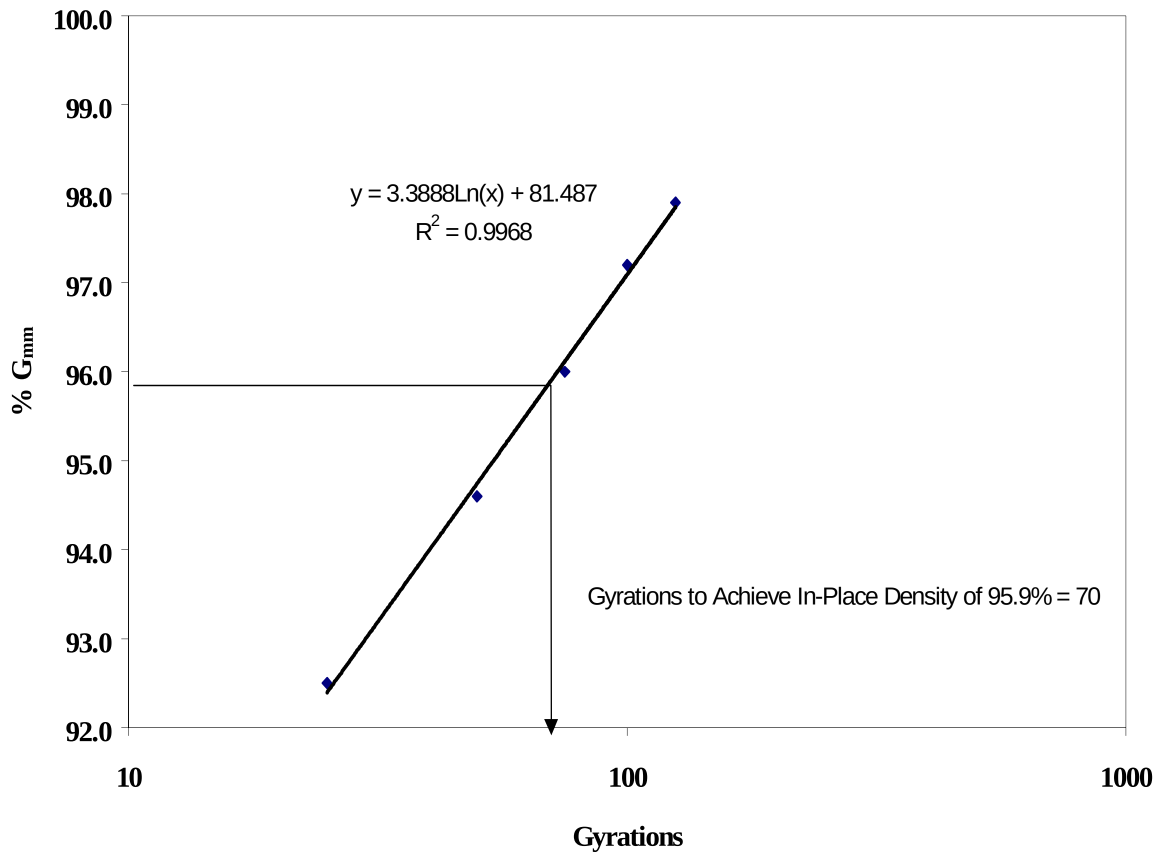

Figure 4.2 Density versus Gyrations for I-680, Nebraska Gap Graded Mixture

4.2.2.2.3 Georgia Interstates 75 and 85

Two gap graded mixtures were evaluated from the state of Georgia. The first mixture was from Interstate 75, just south of Atlanta in Henry County. The second mixture was from Interstate 85 in Jackson County, approximately half way between Atlanta and the South Carolina state line. As is shown in Table 4.4, these sections had the two largest ESAL counts (14,000,000 and 8,000,000) of the gap graded mixtures evaluated in the task.

As was the case with the Nebraska section, samples of loose mix were not available for either of these sections. Samples of the mineral aggregate, asphalt binder, mineral filler, and stabilizing fiber used in the project were obtained and recombined in the laboratory according to the job mix formula. The mineral aggregate used in both sections was granite, although from different sources. Georgia marble dust was used as the mineral filler for both sections. Modified asphalt binders were used in both sections, with styrene butadiene (SB) elastomeric modification used on Interstate 75, and low density polyethylene (LDPE) thermoplastic modification used on Interstate 85. Both sections were approximately 38 mm in depth. The Interstate 75 mixture was placed beneath an open graded friction course mixture, while the Interstate 85 mixture was used as the surface course.

A total of six cores (150 mm diameter) was obtained from each of the sections and their bulk specific gravities determined. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the in-place density of the section calculated. Values of average in-place density for the sections can be found in Table 4.4. The recombined aggregate blends were then mixed with the modified AC-20 asphalt binder. These samples were then placed in a forced draft oven to reach the specified compaction temperature. After the compaction temperature was achieved, duplicate specimens were compacted in the Pine gyratory compactor at each of 25, 50, 75, and 100 gyrations. Bulk specific gravities of the compacted samples were then determined and specimen density calculated. Values of specimen density at each gyration level can be found in Table 4.7 and in Figure 4.3. Gyrations of 25 and 33 were required to match the average in-place density of the Interstate 75 and 85 mixtures, respectively. Again the low number of gyrations determined from testing was somewhat surprising, given that these two sections had the highest traffic levels of all mixtures evaluated. However, the Interstate 75 mixture was placed beneath a friction course which would affect the rate of densification.

Table 4.7 Georgia Gap Graded Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 75	25	2.329	2.446	95.2	95.2	25
	50	2.358		96.4		
	75	2.407		98.4		
	100	2.411		98.6		
Interstate 85	25	2.365	2.458	96.2	97.0	33
	50	2.414		98.2		
	75	2.431		98.9		
	100	2.443		99.4		

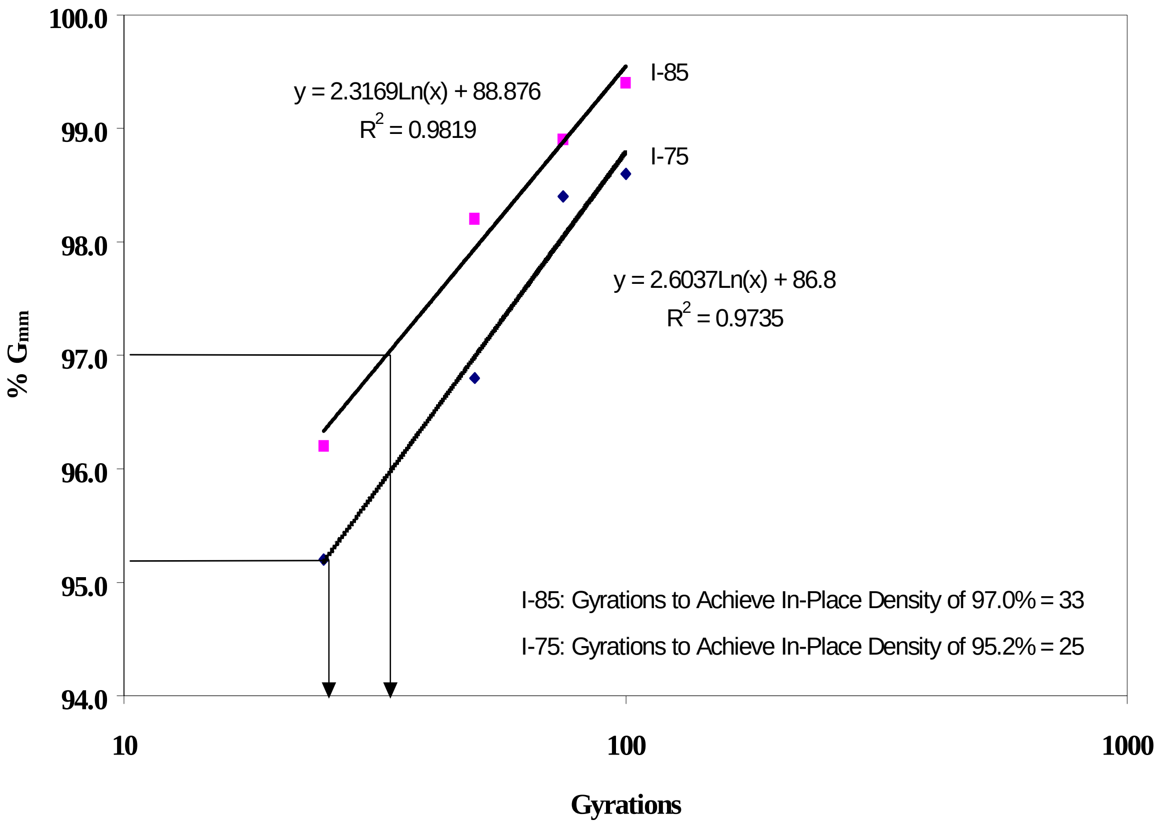

Figure 4.3 Density versus Gyrations for the I-75 and I-85, Georgia Gap Graded Mixtures

4.2.2.2.4 Maryland Interstates 70 and US 50

Two gap graded mixtures were evaluated from the state of Maryland. The first mixture was from Interstate 70, and the second from US 50. Samples of the mixture components were obtained and recombined in the laboratory according to the JMF.

A total of six cores (150 mm diameter) were obtained from each of the sections and their bulk specific gravities determined. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the in-place density of the section calculated. Values of the average in-place density for the sections can be found in Table 4.4.

The recombined aggregate blends were then mixed with an equivalent asphalt binder as used on the project, and were placed in a forced draft oven to reach the specified compaction temperature. After the compaction temperature was achieved, duplicate specimens were compacted in the Pine gyratory compactor at each of 25, 50, 75, and 100 gyrations. Bulk specific gravities of the compacted samples were then determined and specimen density calculated. Values of specimen density at each gyration level can be found in Table 4.8 and Figure 4.4. Gyration levels of 83 and 53 provided the average in-place density of the Interstate 70 and US 50 sections, respectively.

Table 4.8 Maryland Gap Graded Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 70	25	2.446	2.613	93.6	97.4	83
	50	2.516		96.3		
	75	2.532		96.9		
	100	2.558		97.9		
US 50	25	2.504	2.658	94.2	96.9	53
	50	2.584		97.2		
	75	2.610		98.2		
	100	2.621		98.6		

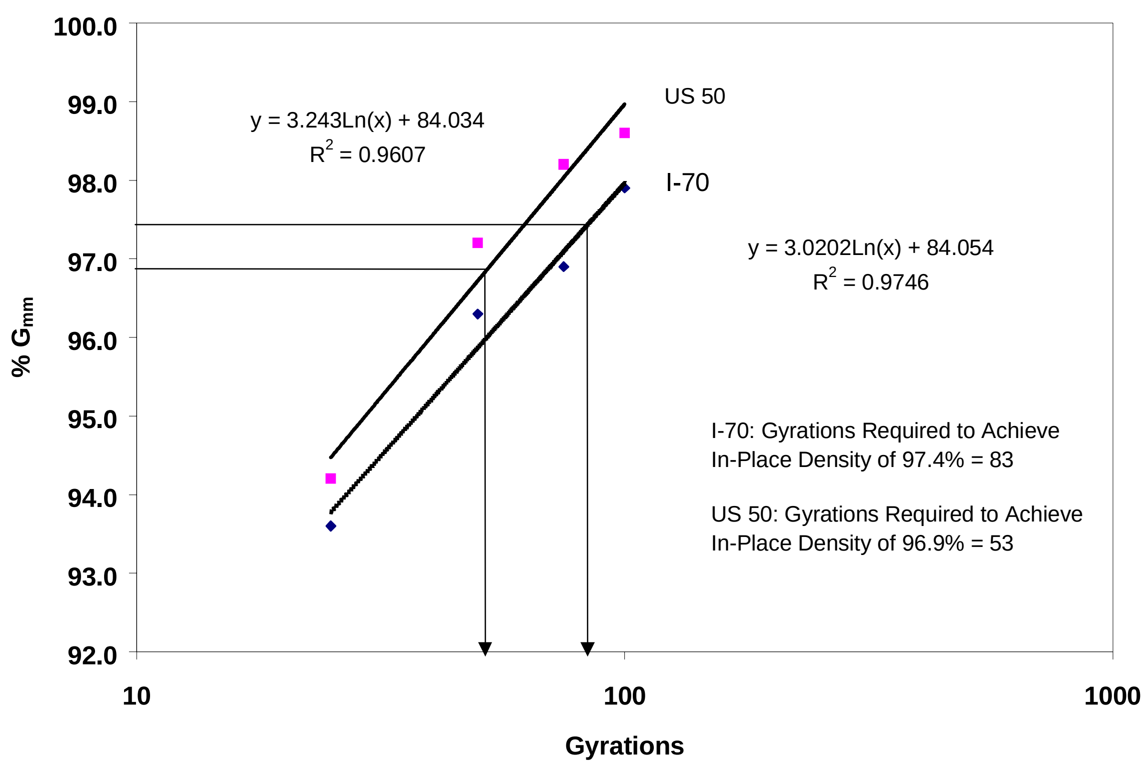

Figure 4.4 Density versus Gyrations for the I-70 and US 50, Maryland Gap Graded Mixtures

4.2.3 Analysis and Discussion of Gap Graded Test Results

The gyratory compactor can be used to compact gap graded mixtures with no difficulty. As mentioned previously, the majority of Superpave mixtures, designed to date, have been coarse graded, which is similar to the gap graded mixtures evaluated. Therefore, it is not a question of can the gyratory compactor be used to design gap graded mixtures, but rather what should be the laboratory compactive effort for gap graded mixtures.

The major goal of this task was to correlate N_{design} with applied traffic levels or ESALs, which would result in gyratory compaction levels for gap graded mixtures. A relationship between gyrations and ESALs is shown in Figure 4.5. The results indicate a relationship between gyrations and ESALs which is obviously incorrect. For the sections evaluated the number of required gyrations decreases with an increase in applied ESALs. It is not logical that this relationship exists; therefore, a possible explanation is provided. Dense graded mixtures evaluated in the original N_{design} experiment were all fine graded. The gap graded mixtures evaluated were all very coarse gradations with high filler (minus 0.075 mm) content. Also, all the mixtures used a polymer modified asphalt binder. Some of the mixtures also used a stabilizing fiber to prevent draindown of the asphalt binder from the mineral aggregate. All of these factors (coarse gradation, high filler content,

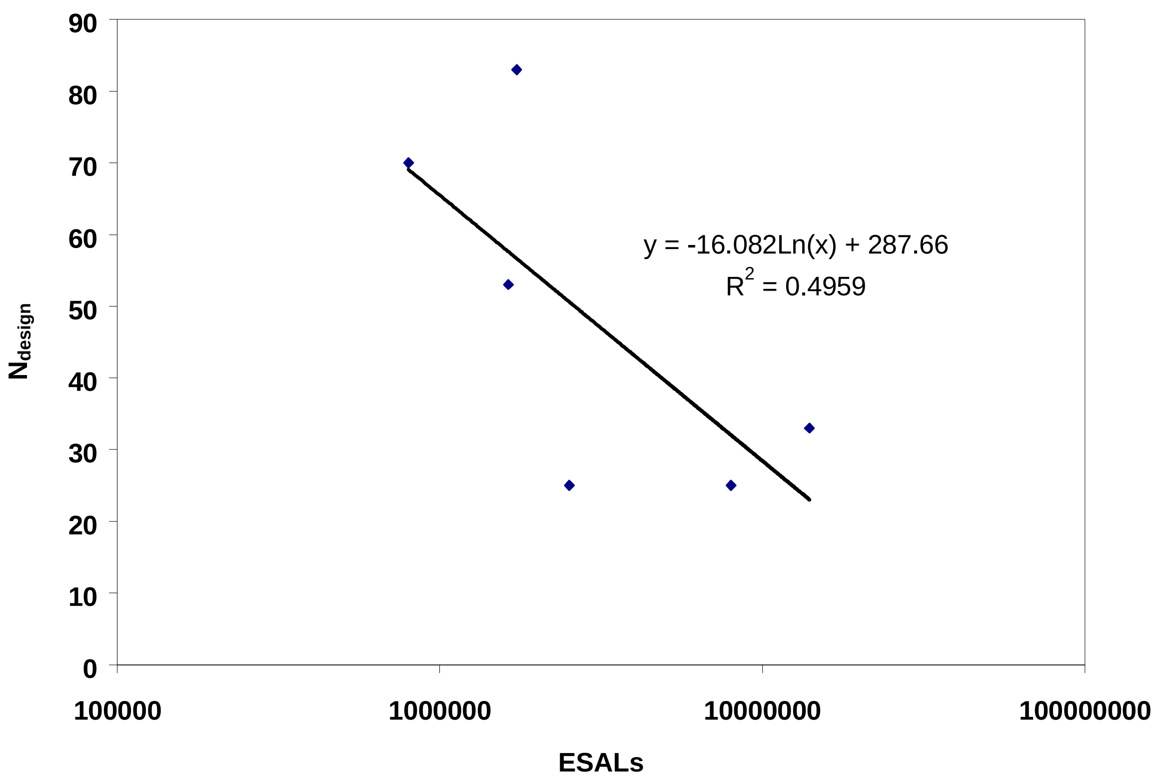

Figure 4.5 N_{design} versus ESALs for Gap Graded Field Mixtures

polymer modified asphalt binder, and stabilizing fiber) would certainly affect the manner in which these mixtures densify with respect to traffic and time in the field.

The sample size used in this study is not sufficient to provide an accurate measure of N_{design} for varying levels of ESALs. However, most gap graded mixtures which have been placed will have similar mixture components as those evaluated. The fact is that the majority of gap graded mixtures are either slightly gaped, as with a Superpave coarse gradation, or severely gaped, as with Stone Matrix Asphalt (SMA). Superpave compaction levels already exist for mixtures which are coarse graded and follow the Superpave gradation requirements. Additionally, recent research work *(38)* has shown that approximately 80 gyrations should be used for the design of SMA mixtures. This N_{design} of 80 gyrations is independent of the design number of ESALs for the project. With the SMA design approach, there is no $N_{initial}$ or $N_{maximum}$ compaction level. Specimens are simply compacted to N_{design} and their volumetric properties determined. The data from reference 38 was used to develop gyratory compaction guidance for SMA. The data developed in Figure 4.5 was not utilized.

4.2.4 Large Stone Testing Results

The evaluation and development of Superpave gyratory compaction procedures for large stone mixtures consisted of two parts. Part I was the laboratory evaluation of large stone mixtures using the existing compaction criteria for Superpave mixtures. Volumetric and compaction properties of the large stone mixtures were evaluated and compared to current Superpave design requirements for dense graded mixtures. Part II consisted of obtaining large stone mixtures from the field and determining the number of gyrations to match the in-place density of obtained cores. From this data it was anticipated that a correlation could be developed between the design number of gyrations and traffic levels (ESALs).

4.2.4.1 Laboratory Testing Using Superpave N_{design} Levels

Large stone mixtures were prepared in the laboratory and evaluated using the Superpave dense mixture compaction protocol. This evaluation was conducted in the same manner as the gap graded mixture evaluation, which was discussed previously. Aggregate sizes for the New York gravel and the Nevada gravel did not allow for the evaluation of a 37.5 mm nominal maximum size mixture. Values of the various volumetric and compaction properties of the mixtures are shown in Table 4.9

Summary mix design information is provided in Appendix B. The results shown in Table 4.9 are indicative of typical large stone mixture design results. Most large stone mixtures generally have optimum asphalt contents ranging from 3.5 to 4.5 percent, with corresponding VMA of 11 to 13.

The designs performed at N_{design} for both aggregates would meet the Superpave volumetric requirements for a 37.5 nominal maximum size mixture, with the only exception being the dust/effective asphalt ratio of the Alabama limestone mixture being slightly lower than the 0.6 minimum value. As with the gap graded mixtures, there were differences between the properties determined at N_{design} and those back calculated from $N_{maximum}$. Because of the size of the mixtures and the low asphalt contents, these differences were more pronounced than with the gap graded mixtures. For both aggregate types, the back calculation of air voids from $N_{maximum}$ showed air voids and VMA which were overestimated by an average of approximately 0.4 percent.

Table 4.9 Large Stone Mixture Volumetric/Compaction Properties

	Volumetric/Compaction Properties						
Mixture	Optimum Asphalt Content	VMA	VFA	Slope	$\%G_{mm}$ at $N_{initial}$	$\%G_{mm}$ at N_{design}	$\%G_{mm}$ at N_{max}
Georgia Granite	4.0	13.1[1]	69.0	8.725	85.9	95.9	-
		13.6[2]	65.9	7.782	86.3	95.6	96.9
Alabama Limestone	3.5	11.7	65.0	10.068	84.3	95.9	-
		12.1	62.4	9.737	84.0	95.4	97.2

Notes: (1) Values from mix design conducted at N_{design} = 128 gyrations.
(2) Values back-calculated from specimens compacted to $N_{maximum}$ at optimum asphalt content.

4.2.4.2 Field Mixture Evaluation and Testing

The field mixture evaluation part of the task consisted of obtaining samples of materials and cores from four large stone mixture sections and determining the number of gyrations to match the in-place density of the pavements. The four sections chosen for evaluation were Interstate 40 in New Mexico (two sections), Interstate 29 in Missouri and Interstate 80 in Wyoming. These projects are shown in Table 4.10. All of the mixtures evaluated were either base or binder courses, with the exception on one of the New Mexico mixtures, which was used as the surface course. The optimum asphalt content for these mixtures varied from 3.5 to 4.0 percent.

4.2.4.2.1 New Mexico Interstate 40 (Two Sections)

Two large stone mixture sections from Interstate 40 in New Mexico were chosen for evaluation in the study. The first section was located near Gallup, near the New Mexico-Arizona border, while the second was located at Rio Puerco, near Albuquerque. The first section consisted of the large stone mixture being used as the surface layer. The second section consisted of 150 mm (two lifts of 75 mm) of large stone mix placed underneath approximately 16 mm of open graded friction course. Only the top lift (75 mm) of the large stone layer was evaluated.

Laboratory work with the large stone mixtures proceeded in the same way as work with the gap graded field mixtures. Samples of loose mix from the construction were not available for either of the sections. Therefore, samples of the mineral aggregates and the asphalt binder were obtained and recombined in the laboratory according to the job mix formula.

A total of five cores (152 mm diameter) were obtained from each of the sections and their bulk specific gravities determined. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the in-place density

Table 4.10 Large Stone Field Mixture Information

	Large Stone Field Mixture			
Sieve Size (mm)	New Mexico Interstate-40 (Gallup)	New Mexico Interstate-40 (Rio Puerco)	Missouri Interstate-29	Wyoming Interstate-80
37.5	100	100	100	100
25.0	94	94	84	87
19.0	72	72	70	74
12.5	65	65	58	63
9.5	58	58	51	55
4.75	40	40	34	36
2.36	-	-	23	27
1.18	-	-	19	22
0.6	-	-	14	17
0.3	15	15	9	13
0.15	-	-	5	8
0.075	5.1	5.1	3	7
Asphalt Content (%)	3.6	3.6	3.5	4
Construction Date	Spring 1992	Summer 1993	December 1993	August 1994
Accumulated ESALs	3,350,000	3,350,000	4,400,000	2,520,000
Individual G_{mb} of Cores	2.381, 2.378 2.399, 2.394 2.383	2.344, 2.337 2.421, 2.403 2.385	2.468, 2.457 2.458, 2.455 2.478	2.390, 2.388 2.401, 2.381 2.392, 2.408 2.416
Avg. G_{mb} of Cores	2.387	2.378	2.463	2.397
Avg. G_{mm} of Cores	2.529	2.498	2.528	2.497
Average In-Place Density, $\%G_{mm}$	94.4	93.6	97.4	96.0

of the section calculated. Values of average in-place density for the section can be found in Table 4.10.

Recombined aggregate blends were mixed with the asphalt binder and were placed in a forced draft oven to reach the specified compaction temperature. After the compaction temperature was achieved, duplicate specimens were compacted in the Pine gyratory compactor at each of 25, 50, 75, 100, and 125 gyrations. Bulk specific gravities of the compacted samples were then determined and specimen density calculated. Values of specimen density at each gyration level can be found in Table 4.11 and Figure 4.6. As can be seen, approximately 33 gyrations were necessary to achieve the in-place density for the I-40, Gallup section. However, for the I-40, Rio Puerco section, specimen densities after only 25 gyrations (95.3 percent of G_{mm}) were greater than the in-place density (93.6 percent). This was unexpected; however, the large stone mixture from Gallup was the surface or wearing course, while the Rio Puerco mixture was placed underneath approximately 16 mm of open graded friction course, which could have significantly slowed the densification of the underlying large stone mixture.

