RELATED DOCUMENTS

MIX DESIGN:

1. AASHTO Designation T 312 "Standard Method of Test for Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor

2. AASHTO Designation R 35 "Standard Practice for Superpave Volumetric Design for Hot-Mix Asphalt (HMA)

RESTRICTED ZONE

1. NCHRP Report 464 – "The Restricted Zone in the Superpave Aggregate Gradation Specification"

> "Mixes meeting the Superpave and FAA requirements with gradations that violated the restricted zone performed similarly to or better than the mixes with gradations passing outside the restricted zone. This conclusion is drawn from the results of experiments with 9.5 and 19-mm NMAS gradations at Ndesign values of 75, 100 and 125 gyrations and is supported by extensive, independent results from the literature"

GYRATION LEVELS

1. NCHRP WEB DOCUMENT 34 – "Literature Review: Verification of Gyration Levels in the Superpave Ndesign Table"

Traffic Level (EASLs)	Gyrations
Less than 300,000	50
300,000 – 3 million	75
3 million – 30 million	100
Greater than 30 million	125

Standard Method of Test for

Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor

AASHTO Designation: T 312-04

1. SCOPE

1.1. This standard covers the compaction of cylindrical specimens of hot-mix asphalt (HMA) using the Superpave gyratory compactor.

1.2. *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- M 231, Weighing Devices Used in the Testing of Materials
- PP 48, Evaluation of the Superpave Gyratory Compactor (SGC) Internal Angle of Gyration
- R 30, Mixture Conditioning of Hot-Mix Asphalt (HMA)
- R 35, Superpave Volumetric Design for Hot-Mix Asphalt (HMA)
- T 166, Bulk Specific Gravity of Compacted Hot-Mix Asphalt Using Saturated Surface-Dry Specimens
- T 168, Sampling Bituminous Paving Mixtures
- T 209, Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
- T 275, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens
- T 316, Viscosity Determination of Asphalt Binder Using Rotational Viscometer

3. SIGNIFICANCE AND USE

3.1. This standard is used to prepare specimens for determining the mechanical and volumetric properties of HMA. The specimens simulate the density, aggregate orientation, and structural characteristics obtained in the actual roadway when proper construction procedure is used in the placement of the paving mix.

3.2. This test method may be used to monitor the density of test specimens during their preparation. It may also be used for field control of an HMA production process.

4. APPARATUS

4.1. *Superpave Gyratory Compactor*—An electrohydraulic or electromechanical compactor with a ram and ram heads as described in Section 4.3. The axis of the ram shall be perpendicular to the platen of the compactor. The ram shall apply and maintain a pressure of 600 ± 18 kPa perpendicular to the cylindrical axis of the specimen during compaction (Note 1). The compactor shall tilt the specimen molds at an external angle of 22 ± 0.35 mrad (1.25 ± 0.02°) or an average internal angle of 20.2 ± 0.35 mrad (1.16 ± 0.02°), determined in accordance with AASHTO PP 48. The compactor shall gyrate the specimen molds at a rate of 30.0 ± 0.5 gyrations per minute throughout compaction.

Note 1—This stress calculates to 10,600 ± 310 N total force for 150 mm specimens.

4.1.1. *Specimen Height Measurement and Recording Device*—When specimen density is to be monitored during compaction, a means shall be provided to continuously measure and record the height of the specimen to the nearest 0.1 mm during compaction once per gyration.

4.1.2. The system may include a printer connected to an RS232C port capable of printing test information, such as specimen height per gyration. In addition to a printer, the system may include a computer and suitable software for data acquisition and reporting.

4.2. *Specimen Molds*—Specimen molds shall have steel walls that are at least 7.5 mm thick and are hardened to at least a Rockwell hardness of C48. The initial inside finish of the molds shall have a root mean square (rms) of 1.60 μm or smoother (Note 2). Molds shall have an inside diameter of 149.90 to 150.00 mm and be at least 250 mm high at room temperature.

Note 2—Smoothness measurement is in accordance with ANSI B 46.1. One source of supply for a surface comparator, which is used to verify the rms value of 1.60 μm, is *GAR Electroforming*, Danbury, Connecticut.

4.3. *Ram Heads and Mold Bottoms*—Ram heads and mold bottoms shall be fabricated from steel with a minimum Rockwell hardness of C48. The ram heads shall stay perpendicular to their axis. The platen side of each mold bottom shall be flat and parallel to its face. All ram and base plate faces (the sides presented to the specimen) shall be flat to meet the smoothness requirement in Section 4.2 and shall have a diameter of 149.50 to 149.75 mm.

4.4. *Thermometers*—Armored, glass, or dial-type thermometers with metal stems for determining the temperature of aggregates, binder, and HMA between 10 and 232°C.

4.5. *Balance*—A balance meeting the requirements of M 231, Class G 5, for determining the mass of aggregates, binder, and HMA.

4.6. *Oven*—An oven, thermostatically controlled to ±3°C, for heating aggregates, binder, HMA, and equipment as required. The oven shall be capable of maintaining the temperature required for mixture conditioning in accordance with R 30.

4.7. *Miscellaneous*—Flat-bottom metal pans for heating aggregates, scoop for batching aggregates, containers (grill-type tins, beakers, containers for heating asphalt), large mixing spoon or small trowel, large spatula, gloves for handling hot equipment, paper disks, mechanical mixer (optional), lubricating materials recommended by the compactor manufacturer.

4.8. *Maintenance*—In addition to routine maintenance recommended by the manufacturer, check the Superpave gyratory compactor's mechanical components for wear, and perform repair, as recommended by the manufacturer.

5. HAZARDS

5.1. Use standard safety precautions and protective clothing when handling hot materials and preparing test specimens.

6. STANDARDIZATION

6.1. Items requiring periodic verification of calibration include the ram pressure, angle of gyration, gyration frequency, LVDT (or other means used to continuously record the specimen height), and oven temperature. Verification of the mold and platen dimensions and the inside finish of the mold are also required. When the computer and software options are used, periodically verify the data-processing system output using a procedure designed for such purposes. Verification of calibration, system standardization, and quality checks may be performed by the manufacturer, other agencies providing such services, or in-house personnel. Frequency of verification shall follow the manufacturer's recommendations.

6.2. The angle of gyration may refer to either the external angle (tilt of mold with respect to a plane external to the gyratory mold) or the internal angle (tilt of mold with respect to end plate surface within the gyratory mold). Procedures used to verify the calibration of the angle of gyration must be appropriate for measuring the angle desired.

6.2.1. *Method A*—The calibration of the external angle of gyration should be verified using the manufacturer's recommendations for the appropriate SGC.

6.2.2. *Method B*—The calibration of the internal angle of gyration should be verified in accordance with AASHTO PP 48.

6.2.3. The two methods (Method A—external and Method B—internal) of verifying the calibration of the gyration angle shall NOT be considered equivalent. The gyration angle for all SGCs in a group for which compaction results are to be compared shall be verified using the same method.

7. PREPARATION OF APPARATUS

7.1. Immediately prior to the time when the HMA is ready for placement in the mold, turn on the main power for the compactor for the manufacturer's required warm-up period.

7.2. Verify the machine settings are correct for angle, pressure, and number of gyrations.

7.3. Lubricate any bearing surfaces as needed per the manufacturer's instructions.

7.4. When specimen height is to be monitored, the following additional item of preparation is required. Immediately prior to the time when the HMA is ready for placement in the mold, turn on the device for measuring and recording the height of the specimen, and verify the readout is in the proper units, mm, and the recording device is ready. Prepare the computer, if used, to record the height data, and enter the header information for the specimen.

8. HMA MIXTURE PREPARATION

8.1. Weigh the appropriate aggregate fractions into a separate pan, and combine them to the desired batch weight. The batch weight will vary based on the ultimate disposition of the test specimens. If a target air void level is desired, as would be the case for Superpave mix analysis and performance specimens, batch weights will be adjusted to create a given density in a known volume. If the specimens are to be used for the determination of volumetric properties, the batch weights will be adjusted to result in a compacted specimen having dimensions of 150 mm in diameter and 115 ± 5 mm in height at the desired number of gyrations.

Note 3—It may be necessary to produce a trial specimen to achieve this height requirement. Generally, 4500–4700 g of aggregate are required to achieve this height for aggregates with combined bulk specific gravities of 2.55–2.70, respectively.

8.2. Place the aggregate and binder container in the oven, and heat them to the required mixing temperature.

8.2.1. The mixing temperature range is defined as the range of temperatures where the unaged binder has a kinematic viscosity of 170 ± 20 mm^2/s (approximately 0.17 ± 0.02 Pa·s for a binder density of 1.00 g/cm^3) measured in accordance with T 316.

Note 4—Modified asphalts may not adhere to the equiviscosity requirements noted, and the manufacturer's recommendations should be used to determine mixing and compaction temperatures.

Note 5—The SI unit of kinematic viscosity is m^2/s; for practical use, the submultiple mm^2/s is recommended. The more familiar centistoke is a *cgs* unit of kinematic viscosity; it is equal to 1 mm^2/s. The kinematic viscosity is the ratio of the viscosity of the binder to its density. For a binder with a density equal to 1.000 g/cm^3, a kinematic viscosity of 170 mm^2/s is equivalent to a viscosity of 0.17 Pa·s measured in accordance with T 316.

8.3. Charge the mixing bowl with the heated aggregate from one pan and dry-mix thoroughly. Form a crater in the dry-blended aggregate, and weigh the required amount of binder into the mix. Immediately initiate mixing.

8.4. Mix the aggregate and binder as quickly and thoroughly as possible to yield HMA having a uniform distribution of binder. As an option, mechanical mixing may be used.

8.5. After completing the mixture preparation perform the required mixture conditioning in accordance with R 30.

8.6. Place a compaction mold and base plate in an oven at the required compaction temperature for a minimum of 30 minutes prior to the estimated beginning of compaction (during the time the mixture is being conditioned in accordance with R 30).

8.7. Following the mixture conditioning period specified in R 30, if the mixture is at the compaction temperature, proceed immediately with the compaction procedure as outlined in Section 9. If the compaction temperature is different from the mixture conditioning temperature used in accordance with R 30, place the mix in another oven at the compaction temperature for a brief time (maximum of 30 minutes) to achieve the required temperature.

8.7.1. The compaction temperature is the mid-point of the range of temperatures where the unaged binder has a kinematic viscosity of 280 ± 30 mm^2/s (approximately 0.28 ± 0.03 Pa·s) measured in accordance with T 316 (Note 4).

8.8. If loose HMA plant mix is used, the sample should be obtained in accordance with T 168. The mixture shall be brought to the compaction temperature range by careful, uniform heating in an oven immediately prior to molding.

9. COMPACTION PROCEDURE

9.1. When the compaction temperature is achieved, remove the heated mold, base plate, and upper plate (if required) from the oven. Place the base plate and a paper disk in the bottom of the mold.

9.2. Place the mixture into the mold in one lift. Care should be taken to avoid segregation in the mold. After all the mix is in the mold, level the mix, and place another paper disk and upper plate (if required) on top of the leveled material.

9.3. Load the charged mold into the compactor, and center the loading ram.

9.4. Apply a pressure of 600 ± 18 kPa on the specimen.

9.5. Apply a 22.0 ± 0.35 mrad (1.25 ± 0.02°) external angle or a 20.2 ± 0.35 mrad (1.16 ± 0.02°) average internal angle, as appropriate, to the mold assembly, and begin the gyratory compaction.

9.6. Allow the compaction to proceed until the desired number of gyrations specified in R 35 is reached and the gyratory mechanism shuts off.

9.7. Remove the angle from the mold assembly; retract the loading ram; remove the mold from the compactor (if required); and extrude the specimen from the mold.

Note 6—No additional gyrations with the angle removed are required unless specifically called for in another standard referencing T 312. The extruded specimen may not be a right angle cylinder. Specimen ends may need to be sawed to conform to the requirements of specific performance tests.

Note 7—The specimens can be extruded from the mold immediately after compaction for most HMA. However, a cooling period of 5 to 10 minutes in front of a fan may be necessary before extruding some specimens to insure the specimens are not damaged.

9.8. Remove the paper disks from the top and bottom of the specimens.

Note 8—Before reusing the mold, place it in an oven for at least five minutes. The use of multiple molds will speed up the compaction process.

10. DENSITY PROCEDURE

10.1. Determine the maximum specific gravity (G_{mm}) of the loose mix in accordance with T 209 using a companion sample. The companion sample shall be conditioned to the same extent as the compaction sample.

10.2. Determine the bulk specific gravity (G_{mb}) of the specimen in accordance with T 166 or T 275 as appropriate.

10.3. When the specimen height is to be monitored, record the specimen height to the nearest 0.1 mm after each revolution in addition to those specified in Section 8.