Table 4.11 New Mexico Large Stone Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 40 Gallup	25	2.353	2.514	93.6	94.4	33
	50	2.406		95.7		
	75	2.421		96.3		
	100	2.446		96.7		
	125	2.449		97.3		
Interstate 40 Rio Puerco	25	2.409	2.483	95.3	93.6	<25
	50	2.438		96.3		
	75	2.461		97.7		
	100	2.468		98.5		
	125	2.473		99.0		

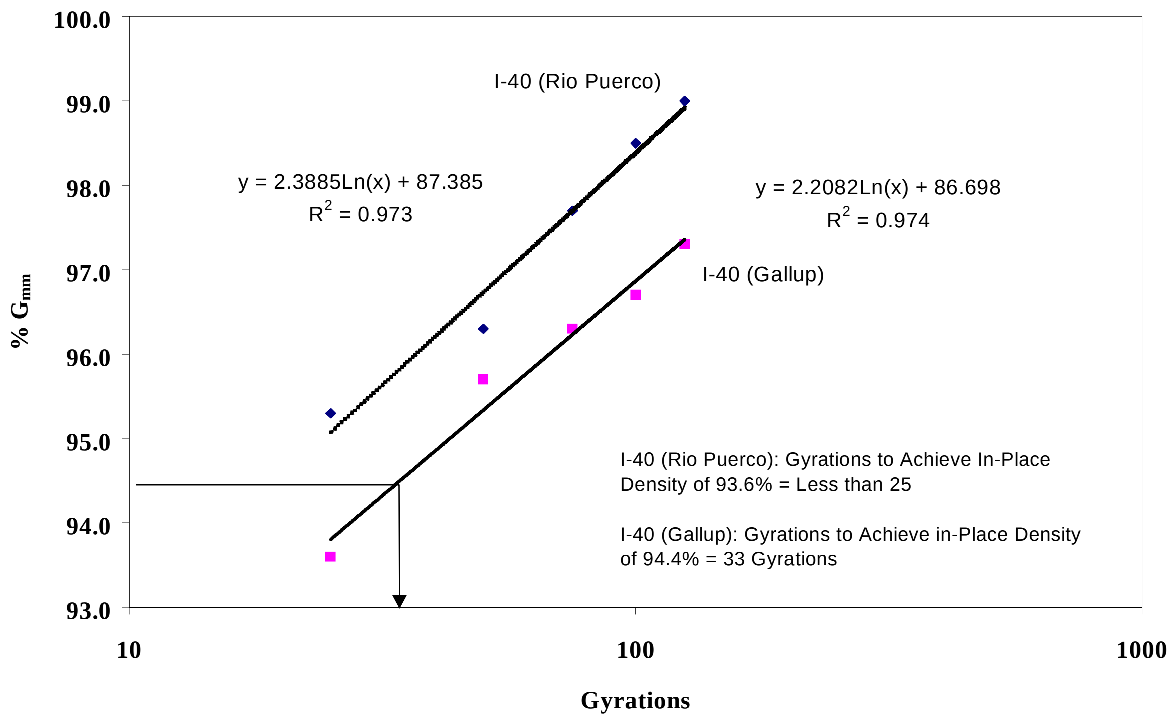

Figure 4.6 Density versus Gyrations for I-40, New Mexico Large Stone Mixtures

4.2.4.2.2 Missouri Interstate 29

A large stone mixture section on Interstate 29, in Platte County, was evaluated in the study. Because the mixture was placed some time ago (1992), some of the mixture components were no longer available for testing, and required other materials to be substituted. All substitute materials were selected to provide similar physical properties as the original materials.

A total of six cores (152 mm diameter) were obtained from the section and their bulk specific gravities determined. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the average in-place density of the section calculated. Values of average in-place density for the section can be found in Table 4.10. Re-mixed aggregate and asphalt blends were placed in a forced draft oven to reach the specified compaction temperature. After the compaction temperature was achieved, duplicate specimens were compacted in the Pine gyratory compactor at each of 25, 50, 75, 100, and 125 gyrations. Bulk specific gravities of the compacted samples were then determined and specimen density calculated. Values of specimen density at each gyration level can be found in Table 4.12 and Figure 4.7. A gyration level of 56 gyrations was shown to provide specimen densities which matched the in-place density of the mixture.

Table 4.12 Missouri Large Stone Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 29	25	2.487	2.601	95.6	97.4	56
	50	2.523		97.0		
	75	2.557		98.3		
	100	2.567		98.7		
	125	2.578		99.1		

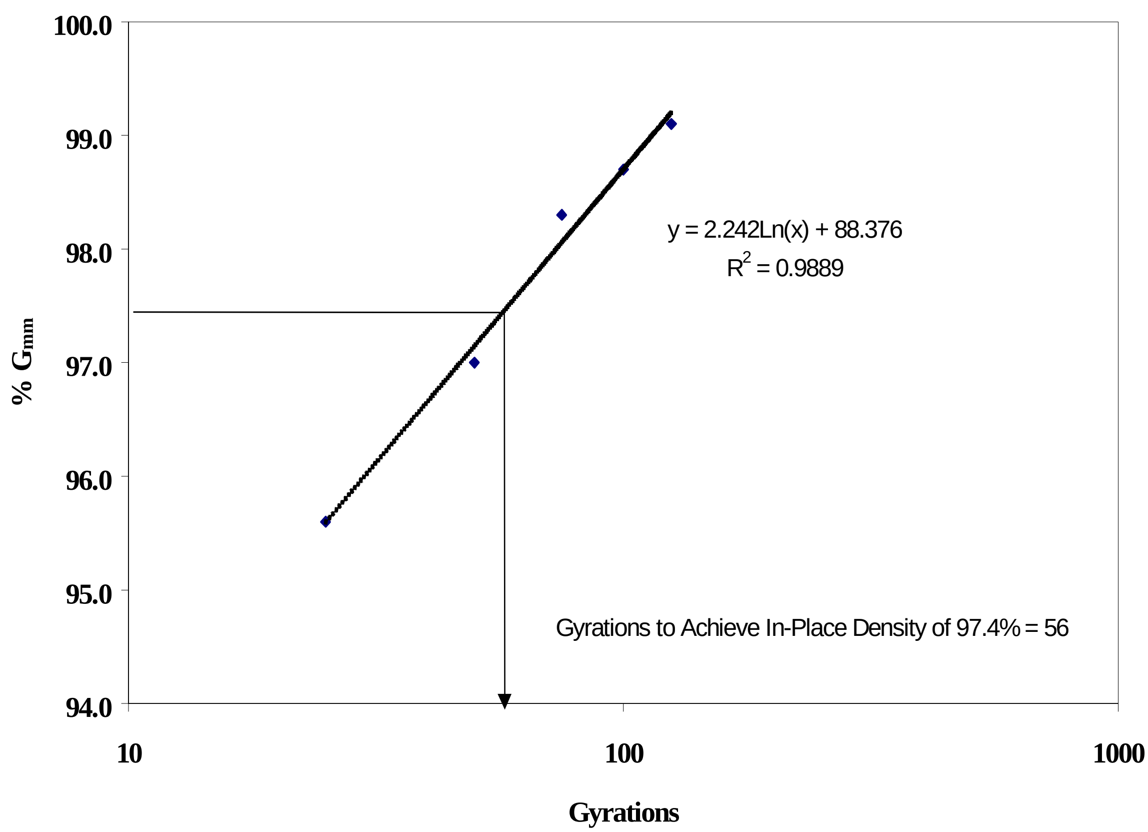

Figure 4.7 Density versus Gyrations for I-29, Missouri Large Stone Mixture

4.2.4.2.3 Wyoming Interstate 80

The large stone mixture evaluated on Interstate 80 was the base mixture for the previously discussed Interstate 80 gap graded mixture. Placement of the large stone mixture occurred at the same time as the gap graded mixture. This mixture was selected so a comparison of the two mixture types could be made on the same project and under the same loading and climate conditions. The section (37.5 mm maximum aggregate size) consisted of similar aggregates as were used in the gap graded mixture. A modified asphalt binder was used for this mixture. The thickness of the mixture was approximately 100 mm, and was placed beneath approximately 50 mm of a gap graded surface mixture.

A total of seven cores was obtained from the project. The average bulk specific gravity of the cores was then compared to the theoretical maximum density, determined from the obtained cores, and the average in-place density of the section calculated. Values of average in-place density are shown in Table 4.10.

From the obtained field cores, the asphalt binder was extracted from the aggregate. Next, the recovered aggregate was mixed with an asphalt binder having approximately the same grade or viscosity as the original asphalt binder, and samples evaluated in the gyratory compactor to levels of 25, 50, and 75 gyrations. Values of specimen density can be found in Table 4.13 and Figure 4.8. Approximately 33 gyrations provided specimen densities which matched the average in-place density of the section.

Table 4.13 Wyoming Large Stone Field Mixture Test Results

Mixture	Gyration Level	G_{mb}	G_{mm}	Laboratory Density (% G_{mm})	In-Place Density (% G_{mm})	Gyrations to Achieve In-Place Density
Interstate 80	25	2.375	2.497	95.1	96.0	33
	50	2.430		97.3		
	75	2.457		98.4		

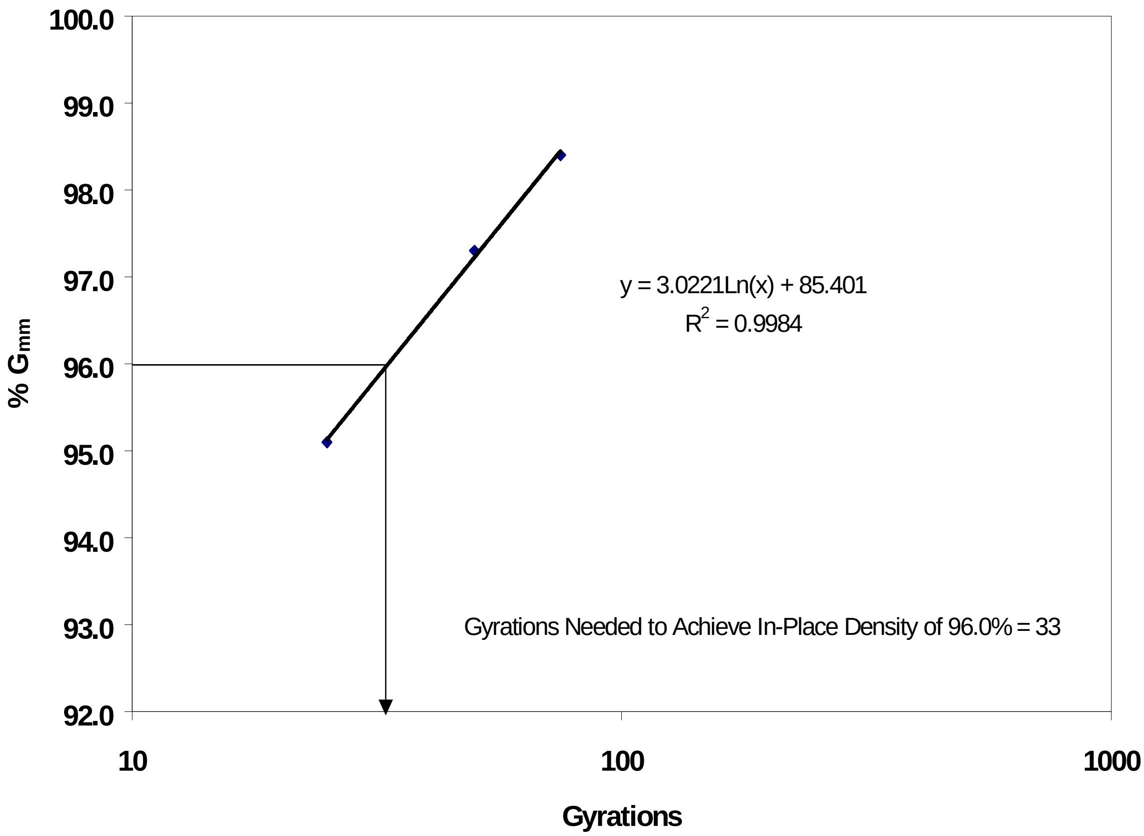

Figure 4.8 Density versus Gyrations for I-80, Wyoming Large Stone Mixture

4.2.5 Analysis and Discussion of Large Stone Results

The results of the laboratory testing indicate that large stone mixtures can be designed using the Superpave gyratory compactor. Volumetric properties from designs performed at a N_{design} of 128 gyrations showed typical properties associated with large stone mixtures which have been designed in the past with the Marshall procedure. However, the task of correlating applied traffic levels or ESALs with the in-place density of large stone mixtures proved a more difficult task. The relationship between ESALs and gyrations is shown in Figure 4.9. The results indicate that for the projects evaluated the number of gyrations required to achieve the in-place density of the field mixtures were well below the current N_{design} levels for the associated traffic levels. The traffic levels of the evaluated field mixtures were such that they would fall in either the 1-3 million or the 3-10 million ESALS categories, as given in AASHTO PP28. The lowest N_{design} values for these categories, which corresponds to a design high air temperature of less than 39°C, are 86 and 96 gyrations, respectively. The required number of gyrations from this limited evaluation ranged from 33 to 56 gyrations. Therefore, based on this limited data, the current N_{design} levels seem to be too high for these large stone mixtures. This is expected since mixes which are placed lower in the pavement structure do not experience the same densification of surface mixtures. Therefore, one would expect, as is shown by this data, that the design compactive effort should be less for mixtures placed in lower layers within the pavement structure.

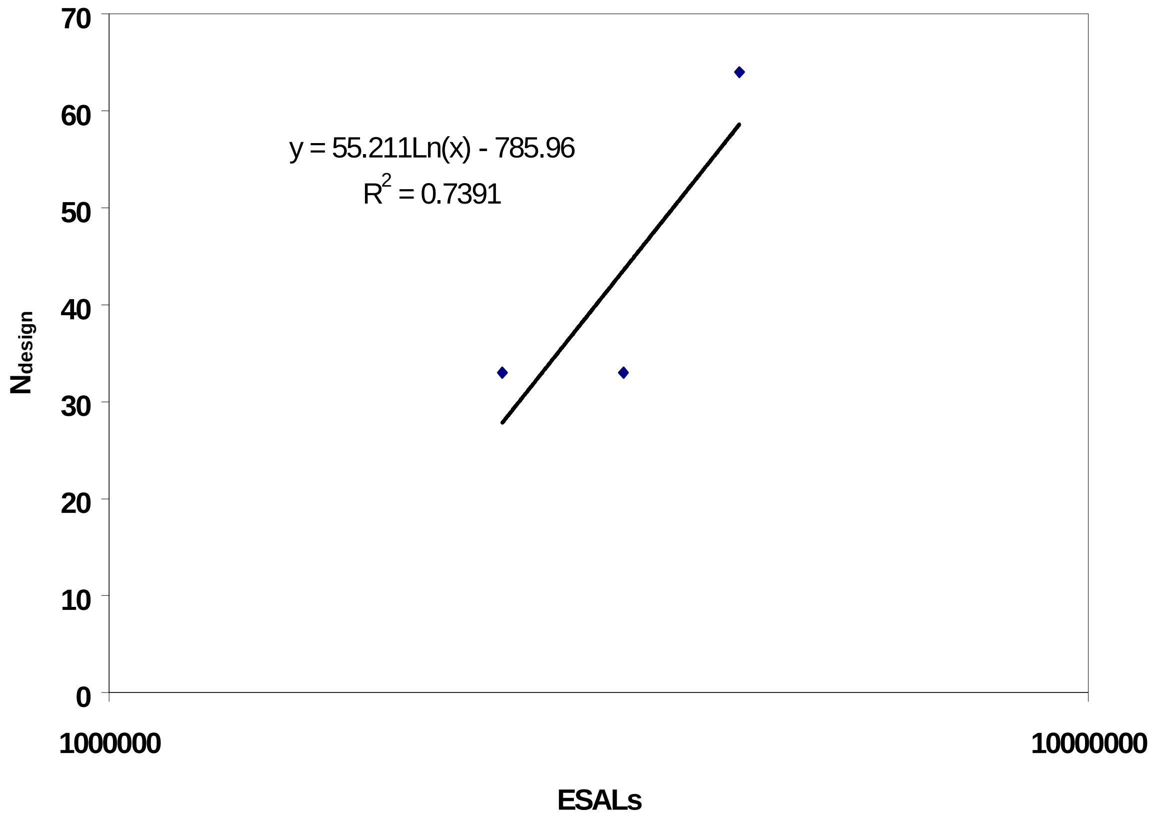

Figure 4.9 N_{design} versus ESALs for Large Stone Mixtures

4.3 TASK 6A: EVALUATION OF THE EFFECT OF VARYING SHORT TERM AGING TEMPERATURE ON MIXTURE VOLUMETRIC PROPERTIES

4.3.1 Test Results

The first step in this task was to determine the appropriate mixing and compaction temperatures for the four asphalt binders evaluated. Table 4.14 indicates mixing and compaction temperatures determined based upon asphalt viscosities. Rotational viscosity was measured at 135˚C and 165˚C to determine the temperature-viscosity relationship of the asphalt binders. The mixing and compaction temperature ranges were selected based on Superpave recommendations (*8)*. These ranges are the temperatures where the asphalt binder has viscosity of 0.17 ± 0.02 Pa-s and 0.28 ± 0.03 Pa-s, respectively.

Table 4.14 Mixing and Compaction Temperatures for Task 6A Asphalt Binders

Asphalt Binder	Mixing Temperature, °C	Compaction Temperature, °C
PG 52-28	137 - 142	126 - 131
AAG-1	139 - 144	129 - 133
PG 64-22	153 - 159	142 - 147
PG 76-22	168 - 172	153 - 157

Since the concept of equiviscous mixing and compaction temperature ranges was established for the Marshall mix design method in the early 1970's (*49*) no consideration was given to modified asphalt binders. Since using the concept of equiviscous mixing and compaction temperatures for modified asphalt binders often result in abnormally high mixing and compaction temperatures (often in excess of 175˚C), the FHWA Mixtures Expert Task Group has recommended that the asphalt binder supplier be consulted to recommend appropriate laboratory mixing and compaction temperatures for modified asphalt binders. For the type and concentration of modifier used in the PG 76-22 asphalt binder, the supplier recommended using temperature-viscosity relationship with different viscosity values. These values were 0.30 Pa-s viscosity for determining mixing temperature and 1.00 Pa-s for determining compaction temperature.

Mixtures were prepared and tested at the design asphalt binder content for each combination of aggregate and gradation. The design asphalt content was selected using a N_{design} of 128 gyrations. The PG 64-22 asphalt binder was used in determining the design asphalt content. Full mix design reports are included in Appendix C. Table 4.15 indicates the design asphalt binder content for each of the eight asphalt mixtures.

Table 4.15 Design Asphalt Binder Content

	Gradation	
Aggregate	Coarse	Fine
NY Gravel	4.7%	4.8%
GA Granite	4.2%	4.3%
AL Limestone	4.5%	3.5%
OH Limestone	5.9%	5.8%

Tables 4.16 through 4.19 indicate the design mixture volumetric and densification properties of the eight aggregate mixtures at optimum asphalt content.

Table 4.16 NY Gravel Design Mixture Properties

		Gradation	
Property	Superpave Criteria	Coarse	Fine
Asphalt Content	n/a	4.7%	4.8%
Air Voids[1]	4.0	4.0%	4.0%
VMA[1]	14.0 % Minimum	12.2%	13.2%
VFA[1]	65 - 75 %	67%	70%
$\%G_{mm}$ @ $N_{initial}$	89 % Maximum	86.8%	89.0%
$\%G_{mm}$ @ $N_{maximum}$	98 % Maximum	97.4%	97.0%
Dust to Effective Asphalt Ratio	0.6 - 1.2	1.0	1.7
G_{mm}	n/a	2.448	2.442
Compaction Slope[2]	n/a	7.98	6.07

[1] Values at N_{design} = 128 gyrations
[2] Calculated as change in $\%G_{mm}$ as a function of the change in the logarithm of the number of gyrations from $N_{initial}$ (9 gyrations) to N_{design} (128 gyrations)

Table 4.17 GA Granite Design Mixture Properties

		Gradation	
Property	Superpave Criteria	Coarse	Fine
Asphalt Content	n/a	4.2%	4.3%
Air Voids[1]	4.0	4.0%	4.0%
VMA[1]	14.0 % Minimum	13.0%	13.6%
VFA[1]	65 - 75 %	69%	70%
$\%G_{mm}$ @ $N_{initial}$	89 % Maximum	87.0%	88.8%
$\%G_{mm}$ @ $N_{maximum}$	98 % Maximum	97.2%	97.0%
Dust to Effective Asphalt Ratio	0.6 - 1.2	1.3	1.9
G_{mm}	n/a	2.537	2.526
Compaction Slope[2]	n/a	7.81	6.24

[1] Values at N_{design} = 128 gyrations
[2] Calculated as change in $\%G_{mm}$ as a function of the change in the logarithm of the number of gyrations from $N_{initial}$ (9 gyrations) to N_{design} (128 gyrations)

Table 4.18 AL Limestone Design Mixture Properties

Property	Superpave Criteria	Gradation: Coarse	Gradation: Fine
Asphalt Content	n/a	4.5%	3.5%
Air Voids[1]	4.0	4.0%	4.0%
VMA[1]	14.0 % Minimum	13.3%	11.2%
VFA[1]	65 - 75 %	70%	64%
$\%G_{mm}$ @ $N_{initial}$	89 % Maximum	84.2%	84.7%
$\%G_{mm}$ @ $N_{maximum}$	98 % Maximum	97.6%	97.3%
Dust to Effective Asphalt Ratio	0.6 - 1.2	1.1	2.8
G_{mm}	n/a	2.554	2.581
Compaction Slope[2]	n/a	10.23	9.80

[1] Values at N_{design} = 128 gyrations
[2] Calculated as change in $\%G_{mm}$ as a function of the change in the logarithm of the number of gyrations from $N_{initial}$ (9 gyrations) to N_{design} (128 gyrations)

Table 4.19 OH Limestone Design Mixture Properties

Property	Superpave Criteria	Gradation: Coarse	Gradation: Fine
Asphalt Content	n/a	5.9%	5.8%
Air Voids[1]	4.0	4.0%	4.0%
VMA[1]	14.0 % Minimum	14.2%	14.8%
VFA[1]	65 - 75 %	72%	73%
$\%G_{mm}$ @ $N_{initial}$	89 % Maximum	84.9%	86.8%
$\%G_{mm}$ @ $N_{maximum}$	98 % Maximum	98.4%	97.9%
Dust to Effective Asphalt Ratio	0.6 - 1.2	0.5	1.4
G_{mm}	n/a	2.481	2.503
Compaction Slope[2]	n/a	9.63	7.98

[1] Values at N_{design} = 128 gyrations
[2] Calculated as change in $\%G_{mm}$ as a function of the change in the logarithm of the number of gyrations from $N_{initial}$ (9 gyrations) to N_{design} (128 gyrations)

Three SGC specimens were compacted for each experimental cell. Specimens were mixed, aged, and compacted at temperatures specified in the experimental matrix. Compaction was stopped at N_{design} (128 gyrations). After the SGC specimens had cooled, the bulk specific gravity, G_{mb}, of the specimens was determined using AASHTO T166. Two specimens were also prepared to determine the mixture's maximum theoretical specific gravity for each experimental cell. Following mixing and aging, the specimens were allowed to cool before determining the maximum theoretical specific gravity, G_{mm}, in accordance with AASHTO T209. The mixture G_{mb}, G_{mm}, and height data from the SGC were then used to determine the compaction characteristics of the specimen set.

Testing results and analysis of each response variable are presented in the following sections of the report.

4.3.1.1 Air Voids at N_{design}

Individual test results for air voids for each of the cells in the experiment are included in the Appendix C to this report. Average test results are indicated in Table 4.20.

Table 4.20 Average Percentage of Air Voids at N_{design}

Asphalt Binder		PG 52-28		AAG-1		PG 64-22		PG 76-22	
Aging Temperature[1]		A	C	A	C	A	C	A	C
NY Gravel	Fine	4.7	4.2	3.8	4.0	4.3	4.4	3.7	3.8
	Coarse	3.6	3.6	3.6	3.1	3.1	3.6	4.3	3.9
GA Granite	Fine	3.8	2.9	3.3	3.3	3.2	2.6	2.6	3.0
	Coarse	3.5	3.1	3.4	3.6	3.3	2.9	3.2	3.1
AL Limestone	Fine	4.0	3.7	3.5	3.3	3.9	3.6	3.7	3.3
	Coarse	3.0	3.6	3.6	3.1	3.6	3.1	3.0	3.1
OH Limestone	Fine	4.2	4.3	n/a	n/a	4.2	4.6	3.0	3.3
	Coarse	5.1	5.3	n/a	n/a	4.5	4.4	3.8	3.8
Average		3.99	3.84	3.53	3.40	3.76	3.65	3.41	3.41
"A" Overall Average		3.68							
"C" Overall Average		3.58							

[1]"A" denotes PP2 temperature (135˚C); "C" denotes mixture compaction temperature (128˚C for PG 52-28, 131˚C for AAG-1, 145˚C for PG 64-22, and 155˚C for PG 76-22).

A two-way analysis of variance (ANOVA) was performed on each row of data, or combination of aggregate type and gradation, in Table 4.20 to determine if the percentage of air voids at N_{design} was affected by asphalt binder type, aging temperature, and/or their interaction. Statistical significance was tested at $\alpha = 0.05$. The results of the ANOVA are indicated in Table 4.21. The results are ordered by asphalt mixture absorption.

Table 4.21 Effect of Asphalt Binder and Aging Temperature on Percentage of Air Voids at N_{design}

			Statistically Significant?		
Mixture	ID	P_{ba}	Asphalt Binder	Aging Temp.	Asphalt Binder X Aging Temp.
OH Limestone Fine	OHF	1.8	yes	yes	no
NY Gravel Coarse	NYC	1.1	yes	no	yes
OH Limestone Coarse	OHC	0.9	yes	no	no
NY Gravel Fine	NYF	0.9	yes	no	yes
AL Limestone Coarse	ALC	0.6	yes	no	yes
AL Limestone Fine	ALF	0.6	yes	yes	no
GA Granite Coarse	GAC	0.4	yes	yes	yes
GA Granite Fine	GAF	0.2	yes	yes	yes

The ANOVA in Table 4.21 indicates that asphalt binder type has a significant effect on the percentage of air voids for all eight aggregate blends. Aging temperature and the interaction of asphalt binder type and aging temperature indicated significant effects for some aggregate blends. To isolate the effects of aging temperature, a sequence of paired comparisons (t-tests) were performed for each asphalt binder and aggregate blend. The results are indicated in Tables 4.22 - 4.25. Table 4.26 presents a summary of all asphalt binders.

Table 4.22 Effect of Aging Temperature on Percentage of Air Voids[1] at N_{design} (PG 52-28)

Asphalt Binder		PG 52-28			
Aging Temp. ˚C		135	128	Diff.	Significant?[2]
NY Gravel	Fine	4.7	4.2	0.5	yes
	Coarse	3.6	3.6	0.0	no
GA Granite	Fine	3.8	2.9	0.9	yes
	Coarse	3.5	3.1	0.4	yes
AL Limestone	Fine	4.0	3.7	0.3	yes
	Coarse	3.0	3.6	0.6	yes
OH Limestone	Fine	4.2	4.3	0.1	no
	Coarse	5.1	5.3	0.2	no

[1] Average of 3 SGC specimens
[2] Significance tested by t-test at $\alpha = 0.05$

Table 4.23 Effect of Aging Temperature on Percentage of Air Voids[1] at N_{design} (AAG-1)

Asphalt Binder		AAG-1			
Aging Temp. ˚C		135	131	Diff.	Significant?[2]
NY Gravel	Fine	3.8	4.0	0.2	no
	Coarse	3.6	3.1	0.5	yes
GA Granite	Fine	3.3	3.3	0.0	no
	Coarse	3.4	3.6	0.2	no
AL Limestone	Fine	3.5	3.3	0.2	yes
	Coarse	3.6	3.1	0.5	yes
OH Limestone	Fine	n/a	n/a	---	---
	Coarse	n/a	n/a	---	---

[1] Average of 3 SGC specimens
[2] Significance tested by t-test at $\alpha = 0.05$

Table 4.24 Effect of Aging Temperature on Percentage of Air Voids[1] at N_{design} (PG 64-22)

Asphalt Binder		PG 64-22			
Aging Temp. ˚C		135	145	Diff.	Significant?[2]
NY Gravel	Fine	4.3	4.4	0.1	no
	Coarse	3.1	3.6	0.5	yes
GA Granite	Fine	3.2	2.6	0.6	yes
	Coarse	3.3	2.9	0.4	yes
AL Limestone	Fine	3.9	3.6	0.3	yes
	Coarse	3.6	3.1	0.5	yes
OH Limestone	Fine	4.2	4.6	0.4	yes
	Coarse	4.5	4.4	0.1	no

[1] Average of 3 SGC specimens

[2] Significance tested by t-test at $\alpha = 0.05$

Table 4.25 Effect of Aging Temperature on Percentage of Air Voids[1] at N_{design} (PG 76-22)

Asphalt Binder		PG 76-22			
Aging Temp. ˚C		135	155	Diff.	Significant?[2]
NY Gravel	Fine	3.7	3.8	0.1	no
	Coarse	4.3	3.9	0.4	yes
GA Granite	Fine	2.6	3.0	0.4	yes
	Coarse	3.2	3.1	0.1	no
AL Limestone	Fine	3.7	3.3	0.4	yes
	Coarse	3.0	3.1	0.1	no
OH Limestone	Fine	3.0	3.3	0.3	yes
	Coarse	3.8	3.8	0.0	no

[1] Average of 3 SGC specimens

[2] Significance tested by t-test at $\alpha = 0.05$

Table 4.26 Effect of Aging Temperature on Percentage of Air Voids at N_{design} (All Binders)

		Significant Difference Between Aging Temperatures?			
Mix Absorption, %	Mix (Agg, Grad)	PG 52-28	AAG-1	PG 64-22	PG 76-22
1.8	OHF	no	n/a	yes	yes
1.1	NYC	no	yes	yes	yes
0.9	OHC	no	n/a	no	no
0.9	NYF	yes	no	no	no
0.6	ALC	yes	yes	yes	no
0.6	ALF	yes	yes	yes	yes
0.4	GAC	yes	no	yes	no
0.2	GAF	yes	no	yes	yes

The data in Tables 4.22- 4.26 indicate no observable trend. The aging temperature had a mixed effect on the percentage of air voids that could not be related to asphalt binder type or mixture absorption. Table 4.26 indicates that aging temperature had a significant effect on the percentage of air voids in 60 percent (18 of 30) of the comparisons. Aging temperature appeared to indicate more significant differences for the four mixtures with the lowest absorption (0.6 percent P_{ba} or less) than the mixtures with highest absorption.

The hypothesis to explain a difference in the percentage of air voids caused by changes in aging temperature can be split into two reactions dependent on the mixture absorption. For low absorption mixtures (GAF, GAC, ALF, and ALC) the reaction is expected to be different from the high absorption mixtures (NYF, OHC, NYC, and OHF) although the response is expected to be the same. The hypothesis can be stated as follows:

If the short-term oven aging of asphalt mixtures occurs at a temperature less than the conventional aging temperature (135 ℃) then the asphalt mixture will have a lower percentage of air voids. For high absorption mixtures, the lower aging temperature results in less asphalt binder absorption, thereby leading to a greater effective asphalt content and lower air voids. For low absorption mixtures, the lower aging temperature should result in less aging (stiffening) of the asphalt binder, thereby leading to greater compactibility, and lower air voids.
If the short-term oven aging of asphalt mixtures occurs at a temperature greater than the conventional aging temperature (135 ℃) then the asphalt mixture will have a higher percentage of air voids. For high absorption mixtures, the higher aging temperature results in more asphalt binder absorption, thereby leading to a lower effective asphalt content and higher air voids. For low absorption mixtures, the higher aging temperature should result in more aging (stiffening) of the asphalt binder, thereby leading to less compactibility, and higher air voids.

Unfortunately, the hypothesis described above was not suitable in explaining differences in air voids for the various mixtures. The hypothesis was correct only half of the time.

The ANOVA in Table 4.21 indicated that asphalt binder type had a significant effect on the percentage of air voids for all mixtures. This was not expected since equiviscous mixing and compaction temperatures should result in equivalent asphalt binder stiffness during compaction. The data in Tables 4.27 and 4.28 indicate average air voids for different asphalt binders at the two aging temperatures.

Several interesting observations can be made from the data in Tables 4.27 and 4.28. The data in Table 4.27 (using 135˚C as the aging temperature) generally indicates a decrease in the percentage of air voids as asphalt binder stiffness increases from a PG 52-28 binder to a PG 76-22 binder. This trend is not as apparent in Table 4.28, which compiles data using the mixture compaction temperature as the aging temperature.

Another observation concerns the range (maximum to minimum) of air voids between the various asphalt binders. The range of air voids in both tables of data is usually highest for the four high absorption mixtures compared to the four low absorption mixtures. It is also noteworthy that the range of air voids is approximately equal for the fine and the coarse graded mixtures for each aggregate. This is illustrated in Figure 4.10.

Table 4.27 Effect of Asphalt Binder on Air Voids at N_{design} (135˚C)

Mix Absorption, %	Mix (Agg, Grad)	%Air Voids PG 52-28	AAG-1	PG 64-22	PG 76-22	Range
1.8	OHF	4.2	n/a	4.2	3.0	1.2
1.1	NYC	3.6	3.6	3.1	4.3	1.2
0.9	OHC	5.1	n/a	4.5	3.8	1.3
0.9	NYF	4.7	3.8	4.3	3.7	1.0
0.6	ALC	3.0	3.6	3.6	3.0	0.6
0.6	ALF	4.0	3.5	3.9	3.7	0.5
0.4	GAC	3.5	3.4	3.3	3.2	0.3
0.2	GAF	3.8	3.3	3.2	2.6	1.2
Individual Average		3.99	3.53	3.76	3.41	-
Overall Average		3.68				-

Table 4.28 Effect of Asphalt Binder on Air Voids at N_{design} (Mix Compaction Temp.)

Mix Absorption, %	Mix (Agg, Grad)	%Air Voids PG 52-28	AAG-1	PG 64-22	PG 76-22	Range
1.8	OHF	4.3	n/a	4.6	3.3	1.3
1.1	NYC	3.6	3.1	3.6	3.9	0.8
0.9	OHC	5.3	n/a	4.4	3.8	1.5
0.9	NYF	4.2	4.0	4.4	3.8	0.6
0.6	ALC	3.6	3.1	3.1	3.1	0.5
0.6	ALF	3.7	3.3	3.6	3.3	0.4
0.4	GAC	3.1	3.6	2.9	3.1	0.7
0.2	GAF	2.9	3.3	2.6	3.0	0.7
Individual Average		3.84	3.41	3.65	3.41	-
Overall Average		3.58				-

In summary, statistical significance was indicated in only 60 percent of all comparisons of aging temperature. No observable trend could be identified to describe the percentage of air voids in the mixture specimens. The hypothesis described previously could only be accepted less than 50 percent of the time. It should also be noted that, even in comparisons when statistical significance was indicated, the difference in the percentage of air voids between the two aging temperatures was as low as 0.2 percent and rarely above 0.5 percent. Following the equations in the Superpave mix design procedure to estimate design asphalt content *(8,37)*, differences in air voids from 0.2 percent to 0.5 percent should only affect the design asphalt binder content by 0.1 percent to 0.2 percent. These differences can be considered relatively minor.

Even though there were significant differences between the air voids for the two aging temperatures 60 percent of the time, neither one of the aging temperatures provided consistently higher air voids. For example, consider the PG 76-22 mixtures; four of these mixtures had significant differences when comparing the two aging temperatures as shown in Table 4.26. However, for the OHF and the GAF mixtures, the mixture's compaction temperature provided the

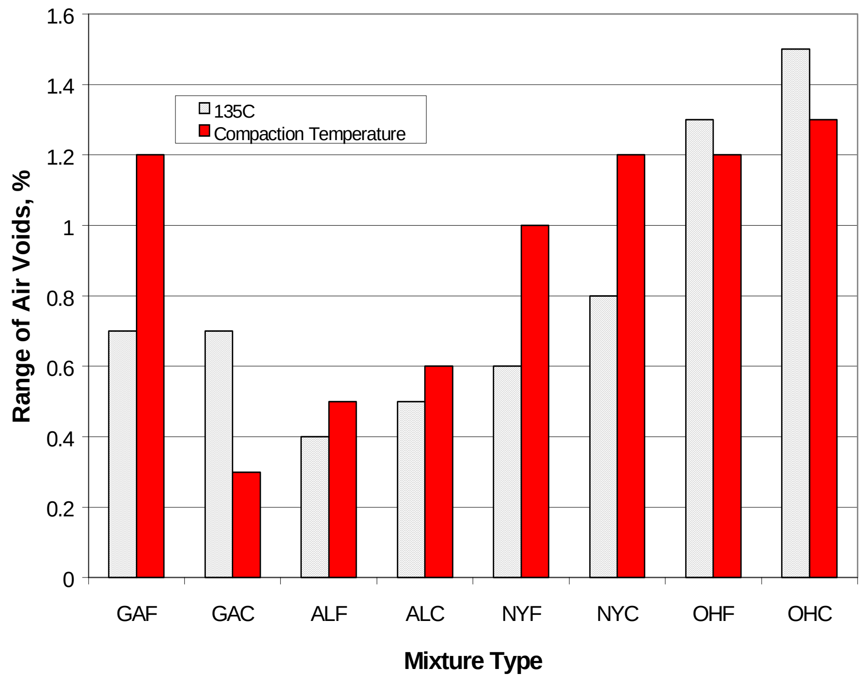

Figure 4.10 Comparison of Air Voids for the Two Aging Temperatures

highest air voids, but for the NYC and the ALF, 135°C provided the highest air voids. So there is no consistent difference between the magnitude of the air voids between aging the mixture at 135°C or at the mixture's compaction temperature. All of this really indicates that the normal variability in the air voids between samples is greater than any differences in air voids caused by aging.

A review of the averages shown in Tables 4.27 and 4.28 indicate that there is no practical differences in average air voids for any mixtures having the same binder. For example, the average voids for the PG 52-28 mixtures is 3.99 with 135°C aging temperature and 3.84 with the mixture's compaction temperature. This is a difference of 0.15 percent air voids and is the worst case. The AAG-1 mixtures have a difference of 0.13 percent voids between the two aging temperatures. The other two binders have differences of 0.11 and 0.00 percent between the two aging temperatures. The overall average difference for all of the mixtures is 0.09 percent.

Two significant points need to be made about the minor differences in air voids between the two aging temperatures. First of all, the differences shown are so small, even if they are statistically significant, there is no practical significance between the two temperatures. Secondly, aging at the mixture's compaction temperature is more realistic because it better represents what actually happens in the field. As a mixture is heated to higher temperatures, the amount of aging increases. Using a constant aging temperature of 135°C does not recognize this effect of temperature on aging.

4.3.1.2 Voids in Mineral Aggregate (VMA)

Individual test results for VMA for each of the cells in the experiment are included in the Appendix C to this report. Average test results are indicated in Table 4.29.

Table 4.29 Average Percentage of VMA at N_{design}

Asphalt Binder		PG 52-28		AAG-1		PG 64-22		PG 76-22	
Aging Temperature[1]		A	C	A	C	A	C	A	C
NY Gravel	Fine	13.1	12.9	12.9	12.9	12.9	13.2	13.0	12.7
	Coarse	12.1	12.0	11.5	11.2	11.7	12.0	12.8	12.3
GA Granite	Fine	13.4	13.0	13.2	13.3	13.2	13.1	12.9	13.1
	Coarse	12.7	12.3	12.5	12.7	12.6	12.2	12.6	12.5
AL Limestone	Fine	11.1	11.0	10.6	10.5	11.1	10.9	11.0	10.8
	Coarse	13.3	13.2	13.0	12.6	13.2	12.8	12.8	12.8
OH Limestone	Fine	14.8	15.2	n/a	n/a	15.0	15.3	14.3	14.5
	Coarse	14.8	15.0	n/a	n/a	14.7	14.3	14.5	13.9
Average		13.16	13.08	12.28	12.20	13.05	12.98	12.99	12.82
"A" Overall Average		12.91							
"C" Overall Average		12.81							

[1]"A" denotes PP2 temperature (135˚); "C" denotes mixture compaction temperature (128˚C for PG 52-28, 131˚C for AAG-1, 145˚C for PG 64-22, and 155˚C for PG 76-22).

A two-way analysis of variance (ANOVA) was performed on each row of data, or combination of aggregate type and gradation, in Table 4.29 to determine if the VMA at N_{design} was affected by asphalt binder type, aging temperature, and/or their interaction. Statistical significance was determined at $\alpha = 0.05$. The results of the ANOVA are indicated in Table 4.30. The results are ordered by asphalt mixture absorption.

Table 4.30 Effect of Asphalt Binder and Aging Temperature on Percentage of VMA at N_{design}

			Statistically Significant?		
Mixture	ID	P_{ba}	Asphalt Binder	Aging Temp.	Asphalt Binder X Aging Temp.
OH Limestone Fine	OHF	1.8	yes	yes	no
NY Gravel Coarse	NYC	1.1	yes	no	yes
OH Limestone Coarse	OHC	0.9	yes	no	no
NY Gravel Fine	NYF	0.9	no	no	yes
AL Limestone Coarse	ALC	0.6	yes	yes	no
AL Limestone Fine	ALF	0.6	yes	yes	no
GA Granite Coarse	GAC	0.4	no	yes	yes
GA Granite Fine	GAF	0.2	yes	no	yes

The ANOVA in Table 4.30 indicates that asphalt binder type has a significant effect on the VMA for six of the eight aggregate blends. Aging temperature and the interaction of asphalt binder type and aging temperature indicated significant effects for some aggregate blends. To isolate the effects of aging temperature, a sequence of paired comparisons (t-tests) were performed for each asphalt binder and aggregate blend. Table 4.31 presents a summary of all asphalt binders.

Table 4.31 Effect of Aging Temperature on Percentage of VMA (All Binders)

		Significant Difference Between Aging Temperatures?			
Mix Absorption, %	Mix (Agg, Grad)	PG 52-28	AAG-1	PG 64-22	PG 76-22
1.8	OHF	yes	n/a	yes	no
1.1	NYC	no	no	no	yes
0.9	OHC	no	n/a	no	no
0.9	NYF	no	no	yes	no
0.6	ALC	no	no	yes	no
0.6	ALF	no	no	no	no
0.4	GAC	yes	no	yes	no
0.2	GAF	yes	no	no	yes

The t-tests indicate that aging temperature had a significant effect on VMA in only 30 percent (9 of 30) of the comparisons. In all instances when a statistically significant difference was indicated the difference in VMA was 0.5 percent or less. In summary, there is no practical difference in the VMA values between the two aging temperatures (0.10 percent overall). The mixture's compaction temperature is a more realistic aging temperature.

4.3.1.3 Maximum Theoretical Specific Gravity (G_{mm})

Individual test results of G_{mm} for each of the cells in the experiment are included in the Appendix C to this report. Average test results are indicated in Table 4.32.

Table 4.32 Average G_{mm}

Asphalt Binder		PG 52-28		AAG-1		PG 64-22		PG 76-22	
Aging Temperature[1]		A	C	A	C	A	C	A	C
NY Gravel	Fine	2.457	2.451	2.441	2.446	2.453	2.445	2.435	2.443
	Coarse	2.444	2.446	2.455	2.459	2.442	2.445	2.441	2.446
GA Granite	Fine	2.529	2.518	2.524	2.520	2.518	2.505	2.514	2.516
	Coarse	2.537	2.535	2.537	2.540	2.535	2.532	2.531	2.532
AL Limestone	Fine	2.582	2.577	2.580	2.579	2.577	2.575	2.574	2.569
	Coarse	2.546	2.546	2.553	2.550	2.545	2.544	2.539	2.544
OH Limestone	Fine	2.512	2.503	n/a	n/a	2.507	2.508	2.495	2.498
	Coarse	2.497	2.497	n/a	n/a	2.485	2.494	2.473	2.488
Average		2.513	2.509	2.515	2.516	2.508	2.506	2.500	2.504
"A" Overall Average		2.509							
"C" Overall Average		2.509							

[1]"A" denotes PP2 temperature (135˚); "C" denotes mixture compaction temperature (128˚C for PG 52-28, 131˚C for AAG-1, 145˚C for PG 64-22, and 155˚C for PG 76-22).

A two-way analysis of variance (ANOVA) was performed on each row of data, or combination of aggregate type and gradation, in Table 4.32 to determine if the G_{mm} was affected by asphalt binder type, aging temperature, and/or their interaction. Statistical significance was tested at $\alpha = 0.05$. The results of the ANOVA are indicated in Table 4.33. The results are ordered by asphalt mixture absorption.

Table 4.33 Effect of Asphalt Binder and Aging Temperature on G_{mm}

			Statistically Significant?		
Mixture	ID	P_{ba}	Asphalt Binder	Aging Temp.	Asphalt Binder X Aging Temp.
OH Limestone Fine	OHF	1.8	no	no	no
NY Gravel Coarse	NYC	1.1	yes	no	no
OH Limestone Coarse	OHC	0.9	yes	yes	yes
NY Gravel Fine	NYF	0.9	yes	no	no
AL Limestone Coarse	ALC	0.6	no	no	no
AL Limestone Fine	ALF	0.6	no	no	no
GA Granite Coarse	GAC	0.4	yes	no	no
GA Granite Fine	GAF	0.2	no	no	no

The ANOVA in Table 4.33 indicates that asphalt binder type has a significant effect on the maximum theoretical specific gravity for half of the eight aggregate blends. Aging temperature and the interaction of asphalt binder type and aging temperature indicated no significant effects for most of the aggregate blends.

In summary, there is no significant difference between the G_{mm} values for the two aging temperatures. Table 4.32 shows that the biggest average difference between the two aging temperatures for any of the asphalt binders is 0.004. This will have no significant effect on air void calculation. Also by observing Table 4.32, it is seen that there is no difference (0.000) between the overall averages for the two aging temperatures. Again, the data supports using the mixture's compaction temperature as the short term aging temperature.

4.3.1.4 Compaction at $N_{initial}$ (%G_{mm} @ $N_{initial}$)

Individual test results for %G_{mm} at $N_{initial}$ for each of the cells in the experiment are included in the Appendix C to this report. Average test results are indicated in Table 4.34.

A two-way analysis of variance (ANOVA) was performed on each row of data, or combination of aggregate type and gradation, in Table 4.34 to determine if the compaction at $N_{initial}$ was affected by asphalt binder type, aging temperature, and/or their interaction. Statistical significance was tested at $\alpha = 0.05$. The results of the ANOVA are indicated in Table 4.35. The results are ordered by asphalt mixture absorption.

The ANOVA in Table 4.35 indicates that asphalt binder type has a significant effect on the compaction at $N_{initial}$ for all of the eight aggregate blends. Aging temperature and the interaction of asphalt binder type and aging temperature indicated significant effects for half of the aggregate blends. To isolate the effects of aging temperature, a sequence of paired comparisons (t-tests) were performed for each asphalt binder and aggregate blend. The results are indicated in Table 4.36 for all asphalt binders.

Table 4.34 Average $\%G_{mm}$ @ $N_{initial}$

Asphalt Binder		PG 52-28		AAG-1		PG 64-22		PG 76-22	
Aging Temperature[1]		A	C	A	C	A	C	A	C
NY Gravel	Fine	88.4	88.6	89.1	88.8	88.6	88.6	89.1	89.1
	Coarse	87.0	86.6	86.8	87.2	87.4	86.6	86.3	86.0
GA Granite	Fine	89.0	89.8	89.5	89.5	89.5	90.1	89.9	89.5
	Coarse	87.2	87.3	87.0	86.5	87.1	87.7	86.8	86.7
AL Limestone	Fine	85.3	85.3	85.4	85.6	85.1	85.4	85.5	86.0
	Coarse	85.3	84.8	84.1	85.0	84.6	85.0	85.5	85.2
OH Limestone	Fine	86.4	86.2	n/a	n/a	86.3	86.0	87.6	87.3
	Coarse	83.8	83.5	n/a	n/a	84.1	84.3	85.0	84.8
Average		86.55	86.51	87.00	87.10	86.59	86.71	86.96	86.82
"A" Overall Average		86.78							
"C" Overall Average		86.78							

[1]"A" denotes PP2 temperature (135˚); "C" denotes mixture compaction temperature (128˚C for PG 52-28, 131˚C for AAG-1, 145˚C for PG 64-22, and 155˚C for PG 76-22).

Table 4.35 Effect of Asphalt Binder and Aging Temperature on $\%G_{mm}$ @ $N_{initial}$

			Statistically Significant?		
Mixture	ID	P_{ba}	Asphalt Binder	Aging Temp.	Asphalt Binder X Aging Temp.
OH Limestone Fine	OHF	1.8	yes	yes	no
NY Gravel Coarse	NYC	1.1	yes	yes	yes
OH Limestone Coarse	OHC	0.9	yes	no	no
NY Gravel Fine	NYF	0.9	yes	no	yes
AL Limestone Coarse	ALC	0.6	yes	no	yes
AL Limestone Fine	ALF	0.6	yes	yes	no
GA Granite Coarse	GAC	0.4	yes	no	yes
GA Granite Fine	GAF	0.2	yes	yes	yes

Table 4.36 Effect of Aging Temperature on $\%G_{mm}$ at $N_{initial}$ (All Binders)

		Significant Difference Between Aging Temperatures?			
Mix Absorption, %	Mix (Agg, Grad)	PG 52-28	AAG-1	PG 64-22	PG 76-22
1.8	OHF	no	n/a	yes	no
1.1	NYC	no	yes	yes	no
0.9	OHC	no	n/a	no	no
0.9	NYF	no	yes	no	no
0.6	ALC	yes	no	yes	no
0.6	ALF	no	no	yes	yes
0.4	GAC	no	no	yes	no
0.2	GAF	yes	no	yes	yes

The data in Table 4.36 indicates no observable trend. The aging temperature had a mixed effect on the compaction at $N_{initial}$ that could not be related to asphalt binder type or mixture absorption. The t-tests indicate that aging temperature had a significant effect on the $\%G_{mm}$ at $N_{initial}$ in only 40 percent (12 of 30) of the comparisons. In most instances when a statistically significant difference was indicated the difference in compaction was 0.5 percent or less. Only 3 of 30 comparisons indicated that aging temperature caused a significant difference in the $\%G_{mm}$ at $N_{initial}$ greater than 0.5 percent.

The average results in Table 4.34 clearly show that there is no practical difference between the $\%G_{mm}$ at $N_{initial}$ value for the two aging temperatures. The worst case between the aging temperature for each of the four asphalt binders was 0.14 percent for the PG 76-22. Also by observing Table 4.34, it is seen that there is no difference (0.000) between the overall averages for the two aging temperatures. These results support changing the short term aging temperature from 135°C to the mixture's compaction temperature.

4.3.1.5 Compaction Slope

Individual test results for compaction slope for each of the cells in the experiment are included in the Appendix C to this report. Average test results are indicated in Table 4.37.

Table 4.37 Average Compaction Slopes

Asphalt Binder		PG 52-28		AAG-1		PG 64-22		PG 76-22	
Aging Temperature[1]		A	C	A	C	A	C	A	C
NY Gravel	Fine	6.05	6.23	6.11	6.25	6.12	6.10	6.20	6.23
	Coarse	8.14	8.52	8.34	8.35	8.25	8.49	8.20	8.75
GA Granite	Fine	6.27	6.28	6.25	6.26	6.35	6.39	6.53	6.51
	Coarse	8.04	8.32	8.38	8.58	8.35	8.16	8.64	8.81
AL Limestone	Fine	9.30	9.52	9.63	9.66	9.54	9.58	9.37	9.30
	Coarse	10.15	10.13	10.61	10.31	10.27	10.29	9.99	10.16
OH Limestone	Fine	8.16	8.18	n/a	n/a	8.20	8.10	8.19	8.20
	Coarse	9.69	9.68	n/a	n/a	9.86	9.81	9.70	9.8
Average		8.22	8.36	8.22	8.24	8.37	8.36	8.35	8.47
"A" Overall Average		8.29							
"C" Overall Average		8.36							

[1]"A" denotes PP2 temperature (135˚); "C" denotes mixture compaction temperature (128˚C for PG 52-28, 131˚C for AAG-1, 145˚C for PG 64-22, and 155˚C for PG 76-22).

A two-way analysis of variance (ANOVA) was performed on each row of data, or combination of aggregate type and gradation, in Table 4.37 to determine if the compaction at $N_{initial}$ was affected by asphalt binder type, aging temperature, and/or their interaction. Statistical significance was tested at $\alpha = 0.05$. The results of the ANOVA are indicated in Table 4.38. The results are ordered by asphalt mixture absorption.