11. DENSITY CALCULATIONS

11.1. Calculate the uncorrected relative density ($\%G_{mmx}$) at any point in the compaction process using the following equation:

$$\%G_{mmx} = \frac{W_m}{V_{mx} G_{mm} G_m} \times 100 \qquad (1)$$

where:

$\%G_{mmx}$	=	uncorrected relative density at any point during compaction expressed as a percent of the maximum theoretical specific gravity;
W_m	=	mass of the specimen in g;
G_{mm}	=	theoretical maximum specific gravity of the mix;
G_m	=	unit weight of water, 1 g/cm^3;
x	=	number of gyrations; and
V_{mx}	=	volume of the specimen, in cm^3, at any point based on the diameter (d) and height (h_x) of the specimen at that point (use "mm" for height and diameter measurements).

It can be expressed as:

$$V_{mx} = \frac{\pi d^2 h_x}{4 \times 1000} \qquad (2)$$

Note 9—This formula gives the volume in cm^3 to allow a direct comparison with the specific gravity.

11.2. At the completion of the bulk specific gravity test (G_{mb}), determine the relative density ($\%G_{mmx}$) at any point in the compaction process as follows:

$$\%G_{mmx} = \frac{G_{mb} h_m}{G_{mm} h_x} \times 100 \qquad (3)$$

where:

$\%G_{mmx}$	=	corrected relative density expressed as a percent of the maximum theoretical specific gravity;
G_{mb}	=	bulk specific gravity of the extruded specimen;
h_m	=	height in millimeters of the extruded specimen; and
h_x	=	height in millimeters of the specimen after x gyrations.

12. REPORT

12.1. Report the following information in the compaction report, if applicable:

12.1.1. Project name

12.1.2. Date of the test;

12.1.3. Start time of the test;

12.1.4. Specimen identification;

12.1.5. Percent binder in specimen, nearest 0.1 percent;

12.1.6. Average diameter of the mold used (d), nearest 1.0 mm;

12.1.7. Mass of the specimen (W_m), nearest 0.1 g;

12.1.8. Maximum specific gravity (G_{mm}) of the specimen by T 209, nearest 0.001;

12.1.9. Bulk specific gravity (G_{mb}) of the specimen by T 166 or T 275, nearest 0.001;

12.1.10. Height of the specimen after each gyration (h_x), nearest 0.1 mm;

12.1.11. Relative density ($\%G_{mm}$) expressed as a percent of the theoretical maximum specific gravity (G_{mm}), nearest 0.1 percent; and

12.1.12. Gyration angle, nearest 0.2 mrad (0.01°), and the method used to determine or verify the gyration angle.

13. PRECISION AND BIAS

13.1. *Precision:*

13.1.1. *Single Operator Precision*—The single operator standard deviations (1s limits) for relative densities at N_{ini} and N_{des} for mixtures containing aggregate with an absorption of less than 1.5 percent are shown in Table 1. The results of two properly conducted tests on the same material, by the same operator, using the same equipment, should be considered suspect if they differ by more than the d2s single operator limits shown in Table 1.

13.1.2. *Multilaboratory Precision*—The multilaboratory standard deviations (1s limits) for relative densities at N_{ini} and N_{des} for mixtures containing aggregate with an absorption of less than 1.5 percent are shown in Table 1. The results of two properly conducted tests on the same material, by different operators, using different equipment, should be considered suspect if they differ by more than the d2s multilaboratory limits shown in Table 1.

Table 1—Precision Estimates[a]

	1s limit Relative Density (%)	d2s limit Relative Density (%)
Single Operator Precision:		
12.5-mm nominal max. agg.	0.3	0.9
19.0-mm nominal max. agg.	0.5	1.4
Multilaboratory Precision:		
12.5-mm nominal max. agg.	0.6	1.7
19.0-mm nominal max. agg.	0.6	1.7

[a] Based on an interlaboratory study described in NCHRP Research Report 9-26 involving 150-mm diameter specimens with 4–5 percent air voids, 26 laboratories, two materials (a 12.5-mm mixture and a 19.0-mm mixture), and three replicates. Specimens were prepared in accordance with T 312-04. The angle of gyration was verified using Method A, external angle.

13.2. *Bias*—No information can be presented on the bias of the procedure because no material having an accepted reference value is available.

Standard Practice for

Mixture Conditioning of Hot-Mix Asphalt (HMA)

AASHTO Designation: R 30-02[1]

1. SCOPE

1.1. This practice describes procedures for mixture conditioning of compacted and uncompacted hot-mix asphalt (HMA). Three types of conditioning are described: (1) mixture conditioning for volumetric mixture design; (2) short-term conditioning for mixture mechanical property testing (both which simulate the precompaction phase of the construction process); and (3) long-term conditioning for mixture mechanical property testing to simulate the aging that occurs over the service life of a pavement. The long-term conditioning for mixture mechanical property testing procedures are preceded by the procedure for short-term conditioning for mixture mechanical property testing.

1.2. *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- PP 3, Preparing Hot-Mix Asphalt (HMA) Specimens by Means of the Rolling Wheel Compactor[2]
- T 312, Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor
- T 316, Viscosity Determination of Asphalt Binder using Rotational Viscometer

3. SUMMARY OF PRACTICE

For mixture conditioning for volumetric mixture design, a mixture of aggregate and binder is conditioned in a forced-draft oven for 2 h at the mixture's specified compaction temperature. For short-term mixture conditioning for mechanical property testing, a mixture of aggregate and binder is conditioned in a forced-draft oven for 4 h at 135°C. For long-term mixture conditioning for mechanical property testing, a compacted mixture of aggregate and binder is conditioned in a forced-draft oven for five days at 85°C.

4. SIGNIFICANCE AND USE

The properties and performance of HMA can be more accurately predicted by using conditioned test samples. The mixture conditioning for the volumetric mixture design procedure is designed to allow for binder absorption during the mixture design. The short-term mixture conditioning for the mechanical property testing procedure is designed to simulate the plant-mixing and construction effects on the mixture. The long-term mixture conditioning for the mechanical property testing procedure is designed to simulate the aging the compacted mixture will undergo during seven to ten years of service.

5. APPARATUS

5.1. *Oven*—A forced-draft oven, thermostatically controlled, capable of maintaining any desired temperature setting from room temperature to 176°C within ±3°C.

5.2. *Thermometers*—Thermometers having a range from 50°C to 260°C and readable to 1°C.

5.3. *Miscellaneous*—A metal pan for heating aggregates, a shallow metal pan for heating uncompacted HMA, a metal spatula or spoon, timer, and gloves for handling hot equipment.

6. HAZARDS

6.1. This standard involves the handling of hot binder, aggregate, and HMA, which can cause severe burns if allowed to contact skin. Follow standard safety precautions to avoid burns.

7. MIXTURE CONDITIONING PROCEDURES

7.1. *Mixture Conditioning for Volumetric Mixture Design*:

7.1.1. The mixture conditioning for the volumetric mixture design procedure applies to laboratory-prepared, loose mixture only. No mixture conditioning is required when conducting quality control or quality assurance testing on plant-produced mixture.

Note 1—The Agency may identify the need to condition plant-produced mixture to be more representative of field conditions, particularly where absorptive aggregates are used.

7.1.2. Place the mixture in a pan, and spread it to an even thickness ranging between 25 and 50 mm. Place the mixture and pan in a forced-draft oven for 2 h ± 5 minutes at a temperature equal to the mixture's compaction temperature ±3°C. The compaction temperature range of a HMA mixture is defined as the range of temperatures where the unaged binder has a kinematic viscosity of 280 ± 30 mm^2/s (approximately 0.28 ± 0.03 Pa·s) measured in accordance with T 316 (Note 2). The target compaction temperature is generally the mid-point of this range.

Note 2—Modified binders may not adhere to the equi-viscosity requirements noted. The agency should consider the manufacturer's recommendations when establishing the mixing and compaction temperatures for modified binders. Practically, the mixing temperature should not exceed 165°C and the compaction temperature should not be lower than 115°C.

7.1.3. Stir the mixture after 60 ± 5 minutes to maintain uniform conditioning.

7.1.4. After 2 h ± 5 minutes, remove the mixture from the forced-draft oven. The conditioned mixture is now ready for compaction or testing.

7.2. *Short-Term Conditioning for Mixture Mechanical Property Testing*:

7.2.1. The short-term conditioning for the mixture mechanical property testing procedure applies to laboratory-prepared, loose mix only.

7.2.2. Place the mixture in a pan, and spread it to an even thickness ranging between 25 and 50 mm. Place the mixture and pan in the conditioning oven for 4 h ± 5 minutes at a temperature of 135 ± 3°C.

7.2.3. Stir the mixture every 60 ± 5 minutes to maintain uniform conditioning.

7.2.4. After 4 h ± 5 minutes, remove the mixture from the forced-draft oven. The conditioned mixture is now ready for further conditioning or testing as required.

7.3. *Long-Term Conditioning for Mixture Mechanical Property Testing*:

7.3.1. The long-term conditioning for the mixture mechanical property testing procedure applies to laboratory-prepared mixtures that have been subjected to the short-term conditioning for the mixture mechanical property testing procedure described in Section 7.2, plant-mixed HMA, and compacted roadway specimens.

7.3.2. *Preparing Specimens from Loose HMA*:

7.3.2.1. Specimens Compacted Using the Superpave Gyratory Compactor:

7.3.2.1.1. Compact the specimens in accordance with T 312. Cool the test specimen at room temperature for 16 ± 1 h.

Note 3—Extrude the specimen from the compaction mold after cooling for 2 to 3 h.

Note 4—Specimen cooling is usually scheduled as an overnight step. Cooling may be accelerated by placing the specimen in front of a fan.

7.3.2.2. Specimens Compacted Using the Rolling Wheel Compactor:

7.3.2.2.1. Compact the specimens in accordance with PP 3.

7.3.2.2.2. Cool the test specimen at room temperature for 16 ± 1 h.

7.3.2.2.3. Remove the slab from the mold, and saw or core the required specimens from the slab.

7.3.3. *Preparing Compacted Roadway Specimens*:

7.3.3.1. Cool test specimens at room temperature for 16 ± 1 h.

7.3.4. *Long-Term Conditioning of Prepared Test Specimens*—Place the compacted test specimens in the conditioning oven for 120 ± 0.5 h at a temperature of 85 ± 3°C.

7.3.5. After 120 ± 0.5 h, turn the oven off; open the doors, and allow the test specimen to cool to room temperature. Do not touch or remove the specimen until it has cooled to room temperature.

Note 5—Cooling to room temperature will take approximately 16 h.

7.3.6. After cooling to room temperature, remove the test specimen from the oven. The long-term-conditioned specimen is now ready for testing as required.

8. REPORT

8.1. Report the binder grade, binder content (nearest 0.1 percent), and the aggregate type and gradation, if applicable.

8.2. Report the following mixture conditioning information for the volumetric mixture design conditions, if applicable:

8.2.1. Mixture conditioning temperature in laboratory (compaction temperature, nearest 1°C),

8.2.2. Mixture conditioning duration in laboratory (nearest minute), and

8.2.3. Laboratory compaction temperature (nearest 1°C).

8.3. Report the following short-term conditioning information for the mixture mechanical property testing conditions, if applicable:

8.3.1. Short-term mixture conditioning temperature in laboratory (nearest 1°C),

8.3.2. Short-term mixture conditioning duration in laboratory (nearest minute), and

8.3.3. Laboratory compaction temperature (nearest 1°C).

8.4. Report the following long-term conditioning information for the mixture mechanical property testing conditions, if applicable:

8.4.1. Laboratory compaction temperature (nearest 1°C),

8.4.2. Long-term mixture conditioning temperature in laboratory (nearest 1°C), and

8.4.3. Long-term mixture conditioning duration in laboratory (nearest five minutes).

9. KEYWORDS

9.1. Conditioning; hot-mix asphalt; long-term conditioning; short-term conditioning.

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[1] This standard is based on SHRP Product 1031.
[2] PP 3-94 was last printed in the May 2002 Edition of the *AASHTO Provisional Standards*.

Standard Practice for

Superpave Volumetric Design for Hot-Mix Asphalt (HMA)

AASHTO Designation: R 35-04

1. SCOPE

1.1. This standard for mix design evaluation uses aggregate and mixture properties to produce a hot-mix asphalt (HMA) job-mix formula. The mix design is based on the volumetric properties of the HMA in terms of the air voids, voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA).

1.2. This standard may also be used to provide a preliminary selection of mix parameters as a starting point for mix analysis and performance prediction analyses that primarily use T 320 and T 322.