Table 4.38 Effect of Asphalt Binder and Aging Temperature on Compaction Slope

			Statistically Significant?		
Mixture	ID	P_{ba}	Asphalt Binder	Aging Temp.	Asphalt Binder X Aging Temp.
OH Limestone Fine	OHF	1.8	no	no	no
NY Gravel Coarse	NYC	1.1	no	yes	yes
OH Limestone Coarse	OHC	0.9	no	no	no
NY Gravel Fine	NYF	0.9	no	yes	no
AL Limestone Coarse	ALC	0.6	yes	no	no
AL Limestone Fine	ALF	0.6	yes	no	yes
GA Granite Coarse	GAC	0.4	yes	yes	yes
GA Granite Fine	GAF	0.2	yes	no	no

The ANOVA in Table 4.38 indicates that asphalt binder type has a significant effect on the compaction slope for half of the eight aggregate blends. Aging temperature and the interaction of asphalt binder type and aging temperature indicated no significant effects for less than half of the aggregate blends. A sequence of paired comparisons (t-tests) were performed for each asphalt binder and aggregate blend. The results are indicated in Table 4.39 for all asphalt binders. The data in Table 4.39 indicates that aging temperature had a significant effect on the compaction slope in only 27 percent (8 of 30) of the comparisons.

Table 4.39 Effect of Aging Temperature on Compaction Slope (All Binders)

		Significant Difference Between Aging Temperatures?			
Mix Absorption, %	Mix (Agg, Grad)	PG 52-28	AAG-1	PG 64-22	PG 76-22
1.8	OHF	no	n/a	no	no
1.1	NYC	yes	no	yes	yes
0.9	OHC	no	n/a	no	no
0.9	NYF	yes	yes	no	no
0.6	ALC	no	no	no	no
0.6	ALF	yes	no	no	no
0.4	GAC	yes	no	no	yes
0.2	GAF	no	no	no	no

The average compaction slopes are shown in Table 4.37. These results indicate that the biggest difference in compaction slope between the two aging temperatures for any of the four asphalt binders is 0.14 percent. The overall average difference is 0.07 percent. This indicates that there is no practical difference in compaction slope for the two aging temperatures.

4.3.2 Test Results: Aging Time and Temperature Effects

Using the data collected in this project, it was decided to evaluate the effect of aging time and temperature on air voids. This was a very limited analysis, but it is provided here to give some indication of expected results.

Individual test results for air voids for each of the cells in the experiment, shown in Table 3.4, are included in the Appendix C to this report. Average test results are indicated in Table 4.40

Table 4.40 Average Percentage of Air Voids at N_{design}

Asphalt Binder		PG 64-22				PG 76-22			
Aging Time, hrs.		2		4		2		4	
Aging Temp. ˚C		135	145	135	145	135	155	135	155
GA Granite	Fine	3.6	3.1	3.2	2.6	3.4	2.8	2.6	3.0
	Coarse	3.4	3.6	3.3	2.9	2.9	2.9	3.2	3.1
2 Hour Overall Average		3.20							
4 Hour Overall Average		3.12							

A two-way analysis of variance (ANOVA) was performed on each combination of aggregate gradation and asphalt binder in Table 4.40 to determine if the percentage of air voids at N_{design} was affected by aging time, aging temperature, and/or their interaction. Statistical significance was tested at $\alpha = 0.05$. The results of the ANOVA are indicated in Table 4.41.

Table 4.41 Effect of Aging Time and Temperature on Air Voids @ N_{design}

		Statistically Significant?		
Mixture	Asphalt	Aging Time	Aging Temp.	Time X Temp.
GA Granite Fine	PG 64-22	yes	yes	no
GA Granite Coarse	PG 64-22	yes	no	yes
GA Granite Fine	PG 76-22	yes	no	yes
GA Granite Coarse	PG 76-22	yes	no	no

The ANOVA in Table 4.41 indicates that aging time has a significant effect on the percentage of air voids at N_{design} for all four mixtures. Aging temperature and the interaction of aging time and temperature indicated some significant effects. To isolate the effects of aging time, a sequence of paired comparisons (t-tests) was performed. The results are indicated in Table 4.42.

Table 4.42 Effect of Aging Time on Percentage of Air Voids at N_{design}

		Significant Difference Between Aging Times?	
Mix	Asphalt Binder	135˚C	Comp. Temp.
GA Granite Fine	PG 64-22	yes	yes
GA Granite Coarse	PG 64-22	no	yes
GA Granite Fine	PG 76-22	yes	yes
GA Granite Coarse	PG 76-22	no	yes

The t-tests indicate that aging time had a significant effect on the percentage of air voids in 6 of 8 comparisons. In most instances when a statistically significant difference was indicated the difference in compaction was 0.5 percent or less. In 4 of 6 cases when a significant difference was indicated the shorter aging time (2 hours) resulted in a higher percentage of air voids. This result does not match expectations as the longer aging time should produce a higher percentage of air voids through either: (a) more absorption in absorptive mixes or (b) stiffer asphalt binder in non-absorptive mixes. Since the percentage of air voids is calculated using the mixture G_{mb} at N_{design} an analysis was conducted also on this response variable. Average test results are indicated in Table 4.43.

Table 4.43 Average G_{mb} at N_{design}

Asphalt Binder		PG 64-22				PG 76-22			
Aging Time, hrs.		2		4		2		4	
Aging Temp. ˚C		135	145	135	145	135	155	135	155
GA Granite	Fine	2.443	2.452	2.439	2.441	2.450	2.444	2.449	2.440
	Coarse	2.455	2.449	2.450	2.460	2.464	2.461	2.449	2.452

To isolate the effects of aging time, a sequence of paired comparisons (t-tests) were performed. The results are indicated in Table 4.44. The t-tests indicate that aging time had a significant effect on the percentage of air voids in 5 of 8 comparisons. In 4 of 5 cases when a significant difference was indicated the shorter aging time (2 hours) resulted in a higher G_{mb} at N_{design}.

Table 4.44 Effect of Aging Time on G_{mb} at N_{design}

		Significant Difference Between Aging Times?	
Mix	Asphalt Binder	135˚C	Comp. Temp.
GA Granite Fine	PG 64-22	no	yes
GA Granite Coarse	PG 64-22	yes	yes
GA Granite Fine	PG 76-22	no	no
GA Granite Coarse	PG 76-22	yes	yes

This result matches expectations as the shorter aging time should produce a higher compacted bulk specific gravity as the mix is less oxidized and therefore more compactible. The data in Table 4.43 also indicates that in 7 of 8 cases, the G_{mb} at N_{design} is higher at the shorter aging time (2 hours) than the longer aging time (4 hours). In most cases the higher G_{mb} would result in less than a 0.5 percent change in air voids, assuming the same maximum theoretical specific gravity.

A final analysis was performed on the data in Table 4.40 to determine if performing short-term oven aging for two hours at the mixture compaction temperature would result in mixture specimens with approximately the same percentage of air voids as specimens prepared using four hours of short-term oven aging at 135˚C. The results of the paired comparisons are indicated in Table 4.45 The t-tests indicate that the interaction of aging time and temperature had a significant effect on the percentage of air voids in half of the comparisons. In both cases when a significant difference was indicated the shorter aging time/higher temperature condition (2 hours at mix compaction temperature) resulted in a higher percentage of air voids at N_{design} than the longer aging time/lower temperature condition (4 hours at 135˚C). However, there was only 0.3 percent difference in the percentage of air voids for the two aging conditions. Therefore, this analysis would support the null hypothesis (Ho).

Table 4.45 Comparison of Aging Time and Temperature Interactions (Air Voids)

Mix	Asphalt Binder	Significant Difference Between Aging Conditions?
GA Granite Fine	PG 64-22	no
GA Granite Coarse	PG 64-22	yes
GA Granite Fine	PG 76-22	yes
GA Granite Coarse	PG 76-22	no

4.3.3 Discussion of Testing Results

An experiment was conducted to determine if aging temperature had a significant effect on an asphalt mixture's volumetric and densification properties. Eight aggregate blends were used in the study (four aggregates and two gradations) with different absorption characteristics. Four asphalt binders were also used, representing a common range of asphalt binder used in Superpave mixtures.

The main controlled variable studied was the short term aging temperature. Standard practice requires short term aging to be performed on loose asphalt mixtures for four hours at 135°C. The experiment was designed to study if the compaction temperature of the asphalt mixture could be used as the aging temperature without significantly affecting the mixture's volumetric and densification properties.

Analysis of the data indicated that there was no practical difference between mixtures aged at 135°C and the mixture's compaction temperature. When comparing specific mixtures, there was often a statistical significance between the two aging temperatures. However, in some cases 135°C provided higher properties and in other cases the mixture's compaction temperature provided higher properties. This indicates that normal variation in properties was greater than the effect caused by aging temperature. When the overall averages were investigated, there was no case where the two aging temperatures made a practical difference in results.

If aging temperature was found to have a significant effect on the results, this would be even more reason to adopt the mixture's compaction temperature as the specified short term aging temperature. During plant production the mixture is heated to approximately that temperature required for compaction. If aging temperature is important, then it is necessary to age the mixture in the laboratory at a similar temperature as the mixture experiences during production.

In summary, the aging temperatures evaluated appear to have no practical effect on the measured mixture properties, and therefore the mixture's compaction temperature should be adopted for use as the short term aging temperature.

A supplemental experiment was performed to study the effects of aging time and temperature on mixture volumetrics and densification properties. The FHWA Mixtures Expert Task Group (ETG) has recommended revisions to AASHTO PP2 to permit two hour short term oven aging at 135°C for low absorption asphalt mixtures rather than the standard practice of four hour aging. Since the previous experiment indicates that aging temperature could and should be changed, the effect of aging time on mixture properties may also change. In this experiment, two low absorption aggregate blends were tested (one aggregate type and two gradations) with two asphalt binders to determine if the alternate aging procedure of two hours at the mixture's compaction temperature would produce similar results to the standard aging procedure (4 hours at 135°C).

The analysis indicates that the interaction of aging time and temperature had a significant effect on the percentage of air voids in 50 percent of the comparisons. In both cases when a significant difference was indicated, the shorter aging time/higher temperature condition (2 hours at the mixture's compaction temperature) resulted in a higher percentage of air voids at N_{design} than the longer aging time/lower temperature condition (4 hours at 135°C). This trend is opposite of the expected trend. However, there was only 0.2 percent difference in the percentage of air voids between two and four hour aging time periods. This limited analysis would support the null hypothesis (Ho) that performing short term oven aging of asphalt mixtures for two hours at the mixture's compaction temperature rather than four hours at 135°C will not significantly affect the asphalt mixture's volumetric and densification properties.

4.4 TASK 6B: EVALUATION OF THE DEPTH OF MIXTURE ON THE REQUIRED NUMBER OF GYRATIONS

4.4.1 Test Results and Discussion

The first step of the experiment consisted of compacting mixtures using a higher vertical pressure than the Superpave protocol. In Superpave, specimens are compacted to N_{design} gyrations using a pressure of 600 kPa. The original N_{design} experiment performed during SHRP was performed on mixtures from the top 100 mm of a pavement. The vertical pressure of 600 kPa is typical of the pressure at 50 mm depth, the mid-point of the mixtures that were analyzed.

Generally truck tire contact pressures are expected to be higher than the 600 kPa pressure used in Superpave. At the surface a pressure of 827 kPa was selected as a representative pressure. A densification curve was obtained using 827 kPa as the vertical pressure instead of 600 kPa used in the standard Superpave protocol.

The mixtures evaluated in this task consisted of the coarse graded (Alabama limestone, Georgia granite, and New York Gravel) mixtures from Task 6A. Mix design properties for these mixtures can be found in Table 4.16 to 4.18. All mixture testing in this task was accomplished by compacting the mixtures at the optimum asphalt content determined from Task 6A.

Triplicate specimens were compacted to 30, 60, 90 and 120 gyrations and the density was determined. The densification curve was obtained without using the back calculation procedure employed in Superpave. These results are shown in Table 4.46.

A plot of percent maximum theoretical density (%G_{mm}) versus the log of gyrations is shown in Figure 4.11. For each mixture a linear regression analysis was performed to estimate the linear densification curve. The regression equation and correlation coefficient for each mixture is shown in Figure 4.11. Using the regression equation the number of gyrations that gave 96% G_{mm} was estimated. The number of gyrations for 96% G_{mm} is referred to as N_{827}. Values of N_{827} are listed in Table 4.47.

The second step of the experiment consisted of compacting mixtures using different vertical compaction pressures. Compaction was performed to the number of gyrations that yielded 96% G_{mm} using a vertical pressure of 827 kPa as listed in Table 4.47. Specimens were compacted using 827, 689, 551, and 414 kPa. "Corrected" densification curves were not measured using techniques similar to the standard Superpave gyratory compaction procedure. Instead, the density of specimens was

Table 4.46 Average Density of Specimens Compacted at a Vertical Pressure of 827 kPa

	Georgia Granite		New York Gravel		Alabama Limestone	
Gyrations	Density	% G_{mm}	Density	% G_{mm}	Density	% G_{mm}
30	-	-	-	-	2.394	93.7
60	2.408	94.9	2.322	94.9	2.464	96.5
90	2.439	96.2	2.335	95.4	2.499	97.8
120	2.447	96.5	2.352	96.1	2.510	98.3

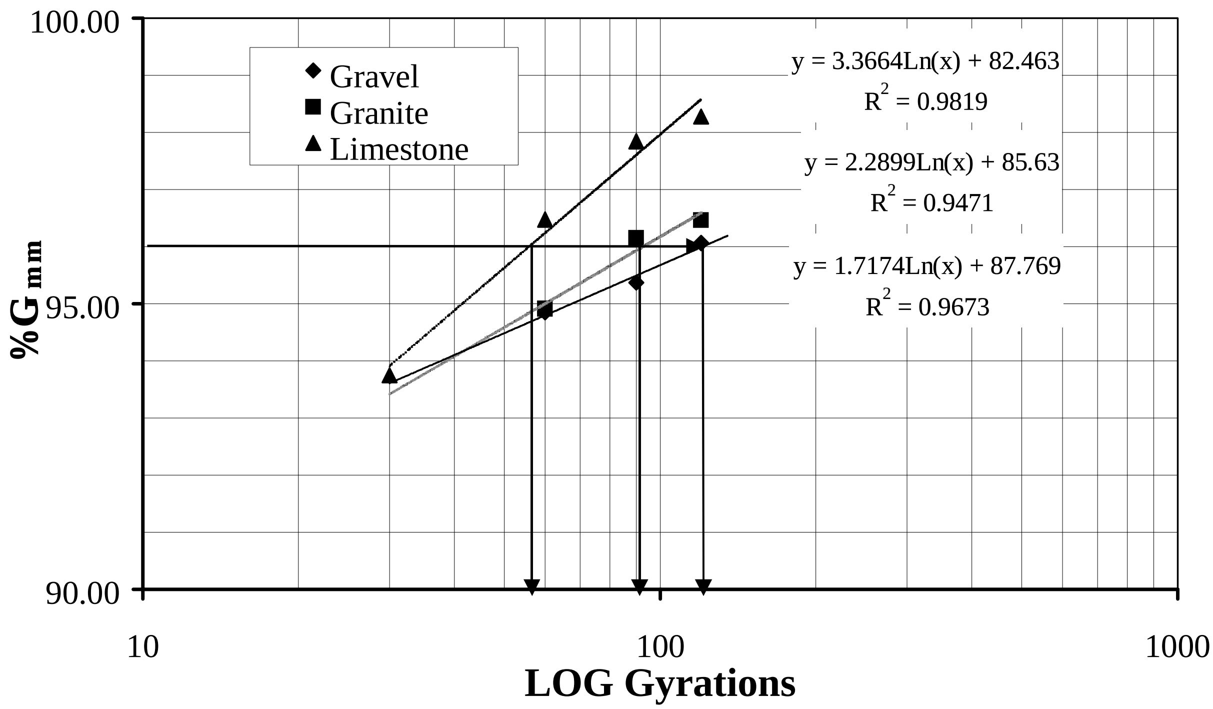

Figure 4.11 Densification Curves for Mixtures Compacted Using 827 kPa Vertical Pressure

Table 4.47 Gyrations Required to Achieve 96 Percent G_{mm} Using a Vertical Pressure of 827 kPa

Aggregate Type	Gyrations Required to Achieve 96% G_{mm}
Georgia Granite	93
New York Gravel	120
Alabama Limestone	56

measured at the end of compaction. Density achieved for each vertical compaction pressure is listed in Table 4.48.

The third step of the experiment consisted of compacting each aggregate mixture using the standard Superpave compaction protocol. The specimens were compacted to N_{max} as normally done during Superpave mix design and the densification curve was corrected using the measured G_{mb}. A regression analysis was done for the densification curve of each mixture. This regression was used to determine the number of gyrations needed to match the density listed in Table 4.48.

Table 4.48 Density After Compaction to N827 Using Different Vertical Pressures

Vertical Pressure (kPa)	Aggregate Type		
	Georgia Granite (93 Gyrations)	New York Gravel (120 Gyrations)	Alabama Limestone (56 Gyrations)
827	96.8	96.1	96.8
689	96.5	95.5	96.0
551	95.9	93.9	94.6
414	94.6	93.4	93.4

An example compaction curve is shown in Figure 4.12 for the gravel mixture. The number of regular Superpave gyrations to match the density obtained for the various vertical compaction pressures are listed in Table 4.49.

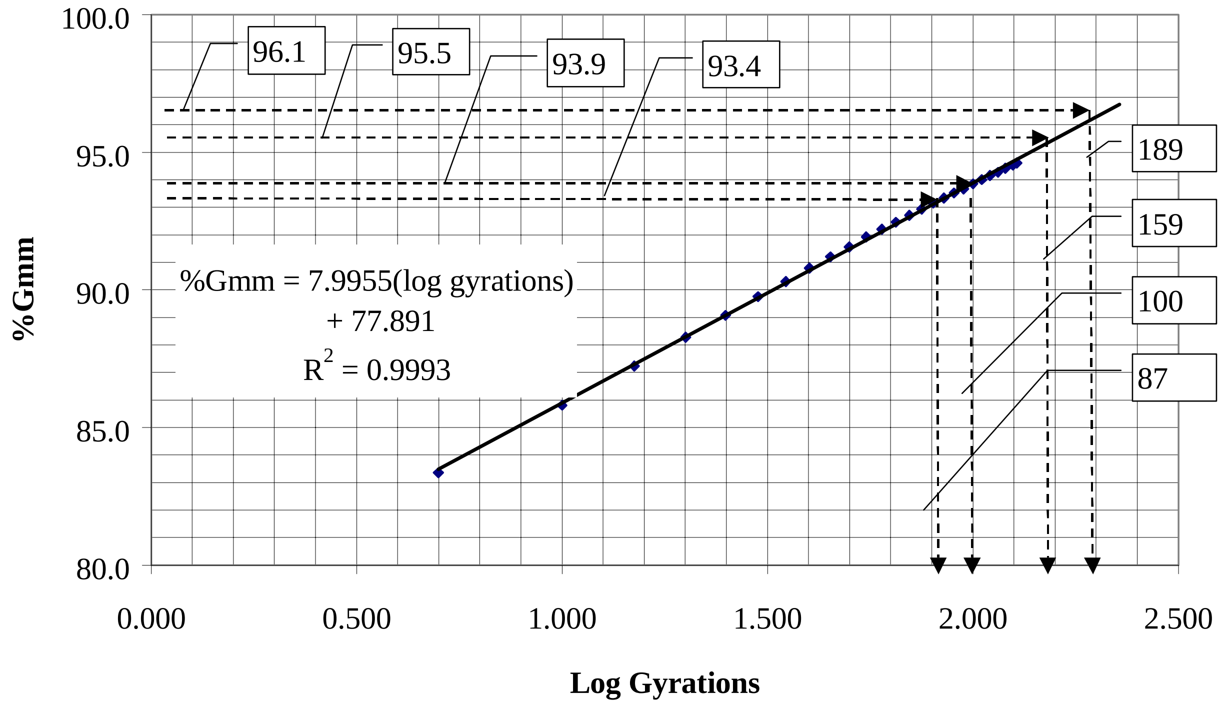

Figure 4.12 Densification Curves for Gravel Mixture Compacted Using Standard Superpave Protocol

Table 4.49 Gyrations Required at 600 kPa to Achieve Density Obtained with Various Vertical Pressures and Gyrations Indicated in Table 4.48

Vertical Pressure (kPa)	Aggregate Type		
	Georgia Granite	New York Gravel	Alabama Limestone
827	142	189	139
689	130	159	115
551	110	101	83
414	77	87	62

The fourth step of the experiment was to plot the vertical compaction pressure and the percent of gyrations at 827 kPa required to match density at that pressure. Each of the mixtures had been compacted to a different number of gyrations as listed in Table 4.47. The desire was to evaluate the various mixtures to determine if each mixture responded in the same way. When using a vertical compaction pressure of 827 kPa, the number of gyrations required to obtain 96% G_{mm} was different for each of the mixtures. Hence, the number of gyrations at a lower pressure (600 kPa) would likewise be expected to be different for the various mixtures. As a result, for each mixture it was necessary to normalize the number of gyrations for each pressure to the number of gyrations for 827 kPa.

Table 4.50 contains a summary of data and the percent gyrations for all three mixtures evaluated.

Table 4.50 Average Density of Specimens Compacted at 827 kPa

Aggregate	Vertical Pressure (kPa)	% G_{mm}	N600	Ratio
New York Gravel	827 (120 Gyrations)	96.1	189	100 (% of 189)
	689 "	95.5	159	84 "
	551 "	93.9	100	53 "
	414 "	93.4	87	45 "
Georgia Granite	827 (93 Gyrations)	96.8	141	100 (% of 141)
	689 "	96.5	130	92 "
	551 "	95.9	110	78 "
	414 "	94.6	77	54 "
Alabama Limestone	827 (56 Gyrations)	96.8	138	100 (% of 138)
	689 "	96.0	114	83 "
	551 "	94.6	83	59 "
	414 "	93.4	62	45 "

A plot of the normalized data and linear regression lines for each mixture is shown in Figure 4.13. The gravel and the limestone mixture behave very similarly. The granite mix plots separately from the other two, but the results are similar.

The fifth step of the experiment was to estimate the change in vertical stress with depth. Several programs are available that use linear elastic response to estimate stress and strain. Two programs were used. DAMA from the Asphalt Institute was used to investigate various structure types and properties of the asphalt pavement layers. JULEA, a layered elastic analysis program from the Federal Aviation Administration, was used to check the reasonableness of DAMA. Output from DAMA and JULEA, as well as other task information are provided in Appendix D.

Two full depth HMA cross sections were defined, one 267 mm thick and the other 318 mm thick. Both structures were thick enough to have asphalt mixtures below the 100 mm depth. These two thicknesses were selected to see if the thickness influenced predicted stress results. Predicted

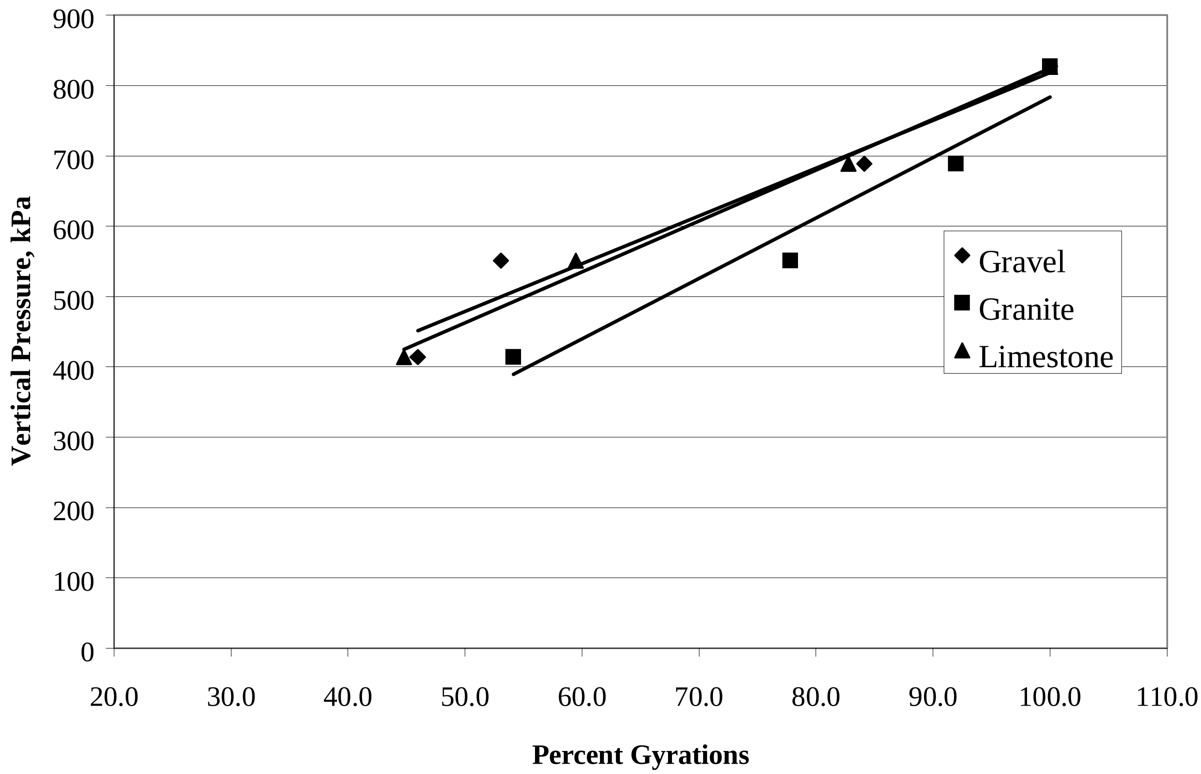

Figure 4.13 Compaction Required to Achieve Density Using Different Vertical Pressures (Normalized to 827 kPa)

vertical stress is shown in Figure 4.14. Little difference exists in predicted stress for either of the two pavements.