1.3. *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this procedure to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- M 320, Performance-Graded Asphalt Binder
- M 323, Superpave Volumetric Mix Design
- R 30, Mixture Conditioning of Hot-Mix Asphalt (HMA)
- T 2, Sampling of Aggregates
- T 11, Materials Finer Than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing
- T 27, Sieve Analysis of Fine and Coarse Aggregates
- T 84, Specific Gravity and Absorption of Fine Aggregate
- T 85, Specific Gravity and Absorption of Coarse Aggregate
- T 100, Specific Gravity of Soils
- T 166, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens
- T 209, Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
- T 228, Specific Gravity of Semi-Solid Bituminous Materials
- T 248, Reducing Samples of Aggregate to Testing Size
- T 275, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens

- T 283, Resistance of Compacted Asphalt Mixture to Moisture-Induced Damage
- T 312, Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor
- T 320, Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester (SST)
- T 322, Determining the Creep Compliance and Strength of Hot-Mix Asphalt (HMA) Using the Indirect Tensile Test Device

2.2. *Asphalt Institute Standards*:

- MS-2, Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types

3. TERMINOLOGY

3.1. *HMA*—hot-mix asphalt.

3.2. *design ESALs*—Design equivalent (80 kN) single-axle loads.

3.2.1. *Discussion*—Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. For pavements designed for more or less than 20 years, determine the design ESALs for 20 years when using this standard.

3.3. *air voids* (V_a)—The total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as a percent of the bulk volume of the compacted paving mixture (Note 1).

Note 1—Term defined in Asphalt Institute Manual MS-2, Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types.

3.4. *voids in the mineral aggregate* (VMA)—The volume of the intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective binder content, expressed as a percent of the total volume of the specimen (Note 1).

3.5. *absorbed binder volume* (V_{ba})—The volume of binder absorbed into the aggregate (equal to the difference in aggregate volume when calculated with the bulk specific gravity and effective specific gravity).

3.6. *binder content* (P_b)—The percent by mass of binder in the total mixture including binder and aggregate.

3.7. *effective binder volume* (V_{be})—The volume of binder which is not absorbed into the aggregate.

3.8. *voids filled with asphalt* (VFA)—The percentage of the VMA filled with binder (the effective binder volume divided by the VMA).

3.9. *dust-to-binder ratio* ($P_{0.075}/P_{be}$)—By mass, the ratio between the percent passing the 75-µm (No. 200) sieve ($P_{0.075}$) and the effective binder content (P_{be}).

3.10. *nominal maximum aggregate size*—One size larger than the first sieve that retains more than 10 percent aggregate (Note 2).

3.11. *maximum aggregate size*—One size larger than the nominal maximum aggregate size (Note 2).

Note 2—The definitions given in Sections 3.10 and 3.11 apply to Superpave mixes only and differ from the definitions published in other AASHTO standards.

3.12. *reclaimed asphalt pavement* (RAP)—Removed and/or processed pavement materials containing asphalt binder and aggregate.

3.13. *primary control sieve* (PCS)—The sieve defining the break point between fine and coarse-graded mixtures for each nominal maximum aggregate size.

4. SUMMARY OF THE PRACTICE

4.1. *Materials Selection*—Binder, aggregate and RAP stockpiles are selected that meet the environmental and traffic requirements applicable to the paving project. The bulk specific gravity of all aggregates proposed for blending and the specific gravity of the binder are determined.

Note 3—If RAP is used, the bulk specific gravity of the RAP aggregate may be estimated by determining the theoretical maximum specific gravity (G_{mm}) of the RAP mixture and using an assumed asphalt absorption for the RAP aggregate to back-calculate the RAP aggregate bulk specific gravity, if the absorption can be estimated with confidence. The RAP aggregate effective specific gravity may be used in lieu of the bulk specific gravity at the discretion of the Agency. The use of the effective specific gravity may introduce an error into the combined aggregate bulk specific gravity and subsequent VMA calculations. The Agency may choose to specify adjustments to the VMA requirements to account for this error based on experience with local aggregates.

4.2. *Design Aggregate Structure*—It is recommended at least three trial aggregate blend gradations from selected aggregate stockpiles are blended. For each trial gradation, an initial trial binder content is determined, and at least two specimens are compacted in accordance with T 312. A design aggregate structure and an estimated design binder content are selected on the basis of satisfactory conformance of a trial gradation meeting the requirements given in M 323 for V_a, VMA, VFA, dust-to-binder ratio at N_{design}, and relative density at $N_{initial}$.

Note 4—Previous Superpave mix design experience with specific aggregate blends may eliminate the need for three trial blends.

4.3. *Design Binder Content Selection*—Replicate specimens are compacted in accordance with T 312 at the estimated design binder content and at the estimated design binder content ±0.5 percent and +1.0 percent. The design binder content is selected on the basis of satisfactory conformance with the requirements of M 323 for V_a, VMA, VFA, and dust-to-binder ratio at N_{design}, and the relative density at $N_{initial}$ and N_{max}.

4.4. *Evaluating Moisture Susceptibility*—The moisture susceptibility of the design aggregate structure is evaluated at the design binder content: the mixture is conditioned according to the mixture conditioning for the volumetric mixture design procedure in R 30, compacted to 7.0 ± 0.5 percent air voids in accordance with T 312, and evaluated according to T 283. The design shall meet the tensile strength ratio requirement of M 323.

5. SIGNIFICANCE AND USE

5.1. The procedure described in this practice is used to produce HMA which satisfies Superpave HMA volumetric mix design requirements.

6. PREPARING AGGREGATE TRIAL BLEND GRADATIONS

6.1. Select a binder in accordance with the requirements of M 323.

6.2. Determine the specific gravity of the binder according to T 228.

6.3. Obtain samples of aggregates proposed to be used for the project from the aggregate stockpiles in accordance with T 2.

Note 5—Each stockpile usually contains a given size of an aggregate fraction. Most projects employ three to five stockpiles to generate a combined gradation conforming to the job-mix formula and M 323.

6.4. Reduce the samples of aggregate fractions according to T 248 to samples of the size specified in T 27.

6.5. Wash and grade each aggregate sample according to T 11 and T 27.

6.6. Determine the bulk and apparent specific gravity for each coarse and fine aggregate fraction in accordance with T 85 and T 84, respectively, and determine the specific gravity of the mineral filler in accordance with T 100.

6.7. Blend the aggregate fractions using Equation 1:

$$P = Aa + Bb + Cc, \text{ etc.} \tag{1}$$

where:

P	=	Percentage of material passing a given sieve for the combined aggregates A, B, C, etc.;
A, B, C, etc.	=	Percentage of material passing a given sieve for aggregates A, B, C, etc.; and
a, b, c, etc.	=	Proportions of aggregates A, B, C, etc. used in the combination, and where the total = 1.00.

6.8. Prepare a minimum of three trial aggregate blend gradations; plot the gradation of each trial blend on a 0.45-power gradation analysis chart, and confirm that each trial blend meets M 323 gradation controls (see Table 3 of M 323). Gradation control is based on four control sieve sizes: the sieve for the maximum aggregate size, the sieve for the nominal maximum aggregate size, the 4.75- or 2.36-mm sieve, and the 0.075-mm sieve. An example of three acceptable trial blends in the form of a gradation plot is given in Figure 1.

6.9. Obtain a test specimen from each of the trial blends according to T 248, and conduct the quality tests specified in Section 6 of M 323 to confirm that the aggregate in the trial blends meets the minimum quality requirements specified in M 323.

Note 6—The designer has an option of performing the quality tests on each stockpile instead of the trial aggregate blend. The test results from each stockpile can be used to estimate the results for a given combination of materials.

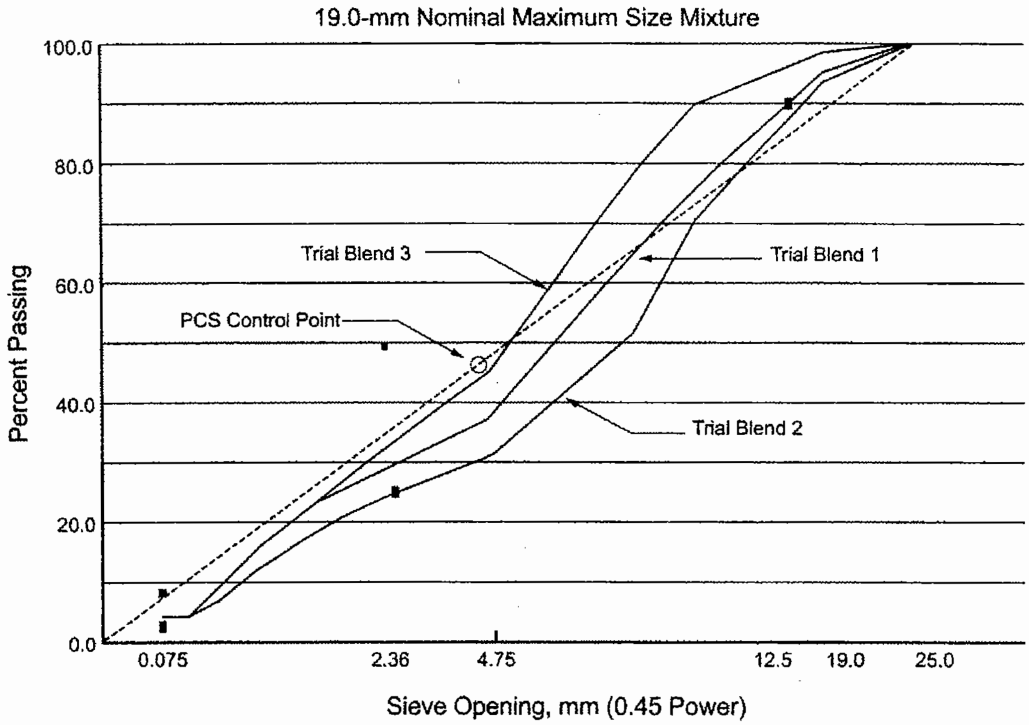


Figure 1—Evaluation of the Gradations of Three Trial Blends (Example)

7. DETERMINING AN INITIAL TRIAL BINDER CONTENT FOR EACH TRIAL AGGREGATE GRADATION

7.1. Designers can either use their experience with the materials or the procedure given in Appendix A1 to determine an initial trial binder content for each trial aggregate blend gradation.

Note 7—When using RAP, the initial trial asphalt content should be reduced by an amount equal to that provided by the RAP.

8. COMPACTING SPECIMENS OF EACH TRIAL GRADATION

8.1. Prepare replicate mixtures (Note 8) at the initial trial binder content for each of the chosen trial aggregate trial blend gradations. From Table 1, determine the number of gyrations based on the design ESALs for the project.

Note 8—At least two replicate specimens are required, but three or more may be prepared if desired. Generally, 4500 to 4700 g of aggregate is sufficient for each compacted specimen with a height of 110 to 120 mm for aggregates with combined bulk specific gravities of 2.55 to 2.70, respectively.

ile
to

8.2. Condition the mixtures according to R 30, and compact the specimens to N_{design} gyrations in accordance with T 312. Record the specimen height to the nearest 0.1 mm after each revolution.

8.3. Determine the bulk specific gravity (G_{mb}) of each of the compacted specimens in accordance with T 166 or T 275 as appropriate.

Table 1—Superpave Gyratory Compaction Effort

Design ESALs[a] (million)	Compaction Parameters			Typical Roadway Application[b]
	$N_{initial}$	N_{design}	N_{max}	
< 0.3	6	50	75	Applications 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 may also be applicable to this level.
0.3 to < 3	7	75	115	Applications include many collector roads or access streets. Medium-trafficked city streets and the majority of county roadways may be applicable to this level.
3 to < 30	8	100	160	Applications include many two-lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium to highly trafficked city streets, many state routes, U.S. highways, and some rural Interstates.
≥ 30	9	125	205	Applications include the vast majority of the U.S. Interstate system, both rural and urban in nature. Special applications such as truck-weighing stations or truck-climbing lanes on two-lane roadways may also be applicable to this level.

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

[b] As defined by *A Policy on Geometric Design of Highways and Streets*, 2004, AASHTO.

Note 9—When specified by the Agency and the top of the design layer is ≥100 mm from the pavement surface and the estimated design traffic level is ≥0.3 million ESALs, decrease the estimated design traffic level by one, unless the mixture will be exposed to significant mainline construction traffic prior to being overlaid. If less than 25 percent of a construction lift is within 100 mm of the surface, the lift may be considered to be below 100 mm for mixture design purposes.

Note 10—When the estimated design traffic level is between 3 and <10 million ESALs, the Agency may, at its discretion, specify $N_{initial}$ at 7, N_{design} at 75, and N_{max} at 115.

8.4. Determine the theoretical maximum specific gravity (G_{mm}) according to T 209 of separate samples representing each of these combinations that have been mixed and conditioned to the same extent as the compacted specimens.

Note 11—The maximum specific gravity for each trial mixture shall be based on the average of at least two tests.