Figure 4.15 shows a comparison of stresses predicted for a full depth hot mix asphalt (FDHMA) pavement and a 200 mm overlay of hot mix asphalt on Portland cement concrete (HMA/PCC). Vertical stress in the asphalt layers is higher when the underlying layer is HMA. At 100 mm, which is the cutoff of the original design gyration experiment, the difference is approximately 90 kPa. For the purpose of this experiment the pressure distribution in the FDHMA was used since it appears to be the most critical.

The sixth step of the experiment was to determine the theoretical reduction in required design gyrations with depth into the pavement. To this point two relationships have been developed. One is the change in gyrations required with change in vertical pressure. The second is the change in vertical pressure that occurs with depth. Using vertical pressure as the common variable, a relationship of required gyrations with depth was obtained.

Figure 4.16 shows the relationship of compaction pressure and the number of standard (600 kPa) Superpave gyrations to match the density when compacted to N_{827} gyrations. When the test results are not normalized, as they were in Figure 4.13, each mixture has a similar but different relationship. In Figure 4.17, the data from all three mixtures was combined into one data set and a regression analysis was performed. The results are used to tie depth to gyrations.

Now two relationships have been developed. The first is a relationship of vertical pressure to depth. The equation as follows is valid within the range of 0 to 300 mm depth:

$$P = -4.5x10^{-7}d^{4} + 3.35x10^{-4}d^{3} - 7.28x10^{-2}d^{2} + 1.37d + 828 \qquad \textit{Equation 1}$$

where P = *vertical pressure, kPa*
d = *depth, mm*

The second relationship indicates the number of gyrations at 600 kPa in the standard Superpave compaction protocol that are required to match density if a different vertical pressure is used in the gyratory.

$$P = 18.385G^{0.7407} \qquad \textit{Equation 2}$$

where P = *vertical pressure, kPa*
G = *gyrations using 600 kPa vertical compaction pressure*

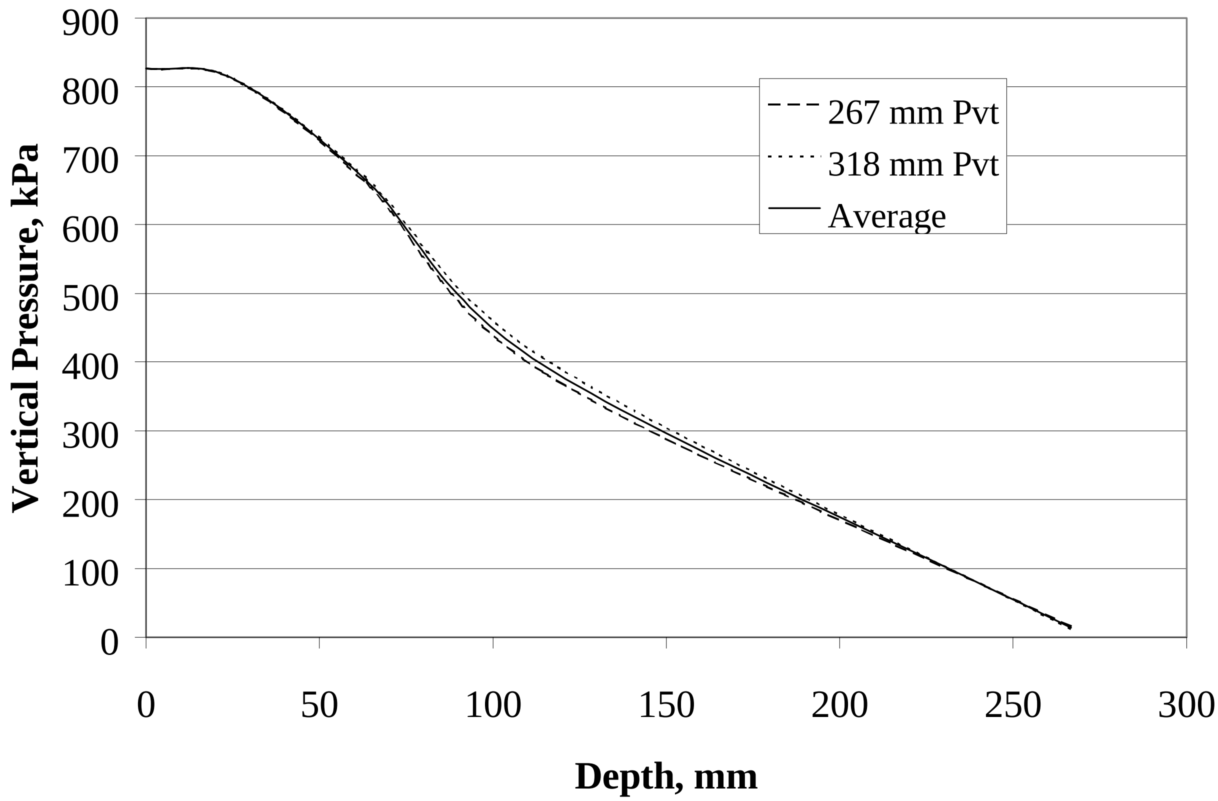

Figure 4.14 Comparison of Stress Predicted for Different Pavement Thicknesses

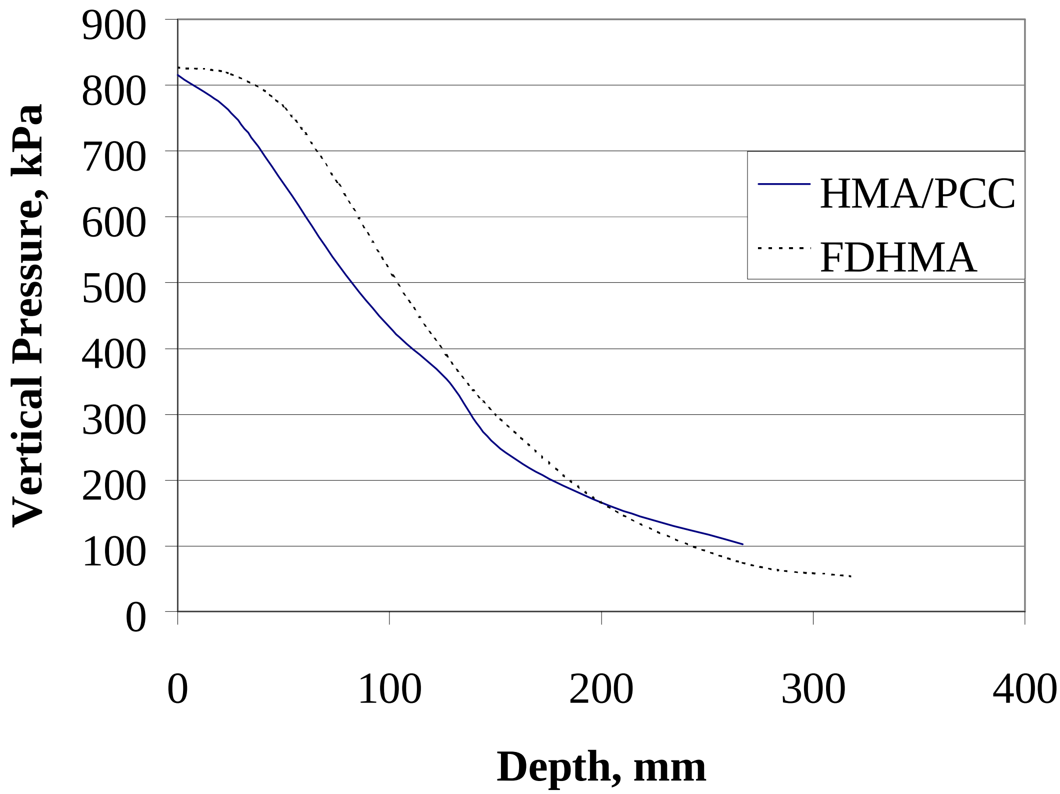

Figure 4.15 Comparison of Stress Predicted for Full Depth Hot Mix Asphalt (FDHMA) and Hot Mix Asphalt Overlay of Portland Cement Concrete (HMA/PCC)

Using these two relationships, a table of gyrations and depth can be determined as shown in Table 4.51. The reasonableness of the results should be discussed. The original mixture design was performed using 128 gyrations for N_{design}. At the surface, the number of standard Superpave gyrations necessary to match the density when compacted using 827 kPa was 171. At 75 mm, 123 gyrations were needed, which is close to the N_{design} used in the mixture design. At increased depth, the gyrations needed to match density obtained with the lower compaction pressures were quite low, 44 gyrations at 150 mm depth where the vertical pressure is 298 kPa and 9 gyrations at 250 mm depth where the vertical pressure is 97 kPa.

The hypothesis of this experiment is that vertical pressure caused by traffic loading will influence the compaction effort and hence final density of asphalt mixture. Pressure is the link used to join depth from the surface with gyrations in the gyratory compactor. By itself, the study of pressure versus the number of gyrations is reasonable. As vertical pressure in the gyratory decreases, the number of standard gyrations needed to obtain the same density decreases. Likewise, the decrease in vertical pressure with depth seems reasonable. As depth increases, the vertical pressure drops. Parametric evaluation of different asphalt layer thickness and pavement structure type showed similar predictions of vertical pressure.

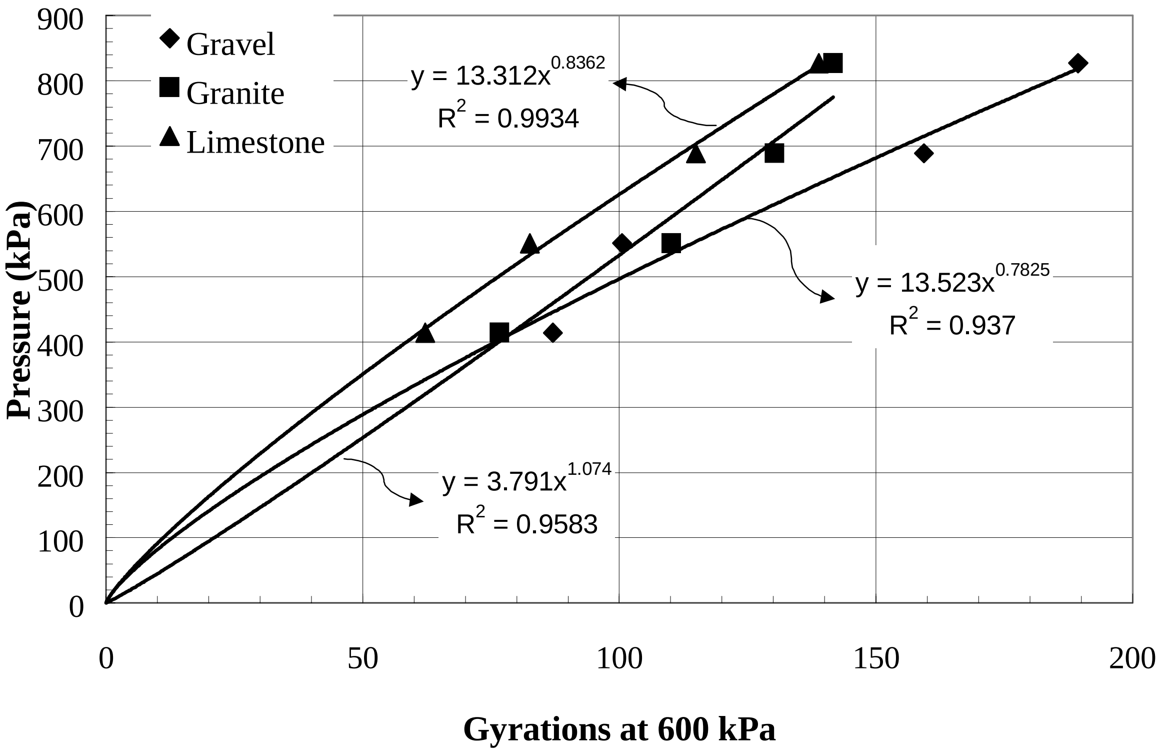

Figure 4.16 Relationship of Compaction Pressure and Number of Standard (600 kPa) Superpave Gyrations (Individual Mixtures)

Perhaps vertical pressure under a truck wheel cannot be equated to vertical pressure in the gyratory compactor. A truck tire generates shear stress in a pavement by the vertical load. Shear stress in the mix is in response to the amount of vertical load applied. Shear strain in the pavement is dependent on the amount of shear stress applied and for a given shear stress the shear strain depends on the mixture shear stiffness.

In the gyratory compactor, shear strain does not depend on mixture stiffness or vertical load. Shear strain is kept constant by the controlled angle. Vertical pressure in the compactor may influence shear stress; that is, the higher the vertical pressure, the more stress is required to gyrate the specimen. However, vertical stress does not influence shear strain.

The role of vertical pressure in the gyratory compactor is different from under a truck tire. In both cases, vertical pressure helps with compaction. In the gyratory compactor vertical pressure has no influence on shear strain. Under a truck tire, shear strain is a response to vertical pressure. Shear strain controls mixture compaction. For example, if pressure is applied in the gyratory compactor but no gyrations occur, there will be very little compaction. Compaction really starts as soon as the gyrations start, as soon as shear strain is induced.

The data in Table 4.51 can be used to establish the number of gyrations for various traffic levels and for various depths into the pavement structure. From a densification standpoint, the number of gyrations can be reduced to 72 percent of the design gyrations at 100 mm and to 13 percent at 200 mm. However, the mixture strength must still be sufficient to support the applied load or vertical stress. If the gyrations are reduced too much, the mixture strength will not be satisfactory. Also, the layers to be placed greater than 100 mm in the pavement structure will support traffic for a period of time prior to being covered by other mixtures.

Based on these results it is reasonable to decrease the number of gyrations at 100 mm and even more at 200 mm and lower. For best results, it is recommended that the gyration level be decreased one level at depths from 100 mm to 200 mm. The gyration level can be dropped two levels for mixtures placed 200 mm or greater from the finished pavement surface. Again, the user must be cautioned about construction traffic and other traffic prior to the underlying mixtures being covered.

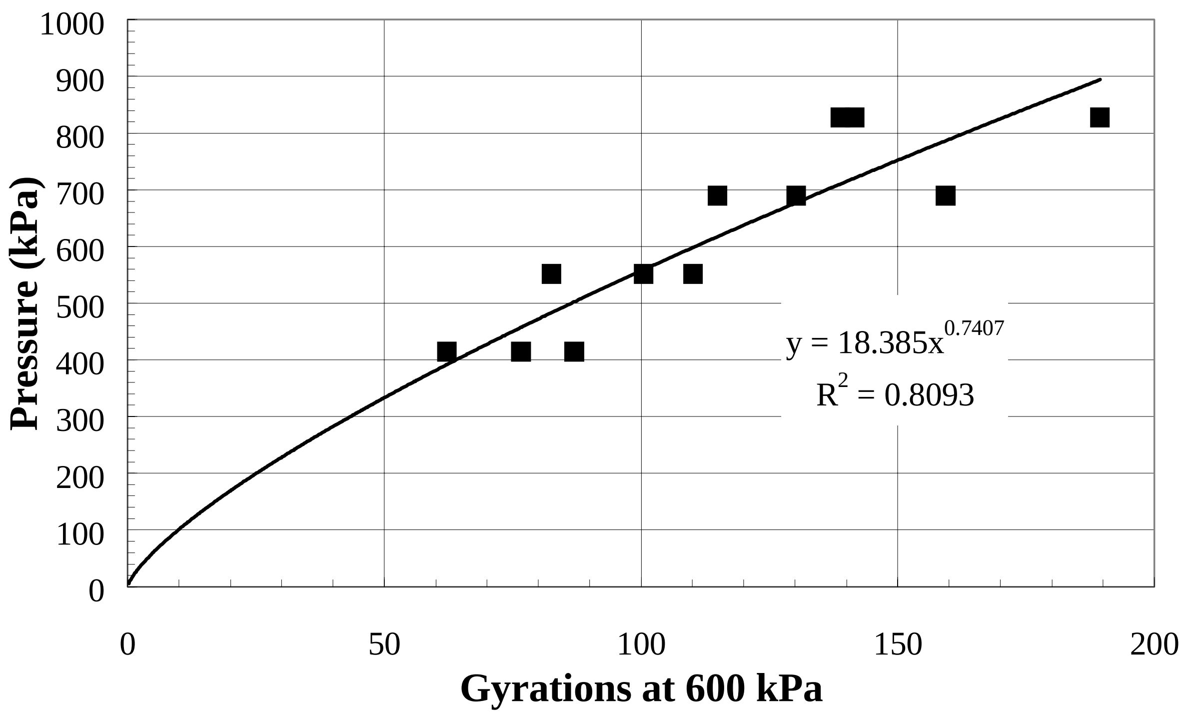

Figure 4.17 Relationship of Compaction Pressure to Number of Standard (600 kPa) Superpave Gyrations (Average of All Mixtures)

Table 4.51 Table of Standard Gyrations versus Depth From Surface

Depth from Surface (mm)	Vertical[1] Pressure (kPa)	Standard Gyrations	Arithmetic Ratio
Design	-	128	1.00
0	827	171	1.33
25	822	169	1.32
50	754	150	1.17
75	648	123	0.96
100	527	93	0.72
125	406	65	0.51
150	298	43	0.34
175	212	27	0.21
200	150	17	0.13
250	97	9	0.07
300	87	8	0.06

Notes: (1) Vertical Pressures as Determined from JULEA layer elastic program.

4.5 TASK 6C: CONSOLIDATION OF THE N_{design} COMPACTION MATRIX AND EVALUATION OF THE $N_{maximum}$ REQUIREMENT

4.5.1 Material Properties

Materials used in this task of the study were the same as those used for Task 4. Properties of the coarse and fine aggregates can be found in Tables 4.1 and 4.2, respectively. The aggregates were chosen for use in this task because they exhibited a wide range of physical properties, such as absorption, flat and elongated particles, Los Angeles abrasion, etc.; which should better provide an indication of performance of various aggregate types that are currently used in the Superpave mix design.

4.5.2 Test Results

Test results are provided in this section of the report. The results are provided as they relate to the following evaluations:

- Evaluation of the Response Variables
- N_{design} Compaction Matrix Evaluation (Consolidation of N_{design} Table)
- Gyratory Compaction Slope Evaluation
- $N_{initial}$ Compaction Requirement Evaluation
- $N_{maximum}$ Compaction Requirement Evaluation

4.5.2.1 Evaluation of the Response Variables

Task 6C was designed, in part, to determine if significant differences exist between the various levels of N_{design} which are currently specified in the Superpave mix design procedure. This effort was basically a sensitivity study to determine and evaluate the effects of varying the compaction level (N_{design}), aggregate type, and gradation on volumetric properties. The first step in the task was to perform mix designs for each combination of N_{design}, aggregate type, and gradation (48 total mix designs: 6 N_{design} levels * 4 aggregates * 2 gradations). Three point mix designs were conducted by varying the asphalt content in 0.5 percent increments and bracketing the optimum asphalt content. Optimum asphalt content, in all cases, was chosen to provide approximately four percent air voids at N_{design}. Some designs did not meet all of the Superpave volumetric requirements; however, this was not of major concern since the task was designed to determine the sensitivity of the volumetric properties to changes in mixture and compaction parameters.

Detailed information for all mix designs is provided in Appendix E. The mixing and compaction temperatures of the mixtures were determined, through rotational viscometer testing, to be 155°C and 147°C, respectively. After mixing, all specimens were short term aged at 135°C for four hours, in accordance with AASHTO PP2, and then placed in a separate oven to reach compaction temperature. The Pine gyratory compactor was used for the compaction of all mixtures in this task. Mix designs were performed by compacting the mixtures to N_{design}, not $N_{max,}$ in order to obtain the true volumetric properties of the mixtures at N_{design}, not properties determined through the back-calculation procedure from N_{max}.

At the chosen optimum asphalt content, the response variables of %G_{mm} at $N_{initial}$, compaction slope, VMA, and VFA were then determined. Values of optimum asphalt content and the response variables are shown in Table 4.52, and Figures 4.18 through 4.27. As seen in Table 4.52 and the various figures, the optimum asphalt content and response variables decreased to some degree with an increase in the N_{design} level or compactive effort. This was expected with the properties of VMA and VFA, since they are a function of the mixture's asphalt content, which decreased with an increased compactive effort.

The fact that the compaction slope (measured from $N_{initial}$ to N_{design}), shown in Table 4.52 and Figures 4.24 and 4.25, decreased with an increase in compactive effort is contrary to the belief that a mixture should have a higher slope at increased N_{design} levels, which correlates to increased temperatures and traffic. A stronger aggregate structure is needed at higher traffic levels to resist permanent deformation. Therefore, if compaction slope is an indication of the strength of the aggregate structure, the results indicate that mixtures at lower levels of N_{design} have a stronger aggregate structure and would resist permanent deformation to a greater extent than mixtures designed at higher N_{design} levels. This is obviously not the case.

Values of percent G_{mm} at $N_{initial}$, shown in Table 4.52 and Figures 4.26 and 4.27, generally show a trend of being less at increased gyration levels. This would indicate that mixtures designed for lower traffic volumes may experience greater difficulties in achieving the current minimum specified density of 89 percent at $N_{initial}$, than would mixtures designed for high traffic volume roadways.

Another observation is that the slope of the compaction curve shown in Table 4.52 and Figures 4.24 and 4.25 is generally steeper for coarse graded mixtures than for the fine graded mixtures. This indicates that selection of the proper compaction level is more critical for coarse graded mixtures. It may also be an indication that the coarse graded mixtures are less sensitive to changes in asphalt content. For example a 0.4 percent increase in asphalt content may move a fine graded mixture from a very high traffic mixture to a very low traffic mixture, but it will have less of an effect on the coarse graded mixtures.

Table 4.52 and Figures 4.26 and 4.27 show that the %G_{mm} at $N_{initial}$ is always higher for fine graded mixtures than for coarse graded mixtures. Some of the fine graded mixtures do not meet the requirements for 89 maximum density (%G_{mm}) at $N_{initial}$. All of the coarse graded mixtures meet this requirement without any problem.

In order to determine the effect of the various factors in the task, a multivariate analysis of variance was conducted. The analysis was conducted for the variables of VMA, compaction slope, and %G_{mm} at $N_{initial}$ at a level of significance (α = 5 percent or 0.05). Main effects and interactions were evaluated in the analysis, with the results provided in Tables 4.53- 4.55. Complete statistical output is provided in Appendix F. The results indicate that for the VMA and compaction slope, all main factors and interactions are statistically significant. However, the analysis of the %G_{mm} at $N_{initial}$ test results showed that the two-way interaction of N_{design}*gradation and the three-way interaction of N_{design}* aggregate*gradation were not statistically significant at the same level of significance.