9. EVALUATING COMPACTED TRIAL MIXTURES

9.1. Determine the volumetric requirements for the trial mixtures in accordance with M 323.

9.2. Calculate V_a and VMA at N_{design} for each trial mixture using Equations 2 and 3:

$$V_a = 100 \times \left(1 - \left(\frac{G_{mb}}{G_{mm}}\right)\right) \quad (2)$$

$$\text{VMA} = 100 \times \left(1 - \frac{G_{mb} P_s}{G_{sb}}\right) \quad (3)$$

where:

G_{mb} = bulk specific gravity of the extruded specimen;
G_{mm} = theoretical maximum specific gravity of the mixture;
P_s = percent of aggregate in the mix; and
G_{sb} = bulk specific gravity of the combined aggregate.

Note 12—Although the initial trial binder content was estimated for a design air void content of 4.0 percent, the actual air void content of the compacted specimen is unlikely to be exactly 4.0 percent. Therefore, the change in binder content needed to obtain a 4.0 percent air void content, and the change in VMA caused by this change in binder content, is estimated. These calculations permit the evaluation of VMA and VFA of each trial aggregate gradation at the same design air void content, 4.0 percent.

9.3. Estimate the volumetric properties at 4.0 percent air voids for each compacted specimen.

9.3.1. Determine the difference in average air void content at N_{design} (ΔV_a) of each aggregate trial blend from the design level of 4.0 percent using Equation 4:

$$\Delta V_a = 4.0 - V_a \quad (4)$$

where:

V_a = air void content of the aggregate trial blend at N_{design} gyrations.

9.3.2. Estimate the change in binder content (ΔP_b) needed to change the air void content to 4.0 percent using Equation 5:

$$\Delta P_b = -0.4(\Delta V_a) \quad (5)$$

9.3.3. Estimate the change in VMA (ΔVMA) caused by the change in the air void content (ΔV_a) determined in Section 9.3.1 for each trial aggregate blend gradation, using Equation 6 or 7.

$$\Delta\text{VMA} = 0.2(\Delta V_a) \text{ if } V_a > 4.0 \quad (6)$$

$$\Delta\text{VMA} = -0.1(\Delta V_a) \text{ if } V_a < 4.0 \quad (7)$$

Note 13—A change in binder content affects the VMA through a change in the bulk specific gravity of the compacted specimen (G_{mb}).

9.3.4. Calculate the VMA for each aggregate trial blend at N_{design} gyrations and 4.0 percent air voids using Equation 8:

$$\text{VMA}_{design} = \text{VMA}_{trial} + \Delta\text{VMA} \quad (8)$$

where:

VMA_{design} = VMA estimated at a design air void content of 4.0 percent; and
VMA_{trial} = VMA determined at the initial trial binder content.

9.3.5. Using the values of ΔV_a determined in Section 9.3.1 and Equation 9, estimate the relative density of each specimen at N_{initial} when the design air void content is adjusted to 4.0 percent at N_{design}:

$$\%G_{mm_{\text{initial}}} = 100 \times \left(\frac{G_{mb} h_d}{G_{mm} h_i}\right) - \Delta V_a \qquad (9)$$

where:

$\%G_{mm_{\text{initial}}}$	=	relative density at N_{initial} gyrations at the adjusted design binder content;
h_d	=	height of the specimen after N_{design} gyrations, from the Superpave gyratory compactor, mm; and
h_i	=	height of the specimen after N_{initial} gyrations, from the Superpave gyratory compactor, mm.

9.3.6. Estimate the percent of effective binder ($P_{be_{est}}$) and calculate the dust-to-binder ratio ($P_{0.075}/P_{be}$) for each trial blend using Equations 10 and 11:

$$P_{be_{est}} = -(P_s \times G_b)\frac{(G_{se} - G_{sb})}{(G_{se} \times G_{sb})} + P_{b_{est}} \qquad (10)$$

where:

$P_{be_{est}}$	=	estimated effective binder content,
P_s	=	aggregate content,
G_b	=	specific gravity of the binder,
G_{se}	=	effective specific gravity of the aggregate,
G_{sb}	=	bulk specific gravity of the combined aggregate, and
$P_{b_{est}}$	=	estimated binder content.

$$P_{0.075}/P_{be} = \frac{P_{0.075}}{P_{be_{est}}} \qquad (11)$$

where:

$P_{0.075}$	=	percent passing the 0.075-mm sieve.

9.3.7. Compare the estimated volumetric properties from each trial aggregate blend gradation at the adjusted design binder content with the criteria specified in M 323. Choose the trial aggregate blend gradation that best satisfies the volumetric criteria.

Note 14—Table 2 presents an example of the selection of a design aggregate structure from three trial aggregate blend gradations.

Note 15—Many trial aggregate blend gradations will fail the VMA criterion. Generally, the $\%G_{mm_{\text{initial}}}$ criterion will be met if the VMA criterion is satisfied. Section 12.1 gives a procedure for the adjustment of VMA.

Note 16—If the trial aggregate gradations have been chosen to cover the entire range of the gradation controls, then the only remaining solution is to make adjustments to the aggregate production or to introduce aggregates from a new source. The aggregates that fail to meet the required criteria will not produce a quality mix and should not be used. One or more of the aggregate stockpiles should be replaced with another material which produces a stronger structure. For example, a quarry stone can replace a crushed gravel, or crushed fines can replace natural fines.

Table 2—Selection of a Design Aggregate Structure (Example)

Volumetric Property	Trial Mixture (19.0-mm Nominal Maximum Aggregate) 20-Year Project Design ESALs = 5 million			Criteria
	1	2	3	
	At the Initial Trial Binder Content			
P_b (trial)	4.4	4.4	4.4	
% $G_{mm_{initial}}$ (trial)	88.3	88.0	87.3	
% $G_{mm_{design}}$ (trial)	95.6	94.9	94.5	
V_a at N_{design}	4.4	5.1	5.5	4.0
VMA_{trial}	13.0	13.6	14.1	
	Adjustments to Reach Design Binder Content (V_a = 4.0% at N_{design})			
ΔV_a	−0.4	−1.1	−1.5	
ΔP_b	0.2	0.4	0.6	
ΔVMA	−0.1	−0.2	−0.3	
	At the Estimated Design Binder Content (V_a = 4.0 % at N_{design})			
Estimated P_b (design)	4.6	4.8	5.0	
VMA (design)	12.9	13.4	13.8	≥ 13.0
% $G_{mm_{initial}}$ (design)	88.7	89.1	88.5	≤ 89.0

Notes:
1. The top portion of this table presents measured densities and volumetric properties for specimens prepared for each aggregate trial blend at the initial trial binder content.
2. None of the specimens had an air void content of exactly 4.0 percent. Therefore, the procedures described in Section 9 must be applied to: (1) estimate the design binder content at which V_a = 4.0 percent, and (2) obtain adjusted VMA and relative density values at this estimated binder content.
3. The middle portion of this table presents the change in binder content (ΔP_b) and VMA (ΔVMA) that occurs when the air void content (V_a) is adjusted to 4.0 percent for each trial aggregate blend gradation.
4. A comparison of the VMA and densities at the estimated design binder content to the criteria in the last column shows that trial aggregate blend gradation No. 1 does not have sufficient VMA (12.9 percent versus a requirement of ≥13.0 percent). Trial blend No. 2 exceeds the criterion for relative density at $N_{initial}$ gyrations (89.1 percent versus a requirement of ≤89.0 percent). Trial blend No. 3 meets the requirement for relative density and VMA and; in this example, is selected as the design aggregate structure.

10. SELECTING THE DESIGN BINDER CONTENT

10.1. Prepare replicate mixtures (Note 8) containing the selected design aggregate structure at each of the following four binder contents: (1) the estimated design binder content, P_b (design); (2) 0.5 percent below P_b (design); (3) 0.5 percent above P_b (design); and (4) 1.0 percent above P_b (design).

10.1.1. Use the number of gyrations previously determined in Section 8.1.

10.2. Condition the mixtures according to R 30, and compact the specimens to N_{design} gyrations according to T 312. Record the specimen height to the nearest 0.1 mm after each revolution.

10.3. Determine the bulk specific gravity of each of the compacted specimens in accordance with T 166 or T 275 as appropriate.

10.4. Determine the theoretical maximum specific gravity (G_{mm}) according to T 209 of each of the four mixtures using companion samples which have been conditioned to the same extent as the compacted specimens (Note 11).

10.5. Determine the design binder content which produces a target air void content (V_a) of 4.0 percent at N_{design} gyrations using the following steps:

10.5.1. Calculate V_a, VMA, and VFA at N_{design} using Equations 2, 3, and 12:

$$VFA = 100 \times \left[\frac{VMA - V_a}{VMA}\right] \quad (12)$$

10.5.2. Calculate the dust-to-binder ratio using Equation 13.

$$P_{0.075} \Big/ P_{be} = \frac{P_{0.075}}{P_{be}} \quad (13)$$

where:

P_{be} = effective binder content.

10.5.3. For each of the four mixtures, determine the average corrected specimen relative densities at $N_{initial}$ ($\%G_{mm_{initial}}$), using Equation 14.

$$\%G_{mm_{initial}} = 100 \times \left(\frac{G_{mb} h_d}{G_{mm} h_i}\right) \quad (14)$$

10.5.4. Plot the average V_a, VMA, VFA, and relative density at N_{design} for replicate specimens versus binder content.

Note 17—All plots are generated automatically by the Superpave software. Figure 2 presents a sample data set and the associated plots.

10.5.5. By graphical or mathematical interpolation (Figure 2), determine the binder content to the nearest 0.1 percent at which the target V_a is equal to 4.0 percent. This is the design binder content (P_b) at N_{design}.

10.5.6. By interpolation (Figure 2), verify that the volumetric requirements specified in M 323 are met at the design binder content.

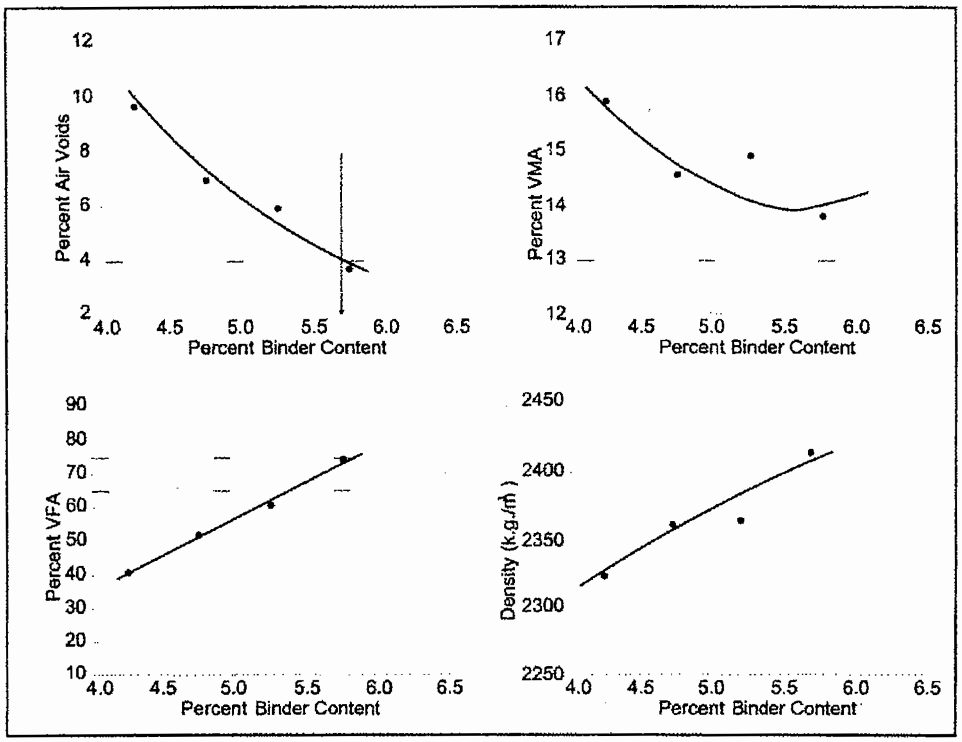
10.6. Compare the calculated percent of maximum relative density with the design criteria at $N_{initial}$ by interpolation, if necessary. This interpolation can be accomplished by the following procedure.

10.6.1. Prepare a densification curve for each mixture by plotting the measured relative density at X gyrations, $\%G_{mm_x}$, versus the logarithm of the number of gyrations (see Figure 3).

10.6.2. Examine a plot of air void content versus binder content. Determine the difference in air voids between 4.0 percent and the air void content at the nearest, lower binder content.

Determine the air void content at the nearest, lower binder content at its data point, not on the line of best fit. Designate the difference in air void content as ΔV_a.

10.6.3. Using Equation 14, determine the average corrected specimen relative densities at $N_{initial}$ ($\%G_{mm_{initial}}$). Confirm that $\%G_{mm_{initial}}$ satisfies the design requirements in M 323 at the design binder content.