Additionally, in an effort to determine the magnitude of differences between the project treatments, a Duncan's multiple range comparison procedure, at a level of significance of 5 percent, was conducted for the variable VMA on the levels of N_{design} with respect to gradation,

Table 4.52 Summary of Response Variables and Optimum Asphalt Contents for Task 6C

Aggregate	Gradation	Response Variables	N_{design} Level					
			40	68	93	113	139	172
New York Gravel	Fine	OAC	5.3	5.0	4.8	4.7	4.6	4.4
		% G_{mm} N_i	90.4	90.0	89.8	89.6	89.5	89.4
		Slope	6.58	6.15	5.88	5.71	5.57	5.28
		VMA	14.4	13.6	13.1	12.9	12.6	12.3
		VFA	71.8	70.8	70.1	70.0	69.4	67.0
	Coarse	OAC	6.2	5.6	4.8	4.7	4.4	4.1
		% G_{mm} N_i	87.0	87.0	86.7	86.0	86.2	86.1
		Slope	9.90	9.16	8.79	8.63	8.23	7.92
		VMA	16.3	15.0	13.3	13.2	12.5	12.0
		VFA	75.5	73.9	70.4	69.2	68.1	65.9
Georgia Granite	Fine	OAC	5.5	5.1	4.8	4.7	4.7	4.5
		% G_{mm} N_i	90.1	90.1	89.9	89.7	89.7	89.7
		Slope	6.54	5.96	5.71	5.54	5.40	5.18
		VMA	16.7	15.9	15.4	15.1	14.9	14.6
		VFA	75.8	75.5	73.6	73.5	73.4	72.9
	Coarse	OAC	5.0	4.4	4.3	4.2	4.1	3.9
		% G_{mm} N_i	86.2	86.5	86.5	86.8	86.8	86.9
		Slope	9.25	8.29	8.25	7.88	7.59	7.29
		VMA	15.9	14.8	14.1	14.1	13.9	13.5
		VFA	74.6	73.0	72.7	71.0	71.0	69.4
Alabama Limestone	Fine	OAC	4.5	4.3	3.9	3.8	3.7	3.6
		% G_{mm} N_i	88.7	88.6	88.3	88.2	88.0	87.8
		Slope	8.02	7.56	7.19	6.69	6.30	5.93
		VMA	13.3	12.7	11.9	11.5	11.5	11.4
		VFA	69.9	69.2	68.2	66.8	64.5	63.4
	Coarse	OAC	4.9	4.2	3.9	3.6	3.4	3.2
		% G_{mm} N_i	86.2	85.9	85.8	85.6	85.5	85.3
		Slope	10.89	10.27	9.86	9.71	9.30	8.86
		VMA	14.6	13.0	12.5	11.7	11.3	10.7
		VFA	72.7	69.6	67.8	66.5	65.1	65.0
Nevada Gravel	Fine	OAC	6.2	5.6	5.6	5.6	5.3	5.2
		% G_{mm} N_i	89.1	88.9	88.3	88.0	88.3	87.9
		Slope	7.80	7.23	7.06	6.92	6.69	6.42
		VMA	14.8	13.9	13.8	13.7	12.8	12.8
		VFA	74.0	71.2	71.0	70.8	70.6	67.8
	Coarse	OAC	7.4	6.3	5.7	5.5	5.4	5.4
		% G_{mm} N_i	87.2	87.0	86.6	86.0	86.2	85.8
		Slope	9.80	9.36	8.87	8.85	8.36	8.23
		VMA	18.0	15.5	14.3	13.8	13.6	13.6
		VFA	77.8	75.6	72.8	72.6	71.8	70.8

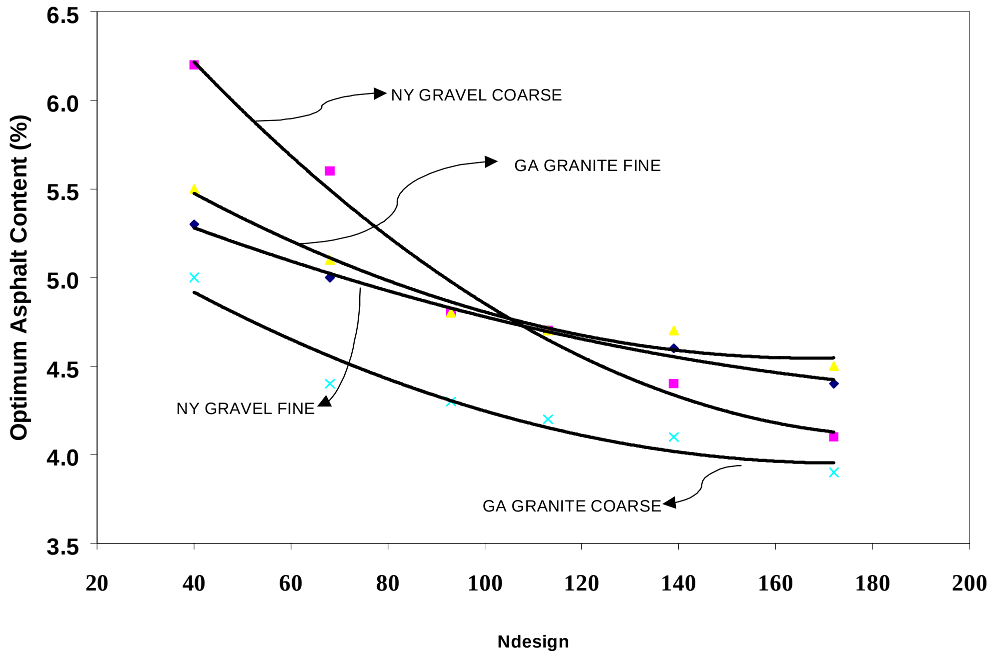

Figure 4.18 Optimum Asphalt Content versus N_{design} - New York Gravel and Georgia Granite Aggregates

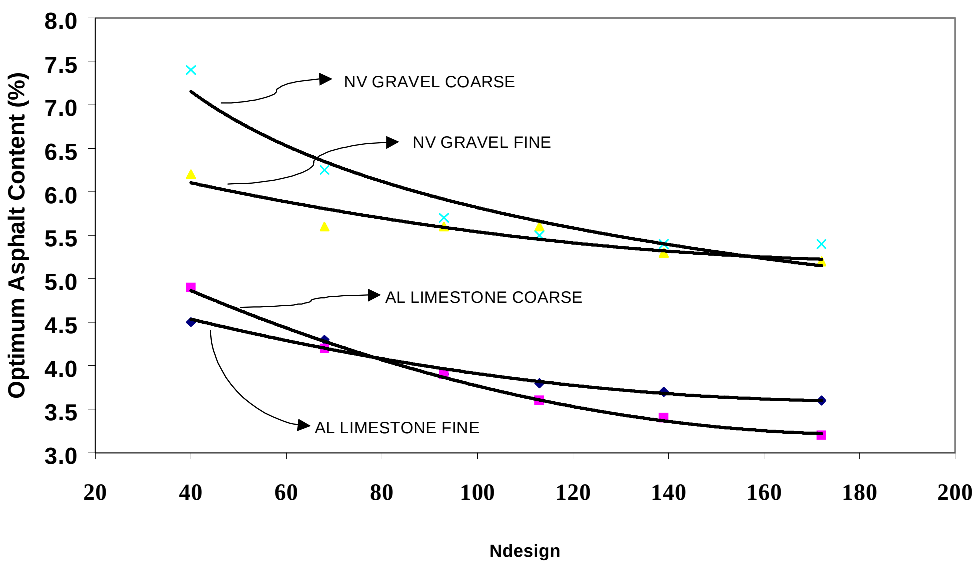

Figure 4.19 Optimum Asphalt Content versus N_{design} - Alabama Limestone and Nevada Gravel Aggregates

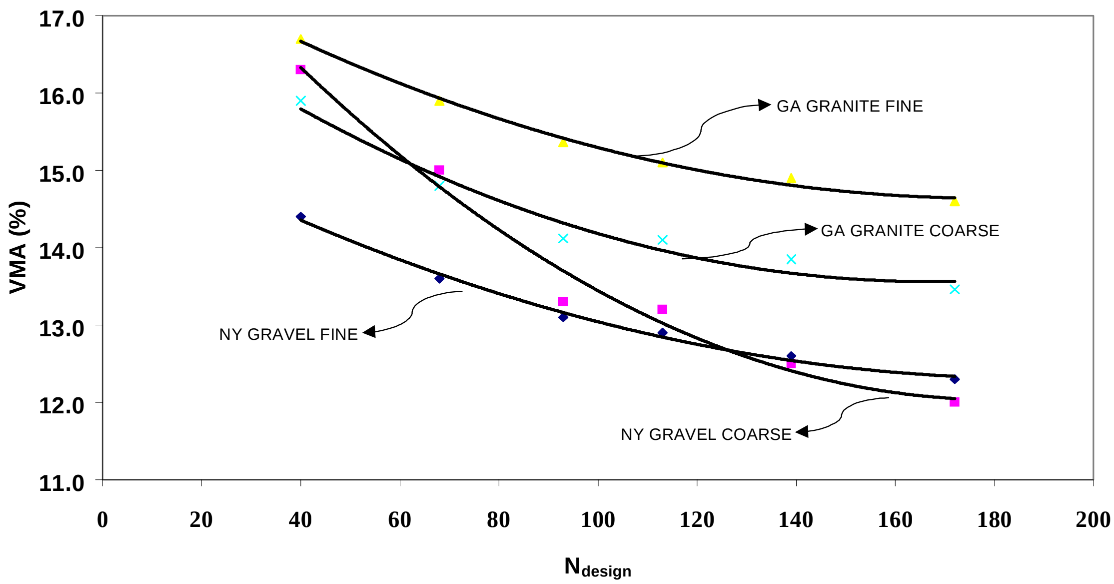

Figure 4.20 VMA versus N_{design}- New York Gravel and Georgia Granite Aggregates

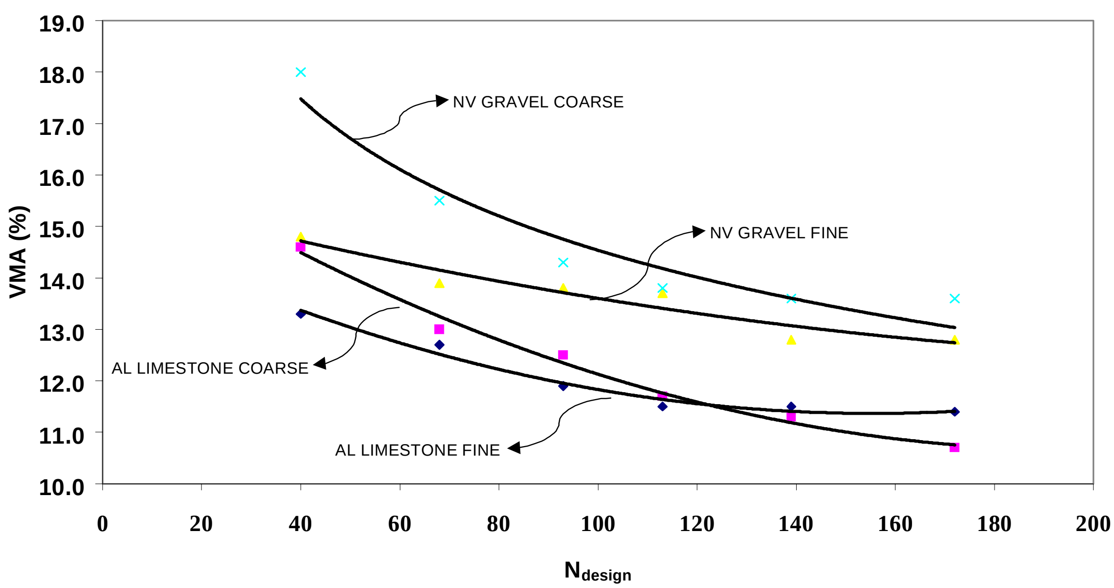

Figure 4.21 VMA versus N_{design}- Alabama Limestone and Nevada Gravel Aggregates

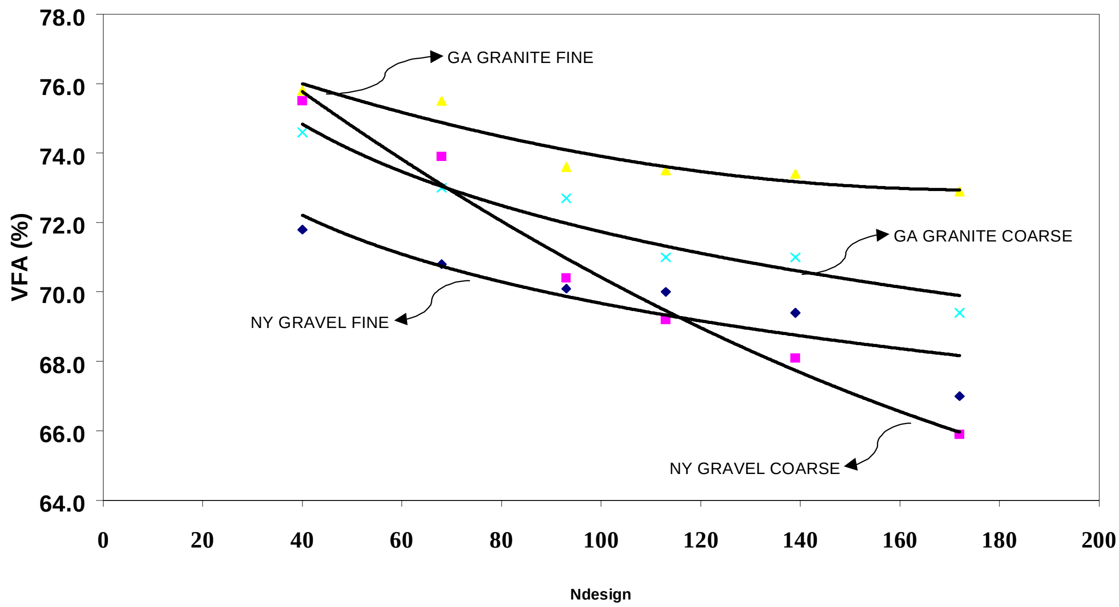

Figure 4.22 VFA versus N_{design}- New York Gravel and Georgia Granite Aggregates

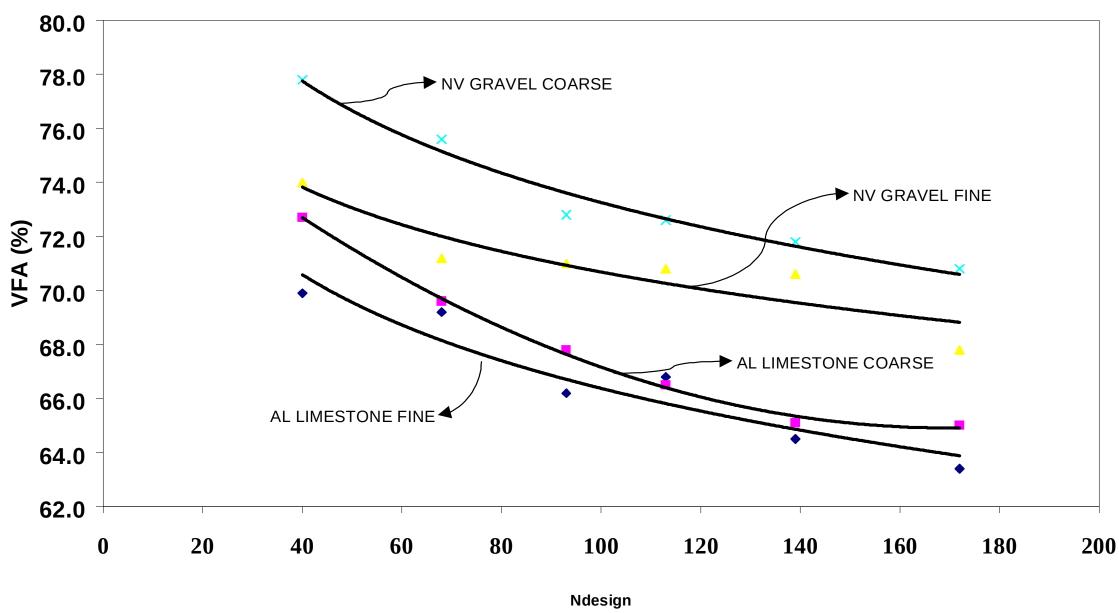

Figure 4.23 VFA versus N_{design}- Alabama Limestone and Nevada Gravel Aggregates

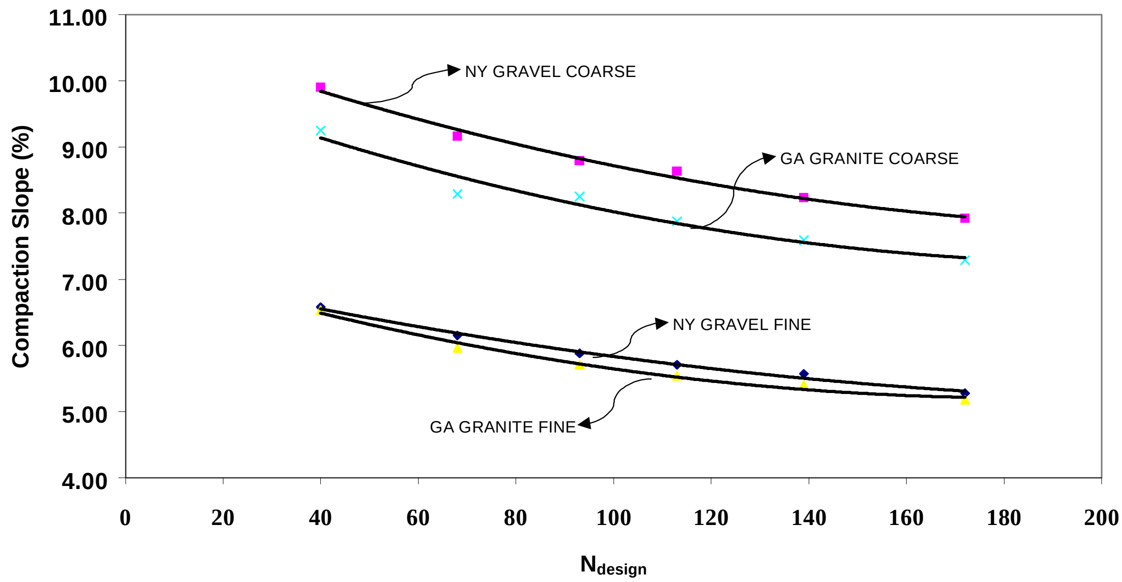

Figure 4.24 Compaction Slope versus N_{design}- New York Gravel and Georgia Granite Aggregates

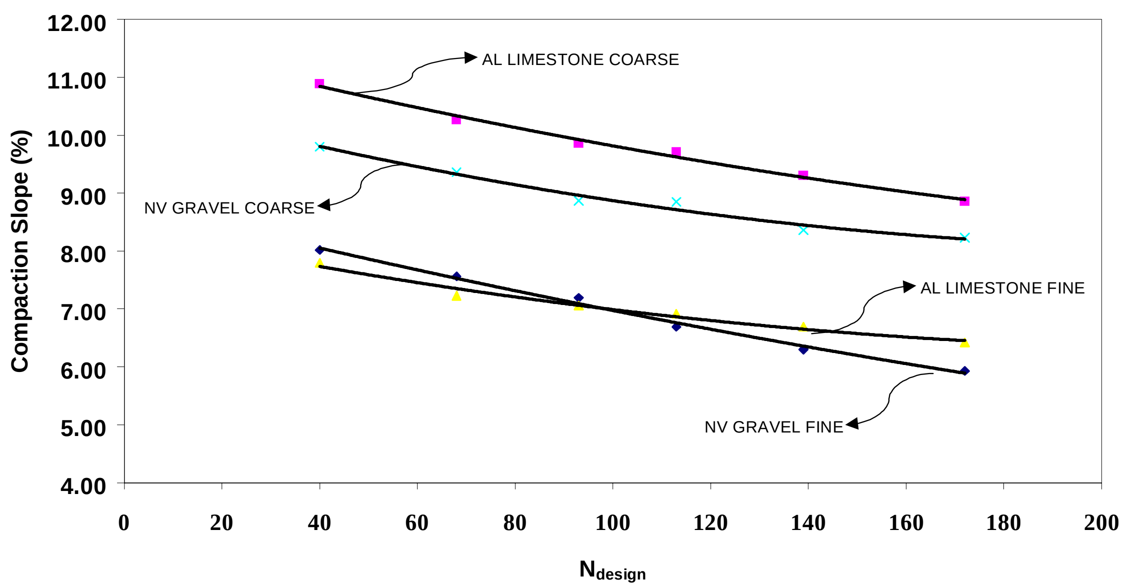

Figure 4.25 Compaction Slope versus N_{design}- Alabama Limestone and Nevada Gravel Aggregates

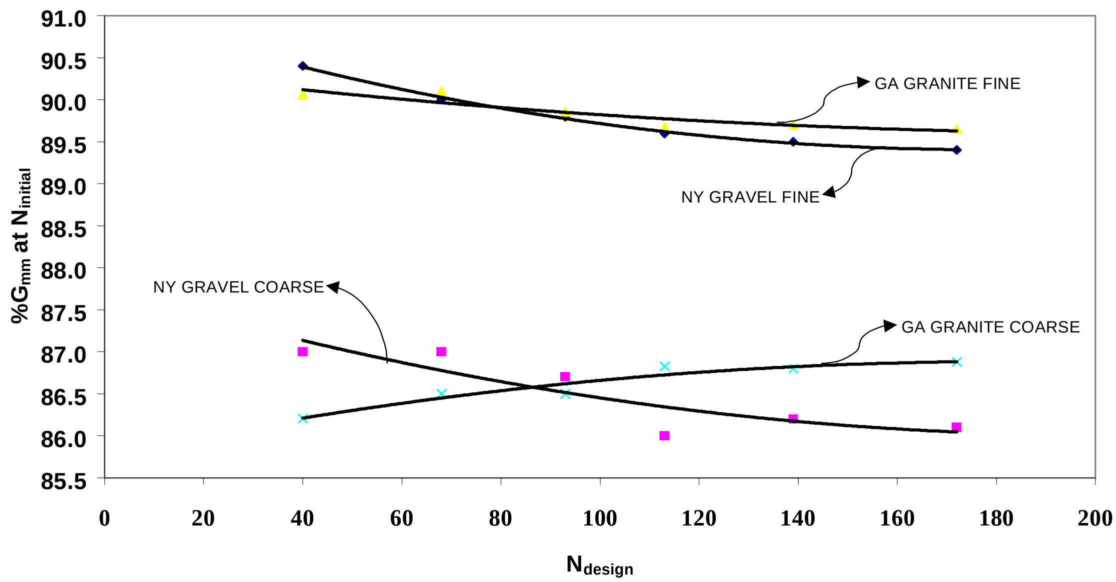

Figure 4.26 $\%G_{mm}$ at $N_{initial}$ versus N_{design}- New York Gravel and Georgia Granite Aggregates

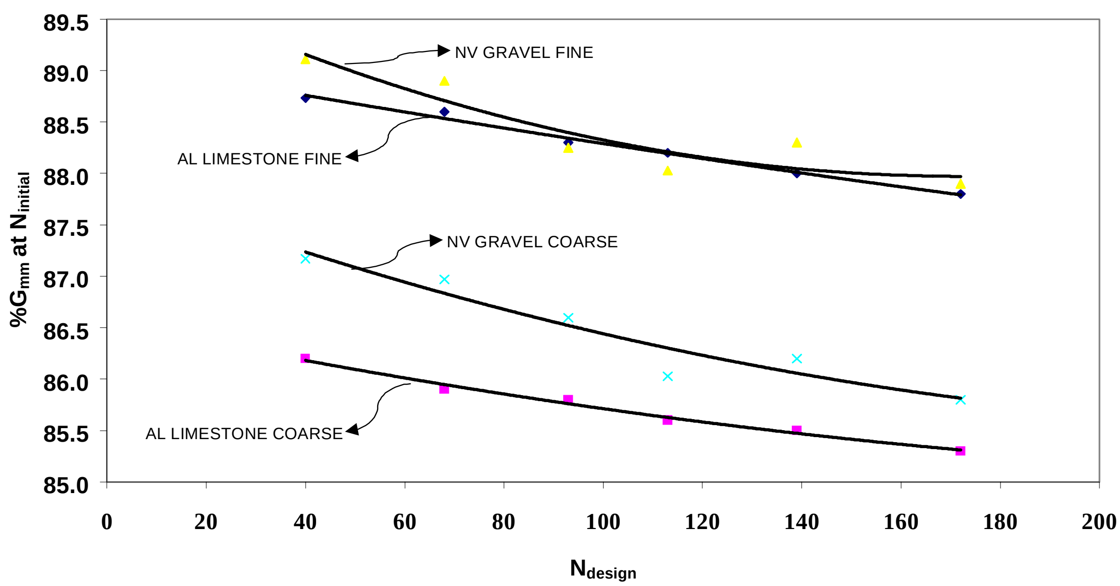

Figure 4.27 $\%G_{mm}$ at $N_{initial}$ versus N_{design}- Alabama Limestone and Nevada Gravel Aggregates

Table 4.53 ANOVA of Voids in Mineral Aggregate (VMA) for Task 6C

Variation Source	df	Sum of Squares	Mean Squares	F calc.	F table $\alpha = 0.05$	P > Fcalc	Significant $\alpha = 0.05$
N_{design}	5	145.91	29.18	447.99	2.33	0.0001	YES
Aggregate	3	137.59	45.87	704.11	2.64	0.0001	YES
Gradation	1	0.50	0.50	7.70	3.88	0.0066	YES
N_{design}*Aggregate	15	4.46	0.30	4.57	1.71	0.0001	YES
N_{design}*Gradation	5	14.18	2.84	43.54	2.33	0.0001	YES
Aggregate*Gradation	3	17.15	5.71	87.77	2.64	0.0001	YES
N_{design}*Aggregate *Gradation	15	6.41	0.43	6.56	1.71	0.0001	YES
Error	96	6.25	0.065	-	-	-	-
TOTAL	143	-	-	-	-	-	-

Table 4.54 ANOVA of Gyratory Compaction Slopes for Task 6C

Variation Source	df	Sum of Squares	Mean Squares	F calc.	F table $\alpha = 0.05$	P > Fcalc	Significant $\alpha = 0.05$
N_{design}	5	42.21	8.44	557.54	2.33	0.0001	YES
Aggregate	3	47.60	15.87	1047.70	2.64	0.0001	YES
Gradation	1	226.78	226.78	14975.74	3.88	0.0001	YES
N_{design}*Aggregate	15	1.34	0.089	5.88	1.71	0.0001	YES
N_{design}*Gradation	5	0.49	0.10	6.44	2.33	0.0001	YES
Aggregate*Gradation	3	6.31	2.10	138.92	2.64	0.0001	YES
N_{design}*Aggregate *Gradation	15	0.53	0.035	2.32	1.71	0.0071	YES
Error	96	1.45	0.015	-	-	-	-
TOTAL	143	-	-	-	-	-	-

Table 4.55 ANOVA of $\%G_{mm}$ at $N_{initial}$ for Task 6C

Variation Source	df	Sum of Squares	Mean Squares	F calc.	F table $\alpha = 0.05$	P > Fcalc	Significant $\alpha = 0.05$
N_{design}	5	8.40	1.68	21.01	2.33	0.0001	YES
Aggregate	3	37.71	12.57	157.23	2.64	0.0001	YES
Gradation	1	340.59	340.59	4259.82	3.88	0.0001	YES
N_{design}*Aggregate	15	3.69	0.25	3.08	1.71	0.0004	YES
N_{design}*Gradation	5	0.30	0.06	0.76	2.33	0.5832	NO
Aggregate*Gradation	3	3.17	1.06	13.22	2.64	0.0001	YES
N_{design}*Aggregate *Gradation	15	1.08	0.07	0.90	1.71	0.5643	NO
Error	96	7.68	0.08	-	-	-	-
TOTAL	143	-	-	-	-	-	-

and not the aggregate type. This resulted in two analyses, coarse and fine, which are provided in Tables 4.56 and 4.57, respectively. As expected, the analysis showed that a significant statistical difference existed between the levels of N_{design}, when evaluated with respect to the coarse and the fine gradation. The range of VMA values across the N_{design} levels evaluated was greater for the coarse gradation than for the fine gradation. Statistical output from the Duncan's analysis is provided in Appendix F.

4.5.2.2 N_{design} Compaction Matrix Evaluation

In considering the possible consolidation of the N_{design} compaction matrix, the ultimate goal was to develop or arrive at compaction levels which were practical and provided acceptable mixture volumetric properties for the designed mixtures. Currently, the compaction matrix, shown in Table 4.58, consists of seven levels of traffic (ESALs) and four levels of average seven day design high air temperature. The lowest air temperature range is less than 39°C, with the temperature of the other levels ranging from 39 to 44°C. Throughout the country, there are only a relatively few locations in which the average seven day design high air temperature exceeds 39°C. Further, it seems logical that increasing the asphalt binder high temperature performance grade or stiffness should account for the increased service temperature of the asphalt mixture; since the asphalt binder's physical properties, viscosity or stiffness, are temperature dependent, and the grade of the asphalt binder is selected in each climate to be equiviscous. To further investigate the change in the volumetric properties between the lowest temperature level (less than 39°C) and the highest temperature level (43-44°C), a student's t-test procedure was used to determine if the change in

Table 4.56 Duncan's Multiple Range Testing Results (N_{design} Levels w/ Respect to Coarse Gradation)

Duncan Grouping[1]	Mean Value	Number of Observations	N_{design} Level
A	16.08	12	40
B	14.55	12	68
C	13.61	12	93
D	13.06	12	113
E	12.55	12	139
F	12.18	12	172

Notes (1) Tested at a level of significance of 5 percent. Means or treatments (N_{design} Levels with respect to coarse gradation) with the same letter are not considered to be statistically significantly different.