Average V_a, VMA, VFA, and Relative Density at N_{design}

P_b (%)	V_a (%)	VMA (%)	VFA (%)	Density at N_{design} (kg/m^3)
4.3	9.5	15.9	40.3	2320
4.8	7.0	14.7	52.4	2366
5.3	6.0	14.9	59.5	2372
5.8	3.7	13.9	73.5	2412

Notes: 1. In this example, the estimated design binder content is 4.8 percent; the minimum VMA requirement for the design aggregate structure (19.0-mm nominal maximum size) is 13.0 percent, and the VFA requirement is 65 to 75 percent.

2. Entering the plot of percent air voids versus percent binder content at 4.0 percent air voids, the design binder content is determined as 5.7 percent.

3. Entering the plots of percent VMA versus percent binder content and percent VFA versus percent binder content at 5.7 percent binder content, the mix meets the VMA and VFA requirements.

Figure 2—Sample Volumetric Design Data at N_{design}

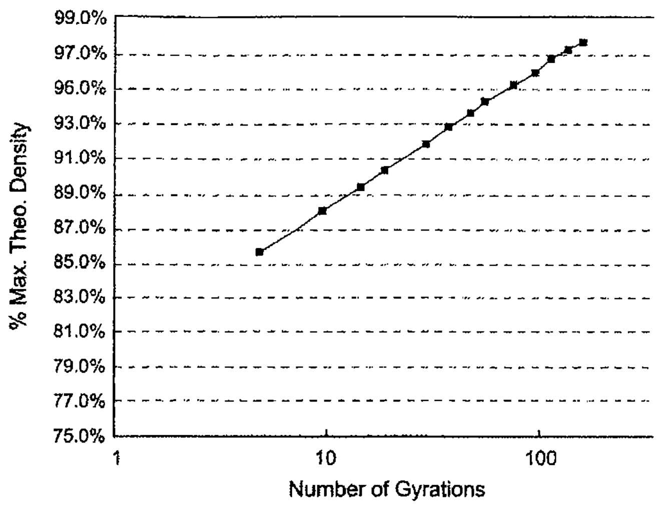


Figure 3—Sample Densification Curve

10.7. Prepare replicate (Note 8) specimens composed of the design aggregate structure at the design binder content to confirm that $\%G_{mm_{max}}$ satisfies the design requirements in M 323.

10.7.1. Condition the mixtures according to R 30, and compact the specimens according to T 312 to the maximum number of gyrations, N_{max}, from Table 1.

10.7.2. Determine the average specimen relative density at N_{max}, $\%G_{mm_{max}}$, by using Equation 15, and confirm that $\%G_{mm_{max}}$ satisfies the volumetric requirement in M 323.

$$\%G_{mm_{max}} = 100\frac{G_{mb}}{G_{mm}} \tag{15}$$

where:

$\%G_{mm_{max}}$ = relative density at N_{max} gyrations at the design binder content.

11. EVALUATING MOISTURE SUSCEPTIBILITY

11.1. Prepare six mixture specimens (nine are needed if freeze-thaw testing is required) composed of the design aggregate structure at the design binder content. Condition the mixtures in accordance with R 30, and compact the specimens to 7.0 ± 0.5 percent air voids in accordance with T 312.

11.2. Test the specimens and calculate the tensile strength ratio in accordance with T 283.

11.3. If the tensile strength ratio is less than 0.80, as required in M 323, remedial action such as the use of anti-strip agents is required to improve the moisture susceptibility of the mix.

When remedial agents are used to modify the binder, retest the mix to assure compliance with the 0.80 minimum requirement.

12. ADJUSTING THE MIXTURE TO MEET PROPERTIES

12.1. *Adjusting VMA*—If a change in the design aggregate skeleton is required to meet the specified VMA, there are three likely options: (1) change the gradation (Note 18); (2) reduce the minus 0.075-mm fraction (Note 19); or (3) change the surface texture and/or shape of one or more of the aggregate fractions (Note 20).

Note 18—Changing gradation may not be an option if the trial aggregate blend gradation analysis includes the full spectrum of the gradation control area.

Note 19—Reducing the percent passing the 0.075-mm sieve of the mix will typically increase the VMA. If the percent passing the 0.075-mm sieve is already low, this is not a viable option.

Note 20—This option will require further processing of existing materials or a change in aggregate sources.

12.2. *Adjusting VFA*—The lower limit of the VFA range should always be met at 4.0 percent air voids if the VMA meets the requirements. If the upper limit of the VFA is exceeded, then the VMA is substantially above the minimum required. If so, redesign the mixture to reduce the VMA. Actions to consider for redesign include: (1) changing to a gradation that is closer to the maximum density line; (2) increasing the minus 0.075-mm fraction, if room is available within the specification control points; or (3) changing the surface texture and shape of the aggregates by incorporating material with better packing characteristics, e.g., less thin, elongated aggregate particles.

12.3. *Adjusting the Tensile Strength Ratio*—The tensile strength ratio can be increased by: (1) adding chemical anti-strip agents to the binder to promote adhesion in the presence of water; or (2) adding hydrated lime to the mix.

13. REPORT

13.1. The report shall include the identification of the project number, traffic level, and mix design number.

13.2. The report shall include information on the design aggregate structure including the source of aggregate, kind of aggregate, required quality characteristics, and gradation.

13.3. The report shall contain information about the design binder including the source of binder and the performance grade.

13.4. The report shall contain information about the HMA including the percent of binder in the mix; the relative density; the number of initial, design, and maximum gyrations; and the VMA, VFA, V_{be}, V_{ba}, V_a, and dust-to-binder ratio.

14. KEYWORDS

14.1. HMA mix design; Superpave; volumetric mix design.

APPENDIX

(Nonmandatory Information)

A1. CALCULATING AN INITIAL TRIAL BINDER CONTENT FOR EACH AGGREGATE TRIAL BLEND

A1.1. Calculate the bulk and apparent specific gravities of the combined aggregate in each trial blend using the specific gravity data for the aggregate fractions obtained in Section 6.6 and Equations 16 and 17:

$$G_{sb} = \frac{P_1 + P_2 + \cdots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \cdots + \frac{P_n}{G_n}} \tag{16}$$

$$G_{sa} = \frac{P_1 + P_2 + \cdots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \cdots + \frac{P_n}{G_n}} \tag{17}$$

where:

G_{sb}	=	bulk specific gravity for the combined aggregate;
G_{sa}	=	apparent specific gravity for the combined aggregate;
P_1, P_2, P_n	=	percentages by mass of aggregates 1, 2, n; and
G_1, G_2, G_n	=	bulk specific gravities (Equation 16) or apparent specific gravities (Equation 17) of aggregates 1, 2, n.

A1.2. Estimate the effective specific gravity of the combined aggregate in the aggregate trial blend using Equation 18:

$$G_{se} = G_{sb} + 0.8\left(G_{sa} - G_{sb}\right) \tag{18}$$

where:

G_{se}	=	effective specific gravity of the combined aggregate;
G_{sb}	=	bulk specific gravity of the combined aggregate; and
G_{sa}	=	apparent specific gravity of the combined aggregate.

Note 21—The multiplier, 0.8, can be changed at the discretion of the designer. Absorptive aggregates may require values closer to 0.6 or 0.5.

Note 22—The Superpave mix design system includes a mixture conditioning step before the compaction of all specimens; this conditioning generally permits binder absorption to proceed to completion. Therefore, the effective specific gravity of Superpave mixtures will tend to be close to the apparent specific gravity in contrast to other design methods where the effective specific gravity generally will lie near the midpoint between the bulk and apparent specific gravities.

A1.3. Estimate the volume of binder absorbed into the aggregate, V_{ba}, using Equations 19 and 20:

$$V_{ba} = W_s\left(\frac{1}{G_{sb}} - \frac{1}{G_{se}}\right) \quad (19)$$

where:

W_s, the mass of aggregate in 1 cm^3 of mix, g, is calculated as:

$$W_s = \frac{P_s(1-V_a)}{\frac{P_b}{G_b} + \frac{P_s}{G_{se}}} \quad (20)$$

and where:

P_b = mass percent of binder, in decimal equivalent, assumed to be 0.05;
P_s = mass percent of aggregate, in decimal equivalent, assumed to be 0.95;
G_b = specific gravity of the binder; and
V_a = volume of air voids, assumed to be 0.04 cm^3 in 1 cm^3 of mix.

Note 23—This estimate calculates the volume of binder absorbed into the aggregate, V_{ba}, and subsequently, the initial, trial binder content at a target air void content of 4.0 percent.

A1.4. Estimate the volume of effective binder using Equation 21:

$$V_{be} = 0.176 - [\,0.0675 \log(S_n)] \quad (21)$$

where:

V_{be} = volume of effective binder, cm^3; and
S_n = nominal maximum sieve size of the largest aggregate in the aggregate trial blend, mm.

Note 24—This regression Equation is derived from an empirical relationship between: (1) VMA and V_{be} when the air void content, V_a, is equal to 4.0 percent: V_{be} = VMA – V_a = VMA – 4.0; and (2) the relationship between VMA and the nominal maximum sieve size of the aggregate in M 323.

A1.5. Calculate the estimated initial trial binder (P_{bi}) content for the aggregate trial blend gradation using Equation 22:

$$P_{bi} = 100 \times \left(\frac{G_b(V_{be}+V_{ba})}{(G_b(V_{be}+V_{ba}))+W_s}\right) \quad (22)$$

where:

P_{bi} = estimated initial trial binder content, percent by weight of total mix.

NCHRP

REPORT 464

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

The Restricted Zone in the Superpave Aggregate Gradation Specification

TRANSPORTATION RESEARCH BOARD

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 464

The Restricted Zone in the Superpave Aggregate Gradation Specification

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SUBJECT AREAS

Materials and Construction

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

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FOREWORD

By Staff
Transportation Research Board

This report presents the findings of a research project to determine whether the restricted zone requirement is necessary for aggregate gradations designed in accordance with AASHTO MP2 and PP28 if mix volumetric and fine aggregate angularity criteria are met. Its main finding is that, based on an evaluation of the performance properties of hot mix asphalt, the restricted zone requirement is redundant in these circumstances. The report will be of particular interest to materials engineers in state highway agencies, as well as to materials suppliers and paving contractor personnel responsible for the specification and production of hot mix asphalt.

In developing the Superpave mix design method, the Asphalt Research Program (1987–1993) of the Strategic Highway Research Program (SHRP) primarily targeted the properties of asphalt binders and hot mix asphalt (HMA) and their effects on pavement performance. Other than asphalt-aggregate adhesion and its consequences to moisture damage, the study of the aggregate's contribution to pavement performance was purposefully excluded from the program. Yet, SHRP researchers were required to produce an aggregate gradation specification without the benefit of experimentation to support or verify its formulation.

In lieu of a formal research program, a group of acknowledged experts in the areas of aggregate production and behavior and HMA mix design developed, through the use of a modified Delphi approach, the set of recommended aggregate properties and criteria that appeared in the original Superpave mix design method. These criteria included a restricted zone in the gradation; the zone lies along the maximum density line between the intermediate size (either 4.75 or 2.36 mm, depending on the nominal maximum size of the aggregate) and the 300-m size and forms a band through which it usually was considered undesirable for a gradation to pass. The original intention of including a restricted zone, which particularly affects (1) the use of natural sands that may be rounded or have a limited size distribution and (2) the allowable ratio of the fine sand fraction (150 to 600 m) to the total sand (passing 2.36 mm), was to help reduce the incidence of tender or rutting-prone HMA. Although the restricted zone was presented in the Superpave mix design method as a guideline, it often has been implemented by specifying agencies as a requirement for the design of acceptable HMA.

In the experience of many agency engineers and materials suppliers, however, it has been found that compliance with the restricted zone criterion was neither desirable nor necessary in every instance to produce well-performing HMA mix designs. For example, when aggregate particles in the size range of the restricted zone are highly angular (i.e., have high fine aggregate angularity [FAA] values), it is likely that high-quality, rut-resistant, nontender paving mixes can be produced regardless of whether the gradation passes through the restricted zone. Furthermore, there are many known examples of aggregate gradations passing through the restricted zone that produce well-performing HMA.

Under NCHRP Project 9-14, "Investigation of the Restricted Zone in the Superpave Aggregate Gradation Specification," the National Center for Asphalt Technology at Auburn University was assigned the task of determining under what conditions, if any, compliance with the restricted zone requirement is necessary when an HMA mix design meets all other Superpave mix volumetric and FAA criteria for a paving project. The research team (1) conducted a literature search and critical review of the use and effectiveness of the restricted zone and (2) carried out a program of laboratory testing to determine the impact of the restricted zone requirement on HMA performance.

The three-part laboratory testing program compared the performance of HMA mix designs measured with three independent mechanical property tests: the Asphalt Pavement Analyzer, a laboratory wheel-tracking device; the repeated load confined creep test; and the repeated shear at constant height test. The testing program included the following experimental factors:

- A PG 64-22 asphalt binder;
- Two coarse aggregates—a crushed granite and a crushed gravel;
- Ten fine aggregates with FAA values between 38 and 50;
- Nominal maximum aggregate sizes of 9.5 and 19 mm;
- Compaction levels of 75, 100, and 125 gyrations; and
- Five gradation types—above, below, and through the restricted zone (ARZ, BRZ, and TRZ); humped through the restricted zone (HRZ); and crossover through the restricted zone (CRZ).