Table 4.57 Duncan's Multiple Range Testing Results (N_{design} Levels w/ Respect to Fine Gradation)

Duncan Grouping[1]	Mean Value	Number of Observations	N_{design} Level
A	14.76	12	40
B	14.06	12	68
C	13.58	12	93
D	13.27	12	113
E	12.95	12	139
F	12.68	12	172

Notes (1) Tested at a level of significance of 5 percent. Means or treatments (N_{design} Levels with respect to fine gradation) with the same letter are not considered to be statistically significantly different.

volumetric properties between the two levels was statistically significantly different. A statistical analysis was conducted for all combinations of aggregate type and gradation at comparison N_{design} gyration levels of the following:

68 versus 82, 76 vs 93, 86 vs 105, 96 vs 119, 109 vs 135, 126 vs 153, and 142 vs 172

A total of 56 t-tests were performed (7 N_{design} comparison levels * 4 aggregates * 2 gradations). The response variable of VMA was selected as the dependent variable in the analysis, with the statistical analysis results shown in Table 4.59. Complete statistical analysis information is provided in Appendix F. The results indicate that 41 out of 56 comparisons (73.2 percent) showed no statistically significant difference between the N_{design} levels at a level of significance of 5 percent.

Further, of the 15 comparisons in which the results were statistically significant, the difference in VMA between the treatment means was 0.57 percent or less.

Table 4.58 Superpave Gyratory Compaction Criteria

Design Traffic ESALs (Millions)	7-Day Average Design High Air Temperature											
	Less than 39°C			39°C-40°C			41°C-42°C			43°C-44°C		
	N_i	N_d	N_m	N_i	N_d	N_m	N_i	N_d	N_m	N_i	N_d	N_m
Less than 0.3	7	68	104	7	74	114	7	78	121	7	82	127
0.3 - 1	7	76	117	7	83	129	7	88	138	8	93	146
1 - 3	7	86	134	8	95	150	8	100	158	8	105	167
3 - 10	8	96	152	8	106	169	8	113	181	9	119	192
10 - 30	8	109	174	9	121	195	9	128	208	9	135	220
30 - 100	9	126	204	9	139	228	9	146	240	10	153	253
Greater than 100	9	142	233	10	158	262	10	165	275	10	172	288

Therefore, based upon the testing results, the first step in the consolidation of the current N_{design} compaction matrix would be to eliminate the three levels of design air temperature higher than 39°C. This would yield a matrix which is now consolidated from 28 to 7 levels. With the levels now reduced to seven, the job of consolidating the table becomes a much more manageable task.

Because the coarse gradation exhibited the greatest sensitivity in VMA between the varying levels of N_{design}, it was decided that any consolidation of the N_{design} compaction matrix must take place based upon changes in VMA with respect to N_{design} for the coarse graded mixtures. Any consolidation proposed for the coarse graded mixtures would therefore yield conservative consolidated levels for the fine graded mixtures.

A plot of the coarse graded VMA values versus N_{design} levels evaluated in the task is shown in Figure 4.28. Also shown on the plot are the VMA values of the mixtures at the existing levels of N_{design} for temperatures less than 39°C (i.e. 68, 76, 86, 96, 109, 126, and 142 gyrations). The results indicate that VMA values differ by an average of 0.32 percent between each level of N_{design} currently specified. Values of VMA versus N_{design} for the fine graded mixtures are shown in Figure 4.29, and indicate a much smaller average difference (approximately 0.18 percent) in VMA between current N_{design} levels. This is to be expected since the fine graded mixtures have already been shown to be less sensitive to changes in compactive effort.

To continue the consolidation procedure, the next step is to determine what is an acceptable range of VMA for a specified level of N_{design}. A maximum range of VMA of 1.0 percent was selected as the criteria for the consolidation of the N_{design} levels. A range of 1.0 percent would theoretically mean that no mixture would be designed with a VMA more than 0.5 percent from the optimum. As previously mentioned, there are currently seven levels of N_{design} for temperature less than 39°C, with a N_{design} of 96 gyrations being the middle range value for traffic levels between 3 and 10 million ESALs. As shown in Figure 4.30, at a N_{design} level of 96 gyrations, the average VMA value for the coarse graded mixtures equals 13.56 percent. If 1.0 percent increase in VMA is required to move to the next gyration level, the number of gyrations to achieve 14.56 percent VMA equals 68 gyrations. Likewise, if a 1.0 percent decrease is required to move to the next gyration level, the number of gyrations to achieve 12.56 percent VMA equals 142 gyrations. This gyration level (142) is equal to the current number of gyrations for temperatures less than 39°C and a traffic level of greater than 100 million ESALs. With respect to the total roadways in the country, there are only a very small number which would use mixture designs for greater than 100 million ESALs projected over the pavement life. Hence, a more rational level of the high end N_{design} level should be provided, so that mixtures would not be over designed or compacted, which could possibly lead to durability problems. A more suitable number of gyrations would possibly be 126 gyrations, which is currently the N_{design} level for 30 to 100 million ESALs. Taking this into consideration, a proposed consolidation of the N_{design} levels would be the three N_{design} levels of 68, 96, and 126 gyrations. For simplicity purposes, these levels were rounded up to the nearest 5 gyrations, which results in the levels being 70, 100, and 130 gyrations.

However, there are a number of roadways in the country in which the applied traffic volumes are extremely low with limited or no truck traffic. For these applications, even the lowest N_{design} level given previously could be very conservative. Past research *(17)* has shown that the current N_{design} levels, particularly for low volume roadways, may be too high. Therefore, there should be a
lower level of N_{design} provided for these special roadway applications. As with the development of the three levels given previously, this lower N_{design} level can be determined in a similar manner by observing the change in VMA between the gyration levels. By observing Figure 4.30, approximately 47 gyrations would provide a VMA of 15.56 percent, which is a 1.0 percent increase from the N_{design} level of 68 gyrations. The proposed N_{design} levels along with their associated design traffic are shown in Table 4.60. The N_{design} values for various traffic levels are approximately equal to those values presently required as shown in Table 4.58.

4.5.2.3 Compaction Slope Evaluation

The slope of the gyratory compaction curve is thought by some to provide an indication of the strength of the mixture's aggregate structure. Mixture's with steep slopes are thought to exhibit an increased resistance to permanent deformation or rutting. In an effort to determine the ability of the compaction slope to predict permanent deformation or the strength of the aggregate structure, selected specimens from this task were evaluated in the Asphalt Pavement Analyzer (APA). The APA is a laboratory wheel tracking device in which gyratory or beam specimens are subjected to cyclic loading at elevated temperatures. This loading takes place by a loaded wheel rolling along a pressurized rubber hose, which lies across the surface of the test specimen. Specimens at three levels of N_{design} (68, 113, and 172), all aggregate types, and both gradations were evaluated in the APA at 64°C in a dry state to determine their rutting susceptibility. A loading pressure of 689 kPa and 8000 cycles were used for the completion of the testing. Prior to being tested in the APA, specimens from

Table 4.59 Effect of N_{design} Levels (<39°C and 43-44°C) on Voids in Mineral Aggregate for Given Traffic Level

Aggregate	Gradation	< 39°C	43-44°C	VMA Difference	Significant Difference ?[1]
New York Gravel	Coarse	68	82	0.87	NO
		76	93	0.92	YES
		86	105	0.87	YES
		96	119	0.83	NO
		109	135	0.63	NO
		126	153	0.28	NO
		142	172	0.00	NO
	Fine	68	82	0.25	NO
		76	93	0.29	NO
		86	105	0.31	NO
		96	119	0.34	NO
		109	135	0.35	NO
		126	153	0.31	NO
		142	172	0.29	NO
Georgia Granite	Coarse	68	82	0.29	NO
		76	93	0.33	NO
		86	105	0.31	NO
		96	119	0.33	NO
		109	135	0.33	NO
		126	153	0.29	NO
		142	172	0.29	NO
	Fine	68	82	0.31	YES
		76	93	0.35	YES
		86	105	0.35	YES
		96	119	0.33	YES
		109	135	0.29	YES
		126	153	0.22	YES
		142	172	0.14	NO

Notes: (1) Level of Significance of 0.05

Table 4.59 Cont. Effect of N_{design} Levels (<39°C and 43-44°C) on Voids in Mineral Aggregate for Given Traffic Level

Aggregate	Gradation	< 39°C	43-44°C	VMA Difference	Significant Difference ?[1]
Alabama Limestone	Coarse	68	82	0.49	YES
		76	93	0.54	YES
		86	105	0.53	YES
		96	119	0.56	YES
		109	135	0.57	YES
		126	153	0.51	YES
		142	172	0.51	YES
	Fine	68	82	0.26	NO
		76	93	0.29	NO
		86	105	0.29	NO
		96	119	0.31	NO
		109	135	0.31	NO
		126	153	0.28	NO
		142	172	0.28	NO
Nevada Gravel	Coarse	68	82	0.58	NO
		76	93	0.62	NO
		86	105	0.61	NO
		96	119	0.66	NO
		109	135	0.65	NO
		126	153	0.60	NO
		142	172	0.59	NO
	Fine	68	82	0.29	NO
		76	93	0.33	NO
		86	105	0.33	NO
		96	119	0.35	NO
		109	135	0.10	NO
		126	153	0.26	NO
		142	172	0.19	NO

Notes: (1) Level of Significance of 0.05

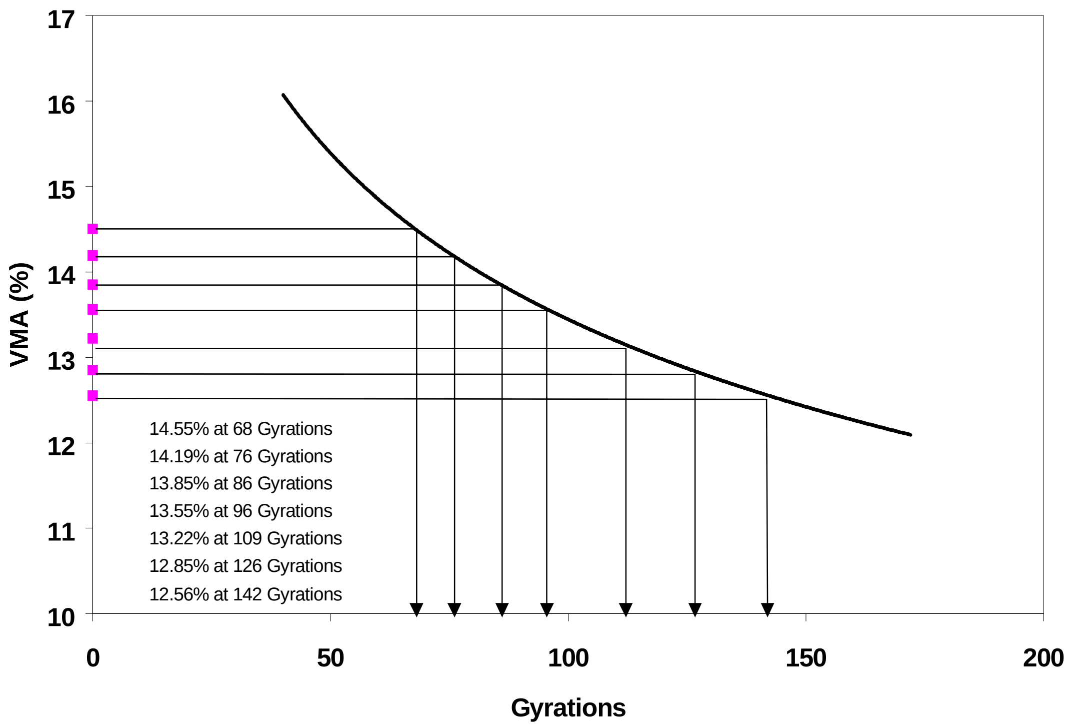

Figure 4.28 VMA versus Gyrations for Task 6C Coarse Graded Mixtures

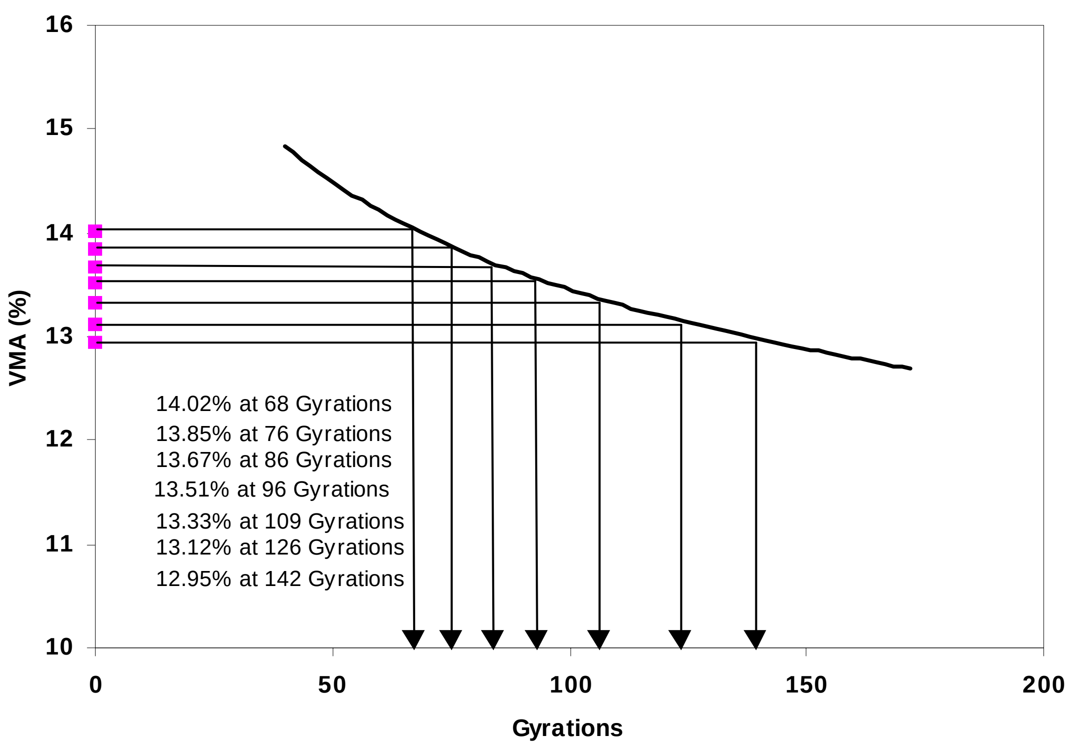

Figure 4.29 VMA versus Gyrations for Task 6C Fine Graded Mixtures

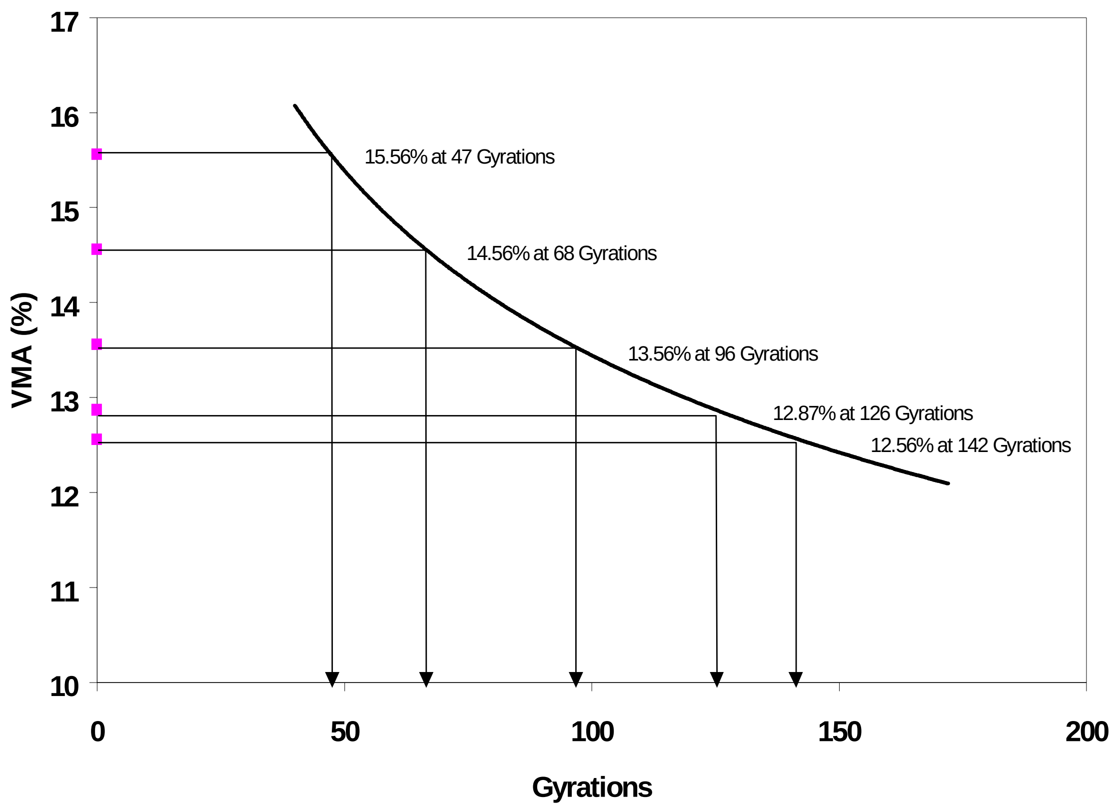

Figure 4.30 Proposed N_{design} Levels Based Upon Changes in

the mix designs were sawed to approximately 75 mm in height and their bulk specific gravity determined. These specimens were chosen to be nearest to the optimum asphalt content, thus having 4 percent air voids, for the given mixtures. Test results from the APA testing include both rut depth versus cycles and also a rutting rate. The rutting rate or slope of the rut depth versus loading curve provides an indication of the stability of the mixture with respect to time and loading. The results of the APA testing can be found in Table 4.61 and in Figure 4.31. Regression analysis of the compaction slope and the APA rut depths showed a poor relationship ($R^2 = 0.289$) for the mixtures evaluated. Further, the relationship that existed was contrary to the belief that an increased compaction slope, indicative of a stronger aggregate structure, is an indication of the mixtures increased resistance to rutting. At this time there is not enough information available to allow any recommendation to include the compaction slope as part of the Superpave volumetric mixture design and evaluation procedure.

Table 4.60 Proposed Consolidation of the N_{design} Compaction Table

Design Traffic Level (ESALs) Millions[3,5]	Compaction Parameters			Typical Roadway Applications [2]
	$N_{initial}$	N_{design} [1]	N_{max}	
Less than 0.1	6	50	74	Applications would include roadways with very light traffic volumes such as local roads, county roads, and city streets where truck traffic is prohibited or at a very minimal level. Traffic on these roadways would be considered local in nature; not regional, intrastate, or interstate. Special purpose roadways serving recreational sites or areas would also be applicable to this level.
0.1 to 1	7	70	107	Applications would include many collector roads or access streets. Medium trafficked city streets and the majority of county roadways would be applicable to this level.
1 to 30	8	100	158	Applications would include many two lane, multilane, and divided partially or completely controlled access roadways. Among these are medium to highly trafficked city streets, many state routes, US highways, and some rural interstates. Use for Stone Matrix Asphalt (SMA)[4]
Greater than 30	9	130	212	Applications include much of the US Interstate system, both rural and urban in nature. Special applications such as truck weighing stations or truck climbing lanes on two lane roadways would also be applicable to this level.

Notes:
(1) It is suggested that Superpave mixtures be compacted to N_{design} gyrations.
(2) Typical applications as defined by A Policy on Geometric Design of Highways and Streets, 1994, AASHTO. *(50)*
(3) Values shown are based upon 20 year ESALs. For roadways designed for more or less than 20 years, determine the estimated ESALs for 20 years and choose the appropriate N_{design} level.
(4) When the Los Angeles (L.A.) abrasion value for the aggregate used in SMA exceeds 30 or when designing for less than 1 million ESALs, consider dropping to the next lower compaction level (70 gyrations).
(5) When the mixture being designed is to be placed more than 100 mm from the finished surface the N_{design} requirement can be dropped one (1) traffic level. When the mixture is more than 200 mm from the surface the N_{design} requirement can be dropped two (2) traffic levels. However, if the mixture being designed at lower gyrations is exposed to significant traffic prior to being overlaid, significant stability problems may occur.

Table 4.61 Asphalt Pavement Analyzer Testing Results

Aggregate	Gradation	N_{design}	Asphalt Content (%)	Air Voids (%)	Compaction Slope (%) N_{inital} - N_{design}	Rut Depth at 8000 cycles (mm)	Rutting Rate (mm/cycle) (10^{-4})
New York Gravel	Fine	68	5.0	3.3	6.15	2.37	2.19
		113	4.5	3.7	5.71	1.64	1.92
		172	4.5	3.3	5.28	1.09	1.33
	Coarse	68	5.5	3.7	9.16	3.23	2.65
		113	4.5	4.3	8.63	2.48	2.28
		172	4.0	3.9	7.92	1.64	1.01
Georgia Granite	Fine	68	5.0	3.5	5.96	2.32	2.42
		113	4.5	4.1	5.54	2.07	2.11
		172	4.5	3.5	5.18	1.57	2.68
	Coarse	68	4.5	3.3	8.29	1.89	1.96
		113	4.0	4.2	7.88	1.67	1.53
		172	4.0	3.3	7.29	1.37	0.86
Alabama Limestone	Fine	68	4.5	2.7	7.56	1.49	1.25
		113	4.0	3.6	6.69	1.88	1.39
		172	3.5	4.7	5.93	2.00	1.19
	Coarse	68	4.0	4.0	10.27	2.54	2.02
		113	3.5	3.7	9.71	2.33	1.71
		172	3.0	4.0	8.86	2.11	1.63
Nevada Gravel	Fine	68	5.5	3.9	7.23	2.57	3.56
		113	5.5	3.8	6.92	1.65	1.76
		172	5.0	4.3	6.42	2.12	1.17
	Coarse	68	6.5	3.0	9.36	2.74	3.60
		113	5.5	3.7	8.85	2.58	1.47
		172	5.5	3.1	8.23	1.78	1.28

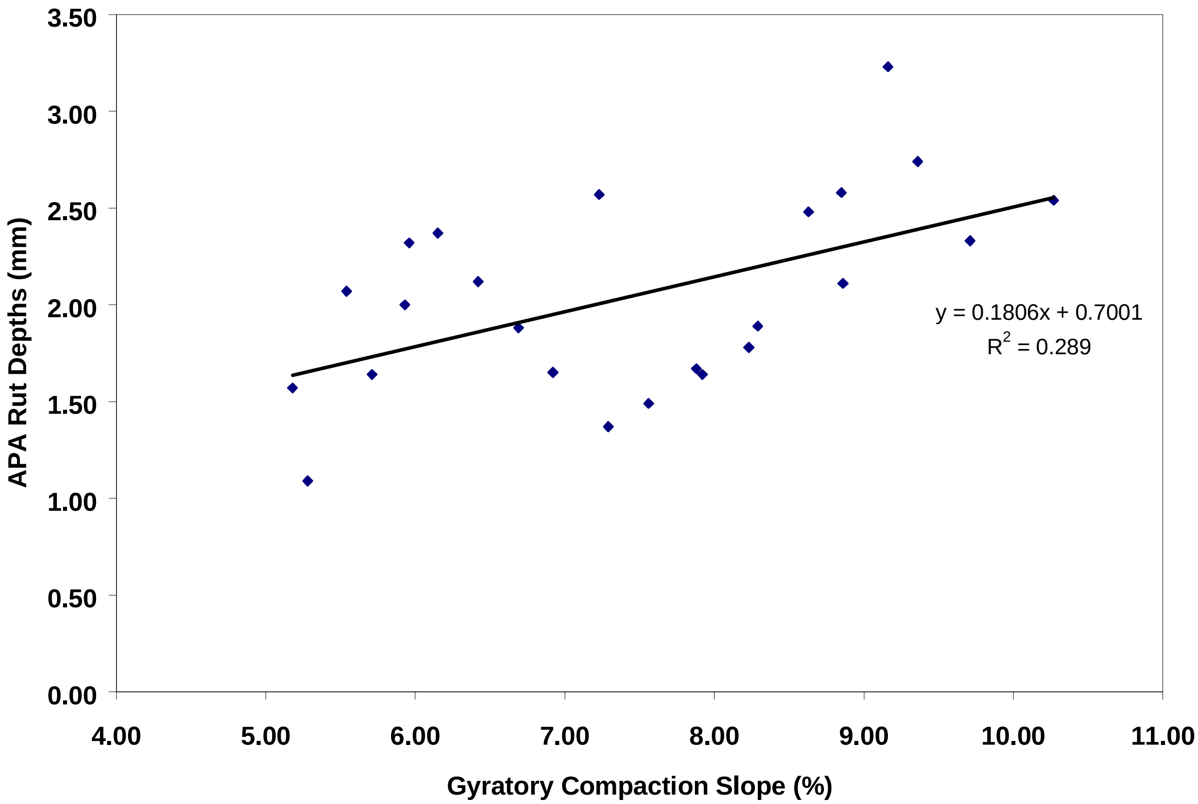

Figure 4.31 Asphalt Pavement Analyzer Rut Depths versus Gyratory Compaction Slope

4.5.2.4 $N_{initial}$ Compaction Requirement Evaluation

The $N_{initial}$ maximum density requirement in Superpave was established to ensure that mixtures which compact too quickly in the gyratory compactor would not be used in the field. It is thought that these mixtures would be prone to tenderness during field compaction. This requirement, along with other aggregate consensus properties, will limit the amount of non-anglular particles (i.e. sand) which can be used in a mixture. However, in many cases, a mixture designed on the fine side of the restricted zone with 100 percent crushed particles will fail to meet the $N_{initial}$ requirement. This may be satisfactory for high volume roadways, but these mixtures should be allowed for lower volume roadways. From an observation of Table 4.52 and Figure 4.32, it is seen that the majority (54 percent or 13 of 24) of fine graded mixtures evaluated in this task did not meet the $N_{initial}$ requirement. None of the New York gravel or the Georgia granite fine graded mixtures met the requirement at any N_{design} level.

These mixtures, the New York gravel and the Georgia Granite, had completely crushed fine aggregates with uncompacted void (FAA) values of 46.9 and 49.4, respectively. Of the eleven mixtures which met the $N_{initial}$ requirement with all crushed fine aggregate particles, nine had densities of 88.0 percent or greater at $N_{initial}$. Based on experience, these mixtures would have an increased difficulty in meeting the density requirement at $N_{initial}$, if even a small portion of uncrushed or natural sand was substituted in the fine aggregate blend. Likewise, the densities at $N_{initial}$ of the mixtures failing the requirement would also increase in a similar manner if the above scenario was to occur.

It is also interesting to note that some mixtures met the $N_{initial}$ requirement for higher number of gyrations (higher traffic level), but failed to meet the requirement for lower gyration levels (lower traffic level). If an aggregate is satisfactory for higher traffic volumes, it should also be satisfactory for lower traffic volumes. It is clear that the $N_{initial}$ requirement needs to be modified for lower volume roadways.

To date in the Superpave mix design system, a majority of mixtures which have been produced have been S-shaped coarse graded mixes. Few fine graded mixtures have been produced, mainly because of the mixtures failing to meet the density requirement at $N_{initial}$. Fine graded mixtures have been used in the past, in some cases, with good field performance results, especially for lower volume roadways. There is no reason to think these fine graded mixtures, when designed properly, cannot perform well in the future, especially in lower volume roadway applications where the mixture strength does not have to be as great. Using these fine graded mixtures could result in a more economical mixture and a mixture which is more readily compacted, while at the same time providing a good performing pavement.

Therefore, it seems evident that the $\%G_{mm}$ at $N_{initial}$ density requirement should be modified for lower volume roadways (i.e. less than 1 million ESALs). In considering the modification, a logical approach is to use the gyratory compaction slope measured from $N_{initial}$ to N_{design}. The compaction slope can be calculated as follows:

$$Slope = \frac{(\%G_{mm} @ N_{design} - \%G_{mm} @ N_{initial})}{(\log N_{design} - \log N_{initial})} \qquad \textit{Equation 3}$$

The minimum compaction slopes for the levels of $N_{initial}$ and N_{design} shown in Table 4.60 were calculated using the above equation and are provided in Table 4.62. These minimum compaction slopes were determined by assuming the mixture had exactly 89 % G_{mm} and 96 % G_{mm} at $N_{initial}$ and N_{design}, respectively. By an observation of the slopes in Table 4.61, it is evident that the smallest compaction slope (6.04) occurs at the highest level of N_{design} (130).

Table 4.62 $N_{initial}$ Values Based upon Minimum Slopes

Traffic Level (Millions)	$N_{initial}$	N_{design}	Minimum Slope for 89 $\%G_{mm}$ at $N_{initial}$	$\%G_{mm}$ at $N_{initial}$ to Provide Slope = 6.04	Recommended $\%G_{mm}$ at $N_{initial}$
< 0.1	6	50	7.60	90.44	91.5
0.1 to < 1.0	7	70	7.00	89.96	90.5
1.0 to < 30.0	8	100	6.38	89.37	89.0
> 30.0	9	130	6.04	89.00	89.0

Therefore, the next step is to determine the $\%G_{mm}$ at $N_{initial}$ for the respective traffic levels, which will provide a compaction slope equivalent to the minimum calculated slope of 6.04. This is accomplished by solving Equation 1 for $\%G_{mm}$ at $N_{initial}$ with the compaction slope and $\%G_{mm}$ at N_{design} held constant at 6.04 and 96, respectively. From an observation of the results shown in Table 4.62, it is seen that for N_{design} levels of 50 (less than 0.1 million ESALs) and 70 gyrations (0.1 to less than 1.0 million ESALs), the calculated $\%G_{mm}$ at $N_{initial}$ to provide the minimum slope

(6.04) is 90.44 and 89.96, respectively.

However, upon observation of the %G_{mm} at $N_{initial}$ results from the study as shown in Table 4.52 and Figure 4.32, some of the New York gravel and the Georgia granite fine graded mixtures had densities which were very close to these calculated values for the same traffic levels. Again,

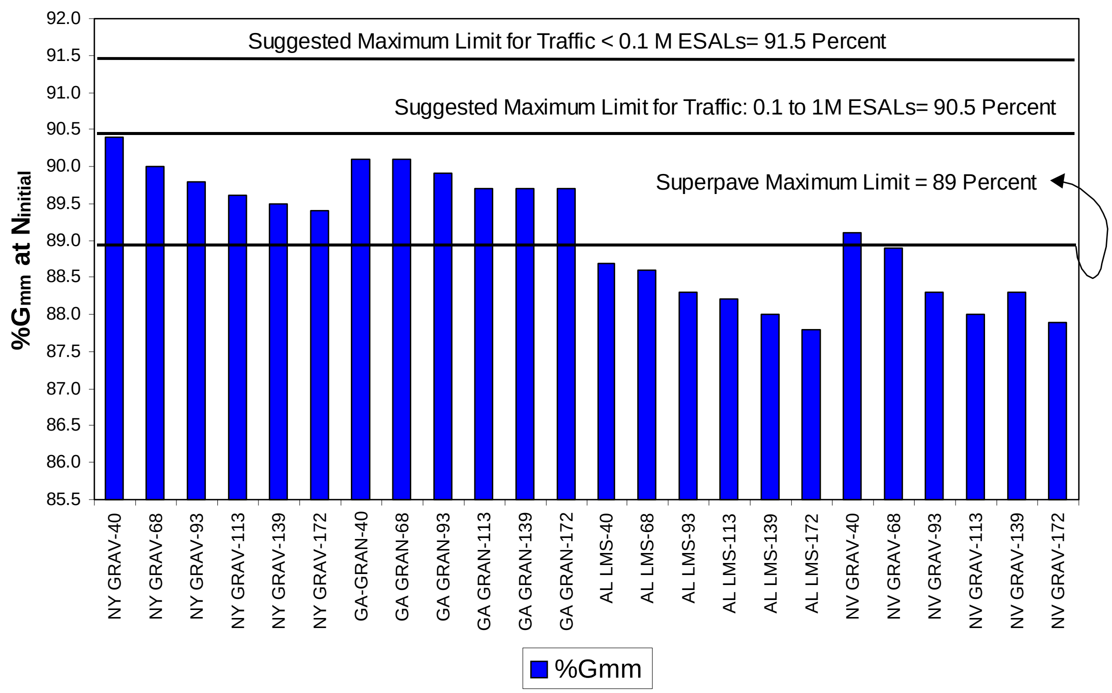

Figure 4.32 %G_{mm} at $N_{initial}$ for Fine Graded Mixtures

remember that all of these mixtures contained crushed fine aggregate which far exceeded the fine aggregate angularity requirements for traffic levels less than one million ESALs. Furthermore, if even a small percentage of natural sand or uncrushed fine aggregate was added to the aggregate blend, the chances of the resulting mixture meeting the %G_{mm} at $N_{initial}$ requirements of 90.44 and 89.96 would be reduced substantially. Therefore, it seems the %G_{mm} at $N_{initial}$ values for less than one million ESALs (i.e N_{design} levels of 50 and 70) should be increased slightly to account for this possibility. Recommended values of %G_{mm} at $N_{initial}$ are shown in the last column of Table 4.62. As can be seen, a value of 91.5 percent is recommended for traffic levels less than 0.1 million ESALs and 90.5 for traffic levels between 0.1 and 1.0 million ESALs. For traffic levels greater than or equal to 1.0 million ESALs, the requirement of %G_{mm} at $N_{initial}$ of 89.0 seems appropriate and should remain unchanged.

4.5.2.5 $N_{maximum}$ Compaction Requirement Evaluation

The $N_{maximum}$ requirement in Superpave specifies that the compacted specimen's density shall not exceed 98 percent of G_{mm}. It is thought that mixtures which compact beyond this density will be prone to continued densification, past the design air voids of 4 percent, which could lead to permanent deformation or rutting.

Duplicate specimens were compacted to $N_{maximum}$ levels of 104 (N_{design} 68), 181 (N_{design} 113), and 288 (N_{design} 172). These specimens were compacted at the previously determined optimum asphalt contents. The results of the evaluation are shown in Table 4.63 and indicate that none of the mixtures evaluated in this study fail the maximum density requirement of 98 percent of G_{mm}. The data indicates that the coarse graded mixtures generally had the highest densities at $N_{maximum}$, ranging from 97.2 to 97.9 percent of G_{mm}. The fine graded mixtures had densities at $N_{maximum}$ ranging from 96.4 to 96.7. It appears that mixtures which have been designed to meet the proper volumetric properties, do not exhibit a difficulty meeting the $N_{maximum}$ density requirement.

Based upon the results of this testing, the requirement of a maximum density at $N_{maximum}$ does not seem to be appropriate. The basic premise behind the idea, in general, is a good thought. However, if all the mixtures meet the requirement, there is no benefit provided by the requirement itself. The mixtures that most often approached failure were the coarse graded mixtures, which should provide the best performance.

Further, it has been shown that compacting samples to $N_{maximum}$ and using a back calculation procedure to determine volumetric properties at N_{design} results in volumetric property errors in many cases. *(32)* It is therefore recommended that the requirement of a maximum density at $N_{maximum}$ be eliminated from the volumetric mix design procedure. Specimens should be compacted to their respective N_{design} levels and their volumetric properties determined accordingly. An amount of error may still be present when the specimen density at $N_{initial}$ is back calculated from N_{design}; however, the error will be less than the error resulting from using the present system.

The specification values of density at $N_{initial}$ should be raised or lowered to account for the change in the mixture's air voids at N_{design}. The amount that the specification values are changed should be equal to the difference in the measured and design $\%G_{mm}$ at N_{design}. For example, if a particular mixture, has air voids of 4.5 percent at N_{design} when the target or design voids is 4.0 percent, the $N_{initial}$ density requirement should be increased the same amount of 0.5 percent.

4.5.2.6 Interpretation of the N_{design} Table

Some clarification of the current Superpave N_{design} gyration table is needed. As the table exists today, the N_{design} levels are assumed to be for a design life of 20 years, so all the traffic levels shown represent 20 years of traffic. The question arises on how to use the table when the design life is something other than 20 years. Another problem is that the rate of loading is also a factor. For example, 100,000 ESALs in one month is a more severe loading application than 100,000 ESALs in one year.

To solve this problem, any HMA mixture which is designed for less than 5 years should be extrapolated for 5 years of traffic. For example, if a detour is designed to carry 1 million ESALs for an expected period of 1 year, it should be designed to carry 5 million ESALs (rate controls) in the table. Any pavement designed for 5 to 20 years should be designed for the actual number of ESALs anticipated in pavement design life. Any mixture which is designed for more than 20 years should be designed for the actual number of ESALs anticipated in the first 20 years.

These numbers were selected on the assumption that most densification will occur within the first 5 years after construction. Some small amount of densification will occur after 5 years, but only a very small amount or none after 20 years.

Table 4.63 Summary of Properties Determined at N_{design} and $N_{maximum}$ for Task 6C

				Volumetric/Compaction Properties					
Aggregate	Gradation	Optimum Asphalt Content	N_{design} ($N_{maximum}$)	%G_{mm} at $N_{initial}$	% G_{mm} at N_{design}	% G_{mm} at $N_{maximum}$	Compaction Slope	VMA	VFA
New York Gravel	Fine	5.0	68 (104)	90.0 (90.1)[1]	96.0 (95.9)	(96.8)	6.15 (5.77)[2]	13.6 (13.5)[1]	70.8 (69.8)[1]
		4.7	113 (181)	89.6 (89.8)	96.1 (95.9)	(96.9)	5.71 (5.31)	12.9 (12.8)	70.0 (68.2)
		4.4	172 (288)	89.4 (88.9)	95.9 (95.4)	(96.4)	5.28 (5.17)	12.3 (12.5)	67.0 (63.7)
	Coarse	5.6	68 (104)	87.0 (87.6)	96.1 (96.5)	(97.9)	9.16 (8.79)	15.0 (14.6)	73.9 (75.9)
		4.7	113 (181)	86.0 (86.3)	95.9 (96.3)	(97.4)	8.63 (8.13)	13.2 (12.9)	69.2 (71.1)
		4.1	172 (288)	86.1 (86.0)	95.9 (95.9)	(97.2)	7.92 (7.68)	12.0 (11.9)	65.9 (65.6)
Georgia Granite	Fine	5.1	68 (104)	90.1 (90.4)	96.1 (96.3)	(97.2)	5.96 (5.79)	15.9 (15.4)	75.5 (76.1)
		4.7	113 (181)	89.7 (89.7)	96.0 (95.9)	(96.7)	5.54 (5.23)	15.1 (14.9)	73.5 (72.9)
		4.5	172 (288)	89.7 (89.9)	96.0 (96.3)	(97.0)	5.18 (4.83)	14.6 (14.2)	72.9 (73.6)
	Coarse	4.4	68 (104)	86.5 (87.6)	96.0 (96.1)	(97.4)	8.29 (8.32)	14.8 (14.2)	73.0 (72.7)
		4.2	113 (181)	86.8 (87.0)	95.9 (96.1)	(97.5)	7.88 (7.73)	14.1 (13.8)	71.0 (71.5)
		3.9	172 (288)	86.9 (87.4)	95.9 (96.7)	(97.8)	7.29 (7.16)	13.5 (12.5)	69.4 (73.7)

Notes: (1) The number outside and inside parenthesis indicates properties back calculated from N_{design} and $N_{maximum}$, respectively.
(2) Slope values outside and inside parenthesis represent slope calculated from $N_{initial}$ to N_{design} and $N_{initial}$ to $N_{maximum}$, respectively.

Table 4.63 cont. Summary of Properties Determined at N_{design} and $N_{maximum}$ for Task 6C

				Volumetric/Compaction Properties					
Aggregate	Gradation	Optimum Asphalt Content	N_{design} ($N_{maximum}$)	$\%G_{mm}$ at $N_{initial}$	$\% G_{mm}$ at N_{design}	$\% G_{mm}$ at $N_{maximum}$	Compaction Slope	VMA	VFA
Alabama Limestone	Fine	4.3	68 (104)	88.6 (88.8)[1]	96.1 (96.6)	(97.7)	7.56 (7.57)[2]	12.7 (12.7)[1]	69.2 (73.5)[1]
		3.8	113 (181)	88.2 (88.2)	96.2 (96.3)	(97.5)	6.69 (6.87)	11.5 (11.8)	66.8 (68.9)
		3.6	172 (288)	87.8 (88.1)	95.9 (96.4)	(97.5)	5.93 (6.43)	11.4 (11.3)	63.4 (68.4)
	Coarse	4.2	68 (104)	85.9 (85.3)	96.0 (95.6)	(97.3)	10.27 (10.29)	13.0 (13.4)	69.6 (67.6)
		3.8	113 (181)	85.6 (84.3)	96.1 (95.9)	(97.4)	9.71 (9.70)	11.7 (11.9)	66.5 (65.2)
		3.6	172 (288)	85.3 (84.6)	96.2 (96.0)	(97.5)	8.86 (8.85)	10.7 (10.9)	65.0 (63.4)
Nevada Gravel	Fine	5.6	68 (104)	88.9 (87.7)	96.0 (95.9)	(97.0)	7.23 (7.20)	13.9 (13.9)	71.2 (70.5)
		5.6	113 (181)	88.0 (88.2)	96.0 (96.1)	(97.2)	6.92 (6.69)	13.7 (13.7)	70.8 (71.5)
		5.2	172 (288)	87.8 (88.1)	95.7 (96.1)	(97.2)	6.42 (6.25)	13.6 (12.9)	67.8 (69.6)
	Coarse	6.3	68 (104)	87.0 (86.5)	96.2 (95.7)	(97.2)	9.36 (9.14)	15.5 (15.9)	75.6 (73.1)
		5.5	113 (181)	86.0 (85.3)	96.2 (95.9)	(97.3)	8.85 (8.29)	13.8 (14.7)	72.6 (66.9)
		5.4	172 (288)	85.8 (85.8)	96.0 (95.6)	(97.3)	8.23 (7.88)	13.6 (13.8)	70.8 (70.6)

Notes: (1) The number outside and inside parenthesis indicates properties back calculated from N_{design} and $N_{maximum}$, respectively.
(2) Slope values outside and inside parenthesis represent slope calculated from $N_{initial}$ to N_{design} and $N_{initial}$ to $N_{maximum}$, respectively.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 INTRODUCTION

The overall goal of this research project was to answer the following questions:

- Determine Superpave mixture design procedures for gap graded and large stone mixtures.
- Evaluate the potential for using the mixture's compaction temperature as the short-term aging temperature.
- Determine the appropriate design number of gyrations for mixtures as a function of depth.
- Evaluate the current N_{design} compaction matrix and determine whether the levels can be consolidated.
- Evaluate the current density requirements at $N_{initial}$ and $N_{maximum}$ and determine whether the specification values are appropriate.

5.2 CONCLUSIONS

The research data indicates that gap graded and large stone mixtures can be designed without problems in the Superpave gyratory compactor. Many S-shaped, medium gaped aggregate blends are currently being used in the Superpave system without major problems. The gap graded mixtures evaluated were stone matrix asphalt mixtures. A correlation between N_{design} and traffic level could not be readily established based upon the limited data evaluated; however, other recent research has shown that the design level for SMA should be approximately 80 gyrations. *(38)*

The large stone mixture research data obtained for field mixtures showed that a lower number of gyrations should be used than is currently specified. However, most of the mixtures evaluated were placed in binder or base courses, not in the surface course. The same criteria should be used for large stone mixtures as for other Superpave mixtures.

Additionally, the research results from laboratory prepared specimens showed that the number of gyrations for mixtures greater than 100 mm below the finished pavement surface may be reduced by approximately 28 percent to account for the reduced vertical pressure and lower temperatures at increased depths within the pavement structure. For mixtures greater than 200 mm from the finished pavement surface, the design number of gyrations may be reduced even more.

The compaction temperature of an asphalt mixture can be used as the short-term aging temperature, in lieu of 135°C, without significantly affecting the mixture volumetric and densification properties. Additionally, based upon a limited study with a low absorption aggregate, there was no significant difference in the measured air voids of mixtures aged at two and four hours.

The current N_{design} compaction matrix can be consolidated from 28 to four compaction levels and still provide a range of mixture quality for all traffic categories. These traffic levels include low, medium, high, and very high.

In addition, it appears that the requirement of $\%G_{mm}$ at $N_{initial}$ of 89 percent for lower traffic volume roadways is too stringent. Fine graded mixtures in the study which were comprised of crushed fine aggregate materials failed this requirement, especially at the lower N_{design} values. This requirement should be raised for the lower volume roadways to allow for more fine graded mixtures to be used.

Generally, mixtures which are more likely to fail the density requirements at $N_{maximum}$ were the coarse graded mixtures, not the fine graded mixture. Mixtures which have been designed to have four percent air voids at N_{design}, do not fail the $N_{maximum}$ density requirement. All the mixtures evaluated passed the $N_{maximum}$ requirement, but the mixtures which came the closest to failing the requirement were the coarse graded mixtures. If mixtures are not controlled at four percent air voids or higher in the field, it will be very difficult to meet $N_{initial}$ and $N_{maximum}$ requirements.

Currently, the gyratory compaction procedure requires that specimens be compacted to $N_{maximum}$ and densities and volumetric properties be back-calculated at N_{design}, This causes an error in the calculated volumetric properties at N_{design}. Since the mixture is designed based upon its volumetric properties at N_{design}, the volumetric mix design procedure should be conducted by compacting samples to N_{design}, not $N_{maximum}$.

Based on the test results, the gyratory compaction slope does not appear to be a good indication of the strength of the aggregate structure of the asphalt mixture. Mixtures designed at lower levels of N_{design} have higher compaction slopes than mixtures designed at higher N_{design} levels. However, the slope does recognize changes that occur in the mixture's asphalt content within a given N_{design} level, with gradation being constant. Therefore, the slope could possibly be used in the quality control or quality assurance testing of an asphalt mixture.

5.3 RECOMMENDATIONS

Considering the research results in this study the following recommendations are made:

It is recommended that the gap graded mixtures, such as stone matrix asphalt (SMA), be designed using 100 gyrations (See Table 5.1). However, there are some cases where the design level should be decreased to 70 gyrations. The decision of the design gyration level should be based upon the experience of the user agency. For higher traffic volume roadways, the designer should consider using 100 gyrations, while 70 gyrations could be used for lower volume roadways. Also, when designing mixtures with aggregates which tend to break down during lab compaction (i.e. Los Angeles Abrasion values greater than 30), the design number of gyrations should be 70.

Large stone mixtures (37.5 nominal maximum size) placed in the top 100 mm of the pavement structure should be designed using the suggested levels of N_{design} for dense graded mixtures. For mixtures which are placed below 100 mm in the pavement structure, the design number of gyrations may be reduced one level from those shown in Table 5.1. When placed more than 200 mm into the pavement structure the design gyrations may be reduced by two levels. However, if it is likely that these mixtures will be open to traffic for any significant period of time (more than 2-3 days) prior to the construction of the overlying mixture, the design number of gyrations for the large stone mixtures should remain at the level used for surface course mixtures.

It is recommended that the short term oven aging temperature used in the Superpave volumetric mix design process be changed from 135°C to the compaction temperature of the asphalt mixture as determined from the temperature-viscosity relationship of the asphalt binder. Based upon the limited study with a low absorption aggregate (less than two percent water absorption), the Superpave mixture expert task group's (ETG) recommendation of a two hour short term aging period for mixtures with low absorption aggregates is valid. However, additional research should be performed with aggregates having a range of absorption values to make further recommendations concerning the reduction of the short term aging time from four to two hours.

The N_{design} level for the mixtures placed below 100 mm in the structure may be reduced by one level from the N_{design} level used for the surface mixtures. When a mixture is placed more than 200 mm below the surface, the design gyration level may be reduced two levels. Again, if these mixtures are to be subjected to traffic for any significant period of time (2-3 days) prior to the construction of the overlying mixture, the design number of gyrations for these mixtures should not be reduced.

The current N_{design} compaction matrix of 28 levels used in the Superpave mixture design procedure (AASHTO PP 28) should be reduced to four levels. Further, the gyratory compaction requirement for $\%G_{mm}$ at $N_{initial}$ of 89 percent should be increased slightly for mixtures designed for low traffic volumes. Values of the proposed N_{design} levels and the revised $\%G_{mm}$ at $N_{initial}$ are provided in Table 5.1.

The Superpave volumetric mix design should be completed by compacting specimens to their respective N_{design} values, and not $N_{maximum}$ as currently exists. Once the optimum asphalt content of the mixture has been determined, triplicate specimens should be prepared at the optimum asphalt content and compacted to the respective $N_{maximum}$. The average specimen density should then be calculated and compared against the density requirement at $N_{maximum}$ of less than 98.0 percent of G_{mm}.

During quality control or quality assurance testing of a mixture's volumetric and densification properties, the specification values of density at $N_{initial}$ should be raised or lowered to account for the change in the mixture's air voids at N_{design}. The amount that the specification values are changed should be equal to the difference in the measured and design $\%G_{mm}$ (four percent) at N_{design}.

Table 5.1 - Superpave Design Compactive Effort and Aggregate Consensus Property Requirements

Estimated Design Traffic Level (Million ESALs)[2]	Superpave Compaction Parameters			$\%G_{mm}$ $N_{initial}$ Requirement	Aggregate Consensus Properties					
					Coarse Aggregate Angularity		Fine Aggregate Angularity[7]		Clay Content[8]	Flat and Elongated[9]
	$N_{initial}$	N_{design}	$N_{maximum}$		≤ 100 mm	> 100 mm	≤ 100 mm	> 100 mm	All Mixtures	All Mixtures
< 0.1	6	50	74	< 91.5	55/-	-/-	-	-	40	< 10 %
0.1 to < 1.0	7	70	107	< 90.5	65/-	-/-	40	-	40	
1.0 to < 30.0	8	100[10]	158	< 89.0	75/- [3]	50/-	40	40	45	
					85/80 [4,6]	60/-	45	40	45	
					95/90[5]	80/75	45	40	45	
≥30.0	9	130	212	< 89.0	100/100	100/100	45	45	50	

Notes:
(1) It is recommended that Superpave mixtures be compacted to N_{design} gyrations.
(2) Values shown are based upon 20 year ESALs. For roadways designed for more or less than 20 years, determine the estimated ESALs for 20 years and choose the appropriate N_{design} level.
(3) Requirements apply to traffic levels from 1 to < 3 million ESALs.
(4) Requirements apply to traffic levels from 3 to < 10 million ESALs.
(5) Requirements apply to traffic levels from 10 to < 30 million ESALs.
(6) "85/80" denotes that 85 % of the coarse aggregate has one fractured face and 80% has two or more fractured faces.
(7) Criteria are minimum presented as percent air voids in loosely compacted fine aggregate. Test to be run in accordance with AASHTO TP-33.
(8) No distinction is made between depth from surface. Test to be run in accordance with AASHTO T176.
(9) Criterion based upon a 5:1 maximum to minimum ratio.
(10) Use for Stone Matrix Asphalt (SMA). However, when the L.A. Abrasion value for the aggregate used in SMA exceeds 30, consider dropping to the next lowest compaction level (70 gyrations).