With a few exceptions requested by the project panel and described in the report, performance testing was only conducted on HMA mix designs that met all Superpave mix design criteria, except the restricted zone requirement.

The research team found that HMA mixes meeting Superpave mix volumetric and FAA requirements with gradations passing through the restricted zone performed similarly to or better than mixes with gradations passing outside the restricted zone. The team concluded that the restricted zone requirement is not necessary to ensure satisfactory performance when all other relevant Superpave design requirements are met, and it recommended changes to AASHTO MP2 to implement this finding.

This final report includes a detailed description of the experimental program, a discussion of the research results, and five supporting appendixes:

- Appendix A: Review of Literature Relevant to the Restricted Zone;
- Appendix B: Compacted Aggregate Resistance Test;
- Appendix C: Volumetric Mix Design and Performance Data for Part 1;
- Appendix D: Volumetric Mix Design and Performance Data for Part 2; and
- Appendix E: Volumetric Mix Design and Performance Data for Part 3.

The entire final report will also be distributed as a CD-ROM (*CRP-CD-10*) along with task and final reports for NCHRP Projects 9-10 and 9-19. The research results have been referred to the TRB Mixtures and Aggregate Expert Task Group for its review and possible recommendation to the AASHTO Highway Subcommittee on Materials for revision of the applicable specifications and recommended practices.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 9-14, "Investigation of the Restricted Zone in the Superpave Aggregate Gradation Specification," by the National Center for Asphalt Technology (NCAT), Auburn University, Alabama. This work was carried out under the direction of Prithvi S. Kandhal, associate director of NCAT and member of the graduate faculty of Civil Engineering, Auburn University, who served as the principal investigator. L. Allen Cooley, Jr., served as the research engineer for this project.

THE RESTRICTED ZONE IN THE SUPERPAVE AGGREGATE GRADATION SPECIFICATION

SUMMARY

The aggregate specification for Superpave® hot-mix asphalt (HMA) mixtures includes a restricted zone that lies along the maximum density gradation between the intermediate size (i.e., either 4.75 or 2.36 mm, depending on the nominal maximum size of the aggregate) and the 0.3-mm size. The restricted zone forms a band through which gradations were recommended not to pass. The restricted zone requirement was adopted in Superpave to reduce the incidence of tender or rut-prone HMA mixes. Although the restricted zone was included in Superpave as a recommended guideline and not as a required specification, some highway agencies interpret it as a requirement.

According to many asphalt paving technologists, compliance with the restricted zone criteria may not be desirable or necessary to produce paving mixes that give good performance in terms of rutting. Some highway agencies and suppliers can provide examples of aggregate gradations that pass through the restricted zone, but produce paving mixes that have performed well.

This research project was undertaken to evaluate the effect of the Superpave restricted zone on permanent deformation of dense-graded HMA mixtures on the basis of a statistically planned and properly controlled laboratory experiment. The project's primary objective was to determine under what conditions, if any, compliance with the restricted zone requirement is necessary when HMA meets all other Superpave requirements such as fine aggregate angularity (FAA) and volumetric mix criteria for the specific project.

The following factors were evaluated: two coarse aggregates, ten fine aggregates, two nominal maximum size mixes (i.e., 9.5 and 19.0 mm), five aggregate gradations, and three compactive efforts (i.e., N_{design} = 75, 100, and 125). Of the five gradations used, three pass through the restricted zone and two (i.e., the control group) fall outside of the restricted zone. Permanent deformation characteristics of mixes meeting Superpave volumetric requirements were evaluated by two different types of tests: empirical and fundamental. For the empirical test, the Asphalt Pavement Analyzer was used. The Superpave shear tester and a repeated load confined creep test were used as fundamental tests. Test results from the three mechanical tests were analyzed statistically to evaluate the effect of the five gradations on permanent deformation of the HMA mixtures.

Mixes meeting Superpave and FAA requirements with gradations that violated the restricted zone performed similarly to or better than the mixes having gradations passing outside the restricted zone; therefore, the restricted zone requirement is redundant for mixes meeting all Superpave volumetric parameters and the required FAA. It has been recommended to delete references to the restricted zone as either a requirement or a guideline from the AASHTO specification (AASHTO MP2) and practice (AASHTO PP28) for Superpave volumetric mix design.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT AND RESEARCH OBJECTIVE

The Strategic Highway Research Program's (SHRP's) asphalt research was aimed at the properties of asphalt binders and paving mixes and their effect on asphalt pavement performance. The study of aggregate properties (including gradation) was intentionally excluded from the asphalt research program. Yet, the SHRP researchers had to recommend a set of aggregate properties and an aggregate gradation specification without the benefit of experimentation so that a comprehensive Superpave mix design system could be formulated.

SHRP formed an Aggregate Expert Task Group (ETG) consisting of 14 acknowledged aggregate experts. In lieu of a formal aggregate research program, the Aggregate ETG used a modified Delphi approach to develop a set of *recommended* aggregate properties and criteria that are now included in the Superpave volumetric mix design method (AASHTO MP2 and PP28). The Delphi process was conducted with five rounds of questionnaires. The final recommended aggregate gradation criteria included control points between which the gradation must fall, as well as a restricted zone that lies along the maximum density line (MDL) between the intermediate size (i.e., either 4.75 or 2.36 mm, depending on the nominal maximum size of the aggregate in the mix) and the 0.3-mm size.

Although the restricted zone was included in Superpave as a recommended guideline and not as a required specification, some highway agencies have interpreted it as a requirement. Many asphalt technologists believe that compliance with the restricted zone criteria may not be desirable or necessary in every case to produce asphalt mixes with good performance. If highly angular aggregates are used in the mix, it is likely that the mix will not exhibit any tenderness during construction and will be rut-resistant under traffic regardless of whether its gradation passes through the restricted zone. The Georgia Department of Transportation (DOT) has used such mixes successfully for many years. Some asphalt technologists also question the need for the restricted zone when the mix has to meet volumetric properties such as minimum voids in the mineral aggregate (VMA) and specified air void contents at N_{initial}, N_{design}, and N_{maximum} gyrations.

This research was carried out to evaluate the effect of restricted zone on mix performance on the basis of a statistically planned and properly controlled experiment. The research's primary objective was to determine under what conditions, if any, compliance with the restricted zone requirement is necessary when the hot-mix asphalt (HMA) meets all other Superpave requirements such as fine aggregate angularity (FAA) and volumetric mix criteria for the specific project.

SCOPE OF STUDY

The following tasks were conducted in two phases to accomplish the objective of this study.

Phase I

The tasks in Phase I were as follows:

- **Task 1:** Conduct a literature search and review of information relevant to the basis, use, and effect of the restricted zone.
- **Task 2:** Select materials (i.e., coarse aggregates, fine aggregates, and asphalt binder) for use in this study. A wide range of material properties should be evaluated.
- **Task 3:** Develop a research plan that utilizes a laboratory investigation to determine under what conditions, if any, the restricted zone requirement is necessary to ensure satisfactory HMA performance.
- **Task 4:** Prepare an interim report that documents the work accomplished in Tasks 1 through 3 and provides the detailed work plan for Phase II.

Phase II

The tasks in Phase II were as follows:

- **Task 5:** Execute the research plan approved in Phase I. Analyze data and draw conclusions based on test results.
- **Task 6:** Develop a recommended experimental plan and budget for a separate project to extend the analysis to other traffic levels and mixture types. (This additional work has been accomplished and is part of this final report.)

- **Task 7:** Submit a final report that documents the entire research effort. The report will include a plan for extending the results of this study and an implementation plan for moving the research results into practice.

RESEARCH APPROACH

The research approach for this project included reviewing literature relevant to the restricted zone (see Appendix A), selecting a variety of coarse and fine aggregates of different mineralogical compositions and angularities, conducting Superpave volumetric mix designs using gradations both conforming to and violating the restricted zone, conducting performance tests on mixtures meeting Superpave volumetric and FAA criteria, and analyzing the relative performance of mixes to determine whether the restricted zone requirement is necessary in Superpave for ensuring better performance.

CHAPTER 2

EXPERIMENTAL PLAN

SELECTION OF MATERIALS

Materials needed for this study consisted of coarse aggregates, fine aggregates, and an asphalt binder. Two coarse aggregates, ten fine aggregates, and one asphalt binder were selected. The descriptions of the materials selected for this study along with properties of the selected materials follows.

Coarse Aggregates

Two coarse aggregates were used. Selection criteria for these two coarse aggregates were that they should come from different mineralogical types and have different angularities and surface textures. These criteria were selected to ensure that the coarse aggregates gave a range of properties. Selected coarse aggregates were a crushed granite and a crushed gravel. The crushed gravel is predominately composed of quartz. Both of these sources were used in NCHRP Project 4-19, "Aggregate Tests Related to Asphalt Concrete Performance in Pavements." Properties of these two coarse aggregates are provided in Table 1.

Fine Aggregates

Because the restricted zone is applied within the fine aggregate sieve sizes, the shape and texture of the fine aggregates are the most important factors affecting the performance of HMA mixtures; therefore, the approach taken in identifying and selecting fine aggregates for use in this study was to select aggregates with varying values of FAA. Also included within the selection criteria were mineralogical composition of the fine aggregates and type of crusher. Maximization of these three criteria ensured using fine aggregates with a wide range of properties.

During the identification process, aggregates that have been or are being used in controlled field pavement performance studies were included. Field studies considered included FHWA WesTrack, ICAR (at the International Center for Aggregate Research), Pooled Fund Study No. 176 at Purdue, and MnRoad.

A large database of FAA values was compiled to select the nine fine aggregates for this study. This database included fine aggregates from Mississippi, Alabama, Georgia, Illinois, Minnesota, Virginia, Tennessee, Nevada, California, Louisiana, North Carolina, Indiana, and Iowa. FAA values within this database ranged from a low of 38 to a high of 52.

The 10 selected fine aggregates, along with their mineralogical type and FAA value (AASHTO T304), are provided in Table 2. Six different mineralogical types were selected and include natural sands, sandstone, dolomite, limestone, granite, and diabase (i.e., traprock). FAA values of the ten fine aggregates ranged from 38.6 to 50.3.

FA-10 was included in this study based upon recommendations from the project panel. This fine aggregate purposely had a FAA value below 40 (i.e., FAA = 38.6). FA-10 was included to provide a "worst-case" reference point for comparing the response variables described later in this report.

As can be seen from Table 2, a wide range of FAA values was selected. As indicated in the approved work plan, three compactive efforts were used during this study. These three compactive efforts included medium, high, and very high. The Superpave FAA requirement for the high and very high compactive efforts is 45 percent voids. For the medium compactive effort, the FAA requirement is 40 percent voids. Because two of the three compactive efforts used in this study require a minimum FAA value of 45, approximately two-thirds (i.e., six) of the fine aggregates shown in Table 2 meet a FAA value of 45.

Additional testing on each fine aggregate is presented in Table 3. This table presents the results of specific gravity (AASHTO T84), sand equivalency (AASHTO T176), and adherent fines testing. The procedure used to measure the percent of adherent fines was a modified version of ASTM D5711. This procedure calls for testing of aggregates larger than 4.75 mm. Since the fine aggregates were the materials in question for this study, ASTM D5711 was followed except testing was conducted on aggregates passing the 4.75-mm (No. 4) sieve and retained on the 0.075-mm (No. 200) sieve.

Table 3 shows that a wide range of physical properties was selected. Apparent specific gravities ranged from 2.614 to 2.973 while bulk specific gravities ranged from 2.568 to 2.909. All but three fine aggregates had water absorption values less than 1.0 percent. The highest absorption value was 1.7 percent for FA-8. An interesting observation from Table 3 is that the sand equivalency and percent adherent fines values appear to be related. Generally, as the adherent fines values increased, sand equivalency values decreased.

TABLE 1 Coarse-aggregate properties

Test	Procedure	Crushed Gravel	Granite
Flat or Elongated 2:1	ASTM D4791	20	57
Flat or Elongated 3:1	ASTM D4791	2	11
Flat or Elongated 5:1	ASTM D4791	0	1
Flat and Elongated 2:1	ASTM D4791	40.1	64.3
Flat and Elongated 5:1	ASTM D4791	0	1.0
Uncompacted Voids (Method A)	AASHTO TP56	41.7	47.0
Apparent Specific Gravity	AASHTO T84	2.642	2.724
Bulk Specific Gravity	AASHTO T85	2.591	2.675
Water Absorption, %	AASHTO T85	0.7	0.6
Los Angeles Abrasion, % loss	AASHTO T96	28	41
Coarse Aggregate Angularity % 1 Fractured Face, % 2 Fractured Faces	ASTM D5821	100/92	100/100

TABLE 2 Fine aggregates selected for study

Fine Aggregate	FAA Value	Mineralogical Type	Comments
FA-1	40.7	River Sand	Washed, uncrushed, river deposit comprised of predominantly quartz, from Kentucky
FA-2	42.6	Quartz Sand	No processing, natural quartz river deposit with some chert, from Tennessee
FA-3	44.1	Natural Sand	Uncrushed, natural quartz sand with some chert, from Alabama
FA-4	49.7	Sandstone	Mined, cone crusher, from Alabama
FA-5	50.3	Dolomite	Mined from Alabama
FA-6	46.9	Limestone	Mined, same source as FA-8 but crushed by impact crusher, from Alabama
FA-7	48.9	Granite	Mined, cone crusher, from Minnesota, used on MnRoad
FA-8	48.3	Limestone	Mined, same source as FA-6 but crushed by cone crusher, from Alabama
FA-9	50.1	Diabase	Mined, impact crusher, from Virginia
FA-10	38.6	Natural Sand	Dredged stream deposit from Mississippi

TABLE 3 Physical properties of fine aggregates

Fine Aggregate	Apparent Specific Gravity	Bulk Specific Gravity	% Absorption	Sand Equivalency, %	Adherent Fines, %
FA-1	2.614	2.610	0.2	100	0.1
FA-2	2.665	2.568	1.4	98	0.3
FA-3	2.664	2.638	0.4	95	0.2
FA-4	2.789	2.731	0.8	29	10.4
FA-5	2.856	2.822	0.5	61	3.2
FA-6	2.737	2.661	1.0	91	2.4
FA-7	2.742	2.711	0.4	94	0.6
FA-8	2.777	2.648	1.7	100	2.1
FA-9	2.973	2.909	0.8	59	7.5
FA-10	2.653	2.636	0.3	100	0.1

In addition to the testing outlined in Tables 2 and 3, the compacted aggregate resistance (CAR) test was also conducted. This test involves compacting the fine aggregate sample in Marshall mold, testing its shear resistance by penetrating a 1.5-in. (38-mm) diameter round bar with the Marshall stability machine, and reading the peak load. The CAR test is not a standard test, so the method is provided in Appendix B. Figures 1 and 2 present the CAR results.

Results of the CAR test appear to relate with the FAA results. Generally, as FAA values increased, the peak loads from the CAR test also increased. It is interesting to note that the four uncrushed natural sands (i.e., FA-1, FA-2, FA-3, and FA-10) all had the lowest peak loads in the CAR test. However, FA-7, with an FAA value of 48.9, also gave relatively lower peak load in the CAR test.

Asphalt Binder

The asphalt binder selected was a Superpave performance-based PG 64-22, which is one of the most commonly used grades in the United States. This binder is one of the National Center for Asphalt Technology (NCAT) labstock asphalt binders and has been used successfully on numerous research projects. Properties of this asphalt binder are provided in Table 4.

EXPERIMENTAL PLAN

Based on the review of literature (see Appendix A) and properties of the selected materials, a statistically based, controlled laboratory experimental plan was developed with the objective of determining under what conditions, if any, the restricted zone requirement is necessary to ensure satisfactory HMA performance when the FAA and the Superpave mixture volumetric criteria are met.

The literature review identified a number of variables with potential for inclusion in the experimental plan: crushed versus uncrushed fine aggregates, compactive efforts during mix design, volumetric properties, FAA values, and nominal maximum aggregate size for gradations.

To achieve the primary objective of this study, a number of gradations using different aggregate types (i.e., coarse and fine aggregates) were tried for mix design. These consisted of gradations that both met and did not meet the restricted zone criteria. These mixes were prepared at optimum asphalt content and tested by performance-related, mechanical test methods. Also, because the literature review suggested that the effect of the restricted zone on mix performance is different for aggregates with different particle shape, angularity, and surface texture, the experiment included a set of aggregates with a significant range of shape and texture properties (i.e., FAA values).

The overall research approach is shown in Figure 3. This figure illustrates that the research effort was broken into three parts to maximize the information obtained. During Part 1, variables included within the research were two coarse aggregates, ten fine aggregates, one nominal maximum aggregate size (NMAS), five gradations, one asphalt binder, and one compactive effort with the Superpave gyratory compactor (SGC). Based on the results of Part 1, Part 2 involved a

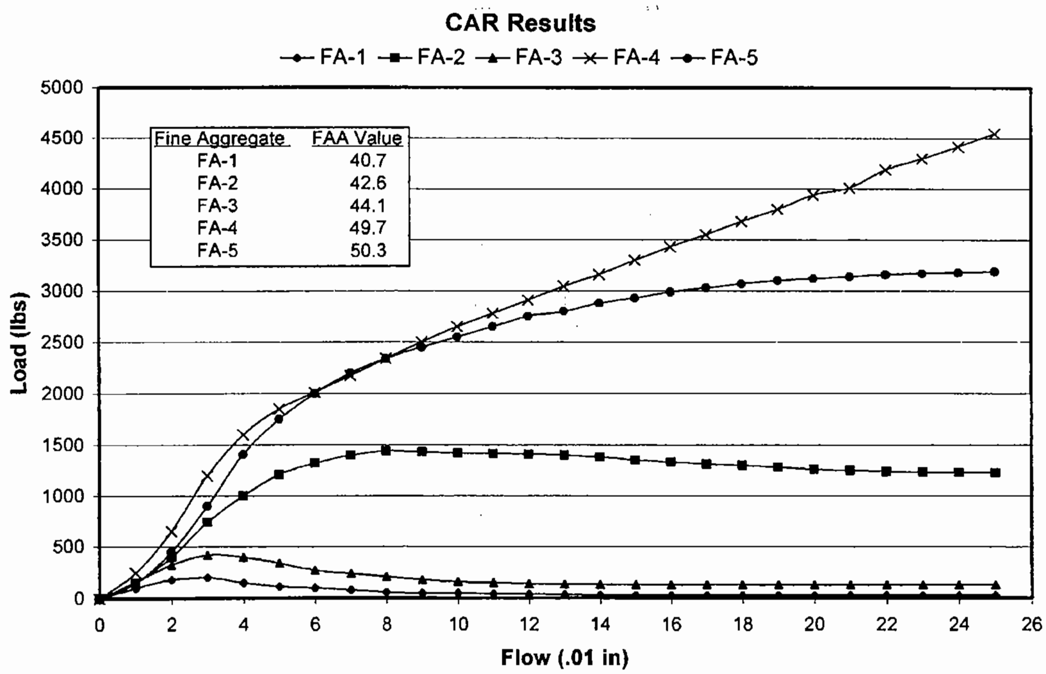

Figure 1. Results of CAR test for fine aggregates FA-1 through FA-5.

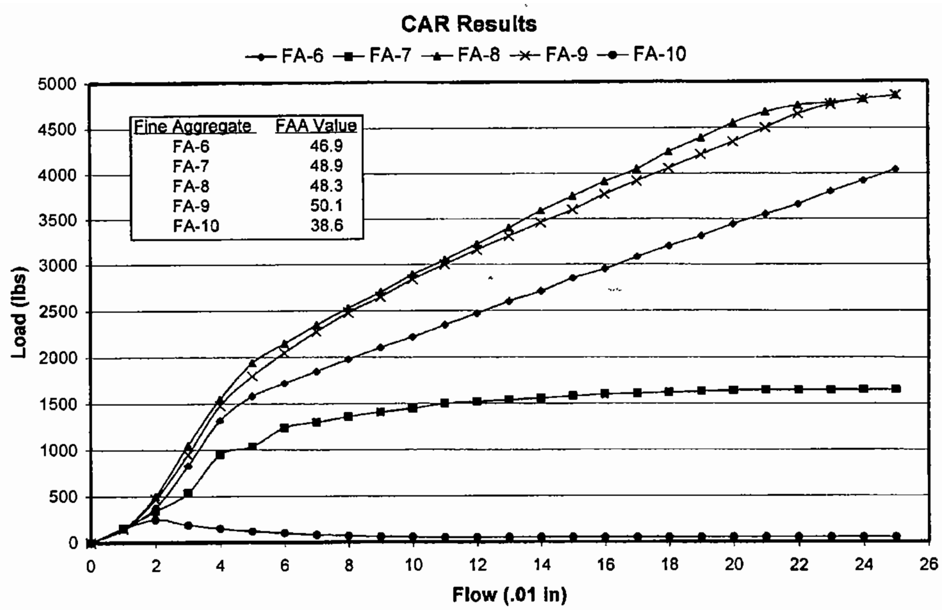

Figure 2. Results of CAR test for fine aggregates FA-6 through FA-10.

TABLE 4 Properties of asphalt binder

Test	Temperature (°C)	Test Result	Requirement
Unaged DSR, $G^*/\sin\delta$ (kPa)	64	1.85	1.00 min
RTFO-Aged DSR, $G^*/\sin\delta$ (kPa)	64	3.83	2.20 min
PAV-Aged DSR, $G^*\sin\delta$ (kPa)	25	4063	5000 max
PAV-Aged BBR, Stiffness (MPa)	−12	244	300 max
PAV-Aged BBR, m-value	−12	0.301	0.300 min

NOTE: DSR = dynamic shear rheometer;
RTFO = rolling thin film oven;
PAV = pressure aging vessel;
BBR = bending beam rheometer.

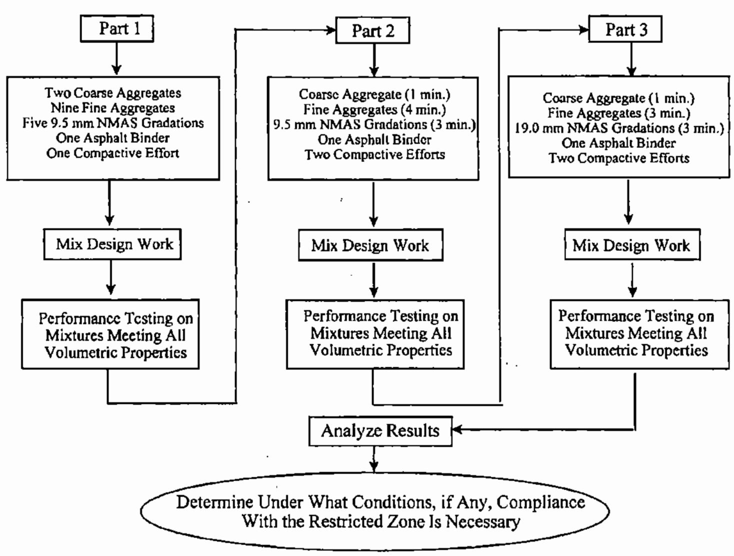

Figure 3. Overall research approach.

critical coarse aggregate (sensitive to the effect of different fine aggregates on HMA performance properties), critical fine aggregates (sensitive to the effect of different gradations on HMA performance properties), and critical gradations for the same NMAS (showing the most significant effect on HMA performance properties) combined with the same asphalt binder and designed using two different compactive efforts with the SGC. In Part 3, the coarse aggregate, fine aggregates, gradations (different NMAS), and compactive effort were based on results from Parts 1 and 2. The detailed work plans for the three parts are described as follows.

Part 1 Work Plan

The work plan for Part 1 is illustrated in Figure 4. Factor-level combinations included in Part 1 consisted of two coarse aggregates, ten fine aggregates, five 9.5-mm NMAS gradations, and one compactive effort. Of the five gradations used in Part 1, three violated the restricted zone (VRZ) while two resided outside the restricted zone (i.e., the control group). These five gradations are given in Table 5 and illustrated in Figure 5. The compactive effort used during Part 1 was that for a 20-year design traffic level of 3 to 30 million equivalent single axle loads (ESALs). The initial, design, and maximum

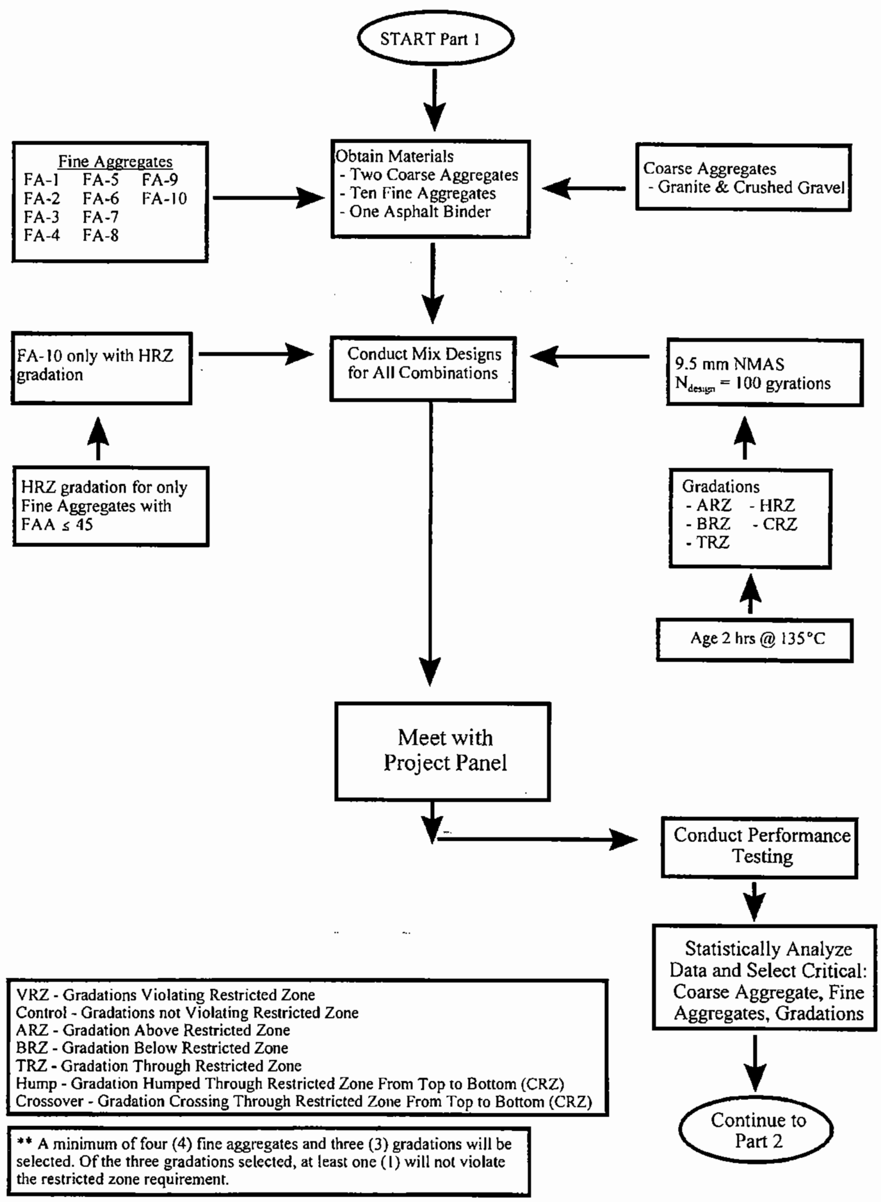

Figure 4. Research approach for Part 1.

TABLE 5 9.5-smm NMAS gradations used in Parts 1 and 2

Sieve, mm	BRZ	ARZ	TRZ	HRZ	CRZ
12.5	100	100	100	100	100
9.5	95	95	95	95	95
4.75	60	60	60	60	60
2.36	42	50	46	46	52
1.18	28	42	34	34	34
0.60	18	32	24	30	20
0.30	14	22	18	24	14
0.15	10	10	10	10	10
0.075	5	5	5	5	5

number of gyrations for this design traffic level are 8, 100, and 160, respectively (see Table 6).

As seen in Figure 5, all five gradations follow the same trend from the 12.5-mm sieve down to the 4.75-mm sieve. From the 4.75-mm sieve, the BRZ (below the restricted zone) gradation passes below the restricted zone and above the lower control points. The ARZ (above the restricted zone) gradation passes above the restricted zone and below the upper control points. These two gradations are designated the control gradations because they do not violate the Superpave restricted zone. Figure 5 shows that the remaining three gradations do violate the restricted zone. From the 4.75-mm sieve, the TRZ (through the restricted zone) gradation passes almost directly along the MDL. The HRZ (humped through the restricted zone) gradation follows a similar gradation as the TRZ gradation down to the 1.18-mm sieve where it humps on the 0.6- and 0.3-mm sieves and represents gradations generally containing a large percentage of natural, windblown sands. From the 4.75-mm sieve, the CRZ (crossover through the restricted zone) gradation begins above the restricted zone on the 2.36-mm sieve but then crosses through the restricted zone between the 0.6- and 0.3-mm sieves. The CRZ gradation represents gradations that are not continuously graded between 2.36-mm and 0.60-mm sizes and generally exhibit low mix stability. All five of the gradations then meet at the 0.15-mm sieve and follow the same trend down to the 0.075-mm sieve. A common material passing the 0.075-mm sieve (No. 200) sieve (P200) was used in all HMA mixtures to eliminate P200 as a variable. Different P200 materials stiffen the asphalt binder and HMA mix-

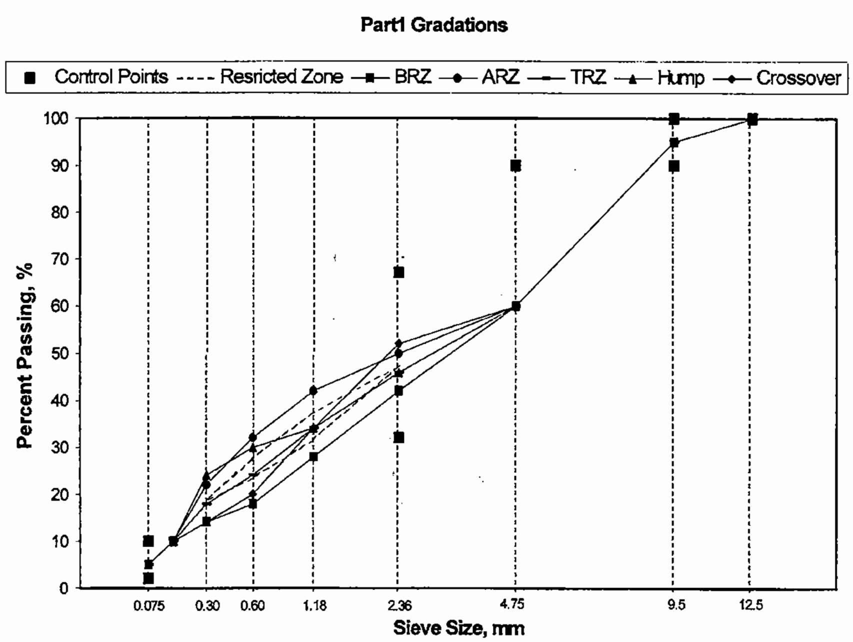

Figure 5. Part 1 gradations.

TABLE 6 Superpave design compactive effort and aggregate consensus property requirements

Estimated Design Traffic Level (Million ESALs)[1]	Superpave Compaction Parameters[2]			%G_{mm} $N_{initial}$ Requirement	Aggregate Consensus Properties					
					Coarse Aggregate Angularity[3]		*Fine Aggregate Angularity*[4]		*Sand Equivalent Value*[5]	Flat and Elongated[6]
	$N_{initial}$	N_{design}[7]	$N_{maximum}$		*≤ 100 mm*	*> 100 mm*	*≤ 100 mm*	*> 100 mm*	*All Mixtures*	All Mixtures
< 0.3	6	50	75	≤ 91.5	55/–	–/–[9]	–[9]	–[9]	40	–[9]
0.3 – 3	7	75	115	≤ 90.5	75/–	50/–	40	40	40	< 10 %
3 – 10	8	100[8]	160	≤ 89.0	85/80	60/–	45	40	45	< 10 %
10 – 30	8	100[8]	160	≤ 89.0	95/90[5]	80/75	45	40	45	< 10 %
>30.0	9	125	212	≤ 89.0	100/100	100/100	45	45	50	< 10 %

[1] Values shown are based upon 20-year equivalent single axle loads (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.
[2] It is recommended that Superpave mixtures be compacted to N_{design} gyrations.
[3] "85/80" denotes that 85% of the coarse aggregate has one fractured face and 80% has two or more fractured faces.
[4] Criteria are minimum presented as percent air voids in loosely compacted fine aggregate. Test is to be run in accordance with AASHTO TP-33.
[5] No distinction is made between depth from surface. Test is to be run in accordance with AASHTO T176.
[6] Criterion based upon a 5:1 maximum-to-minimum ratio.
[7] (a) N_{design} compactive effort is for typical traffic speeds. For slow/standing traffic, increase N_{design} by one (1) traffic level or increase high-temperature binder grade by one. (No changes in aggregate properties with increased compactive effort and do not exceed N_{design} of 125 gyrations.)
(b) For pavement layers where the top of the design layer is more than 100-mm below the surface, decrease the compactive effort by one level, but not less than N_{design} of 50 gyrations.
[8] 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 lower compaction level (75 gyrations).
[9] Dash means no requirement.

tures to a different degree and, therefore, affect the mix performance test results. A limestone filler (which has a Rigden voids value of 33.5 percent) was utilized as the P200.

Based on Figure 4, factor-level combinations were designed using an SGC (N_{design} = 100 gyrations). In accordance with recommendations by the project panel, FA-10 was combined with the two coarse aggregates only for the HRZ gradation. The project panel also recommended not combining fine aggregates having an FAA value greater than 45 with the HRZ gradation because the HRZ gradation is indicative of gradations having a large percentage of natural rounded sand. Natural rounded sands very rarely have FAA values greater than 45. It was therefore deemed unnecessary to evaluate HRZ gradations with fine aggregates having FAA values greater than 45.

Part 2 Work Plan

The work plan for Part 2 was very similar to that of Part 1, with two major differences: (1) fewer factor-level combinations and (2) two different compactive efforts. The factor-level combinations included were one critical coarse aggregate (i.e., granite), three 9.5-mm NMAS gradations (i.e., BRZ, TRZ, and CRZ), and two compactive efforts. The BRZ gradation was included as the control gradation. For Part 2, the two compactive efforts were equal to the medium and very high traffic levels from Table 6 (i.e., N_{design} = 75 and 125 gyrations, respectively). Based upon the Part 1 mix design data and guidance from the project panel, seven fine aggregates were investigated in Part 2. For the lower compactive effort (i.e., N_{design} = 75), mix designs were conducted for FA-2, FA-3, FA-4, FA-6, FA-7, and FA-10. For the higher compactive effort (i.e., N_{design} = 125), mix designs were conducted for FA-4, FA-7, FA-9, and FA-10. Similar to Part 1, FA-10 was only used with the HRZ gradation.

Mix designs were conducted for all combinations of fine aggregate, gradation, and compactive effort. Performance testing was then accomplished on those mixtures meeting all volumetric requirements.

For the lower compactive effort experiment (i.e., N_{design} = 75), humped gradations (i.e., HRZ) were included for the fine aggregates having a FAA value less than 45.0 (FA-2 and FA-3). Realistically, the potential for using natural sands (which have low FAA values) is greatest for low-volume roadways. Additionally, when natural sands are incorporated into an aggregate gradation, there is a higher potential for humped gradations.

Similar to the Part 1 work, a mix design and performance testing using FA-10, granite coarse aggregate, HRZ gradation, and 75-gyration design level were conducted. This information was used as a baseline against which to compare other results.

Part 3 Work Plan

The primary objective of Part 3 was to extend the Part 1 and Part 2 research results to 19.0-mm NMAS gradations. During Parts 1 and 2, only 9.5-mm NMAS gradations were used.

Figure 6 presents the experimental plan for Part 3. This figure shows that two compactive efforts were used: 75 and 100 gyrations. Within the lower compactive effort experiment (i.e., $N_{design} = 75$), a gravel coarse aggregate was used because preliminary testing indicated that mixes containing the gravel coarse aggregate should prevent mixtures with excessive VMA (as seen at $N_{design} = 75$ during Part 2). Five fine aggregates were used including FA-2, FA-3, FA-4, FA-6, and FA-7. These fine aggregates are identical to those used during the Part 2 work at $N_{design} = 75$. As suggested by the project panel, three gradations were included: BRZ, TRZ, and ARZ. These gradations are illustrated in Figure 7 and presented in Table 7. The same asphalt binder was used in Part 3 as in Parts 1 and 2. Mix designs were conducted for the HRZ for FA-2 and FA-3 (which have FAA values less than 45.0).

Within the higher compactive effort experiment (i.e., $N_{design} = 100$), a granite coarse aggregate was used with five fine aggregates: FA-2, FA-4, FA-6, FA-7, and FA-9. Again, the BRZ, TRZ, and ARZ gradations were investigated.

For both compactive effort experiments, mix designs and performance testing using FA-10 and the HRZ gradation were conducted. Similar to Parts 1 and 2, this information should provide a "worst-case" baseline.

Figure 6 shows the flow of work in Part 3. For a given factor-level combination, mix designs were first conducted for the gradation(s) violating the restricted zone. If the mixture(s) met all Superpave volumetric requirements, then mix designs were conducted for the two control gradations (i.e., BRZ and ARZ). However, if none of the mixes violating the restricted zone met all volumetric criteria, testing was stopped for that factor-level combination. Mixtures meeting all volumetric criteria were used for performance testing. For Part 3, only the Asphalt Pavement Analyzer (APA) was used as a performance test.

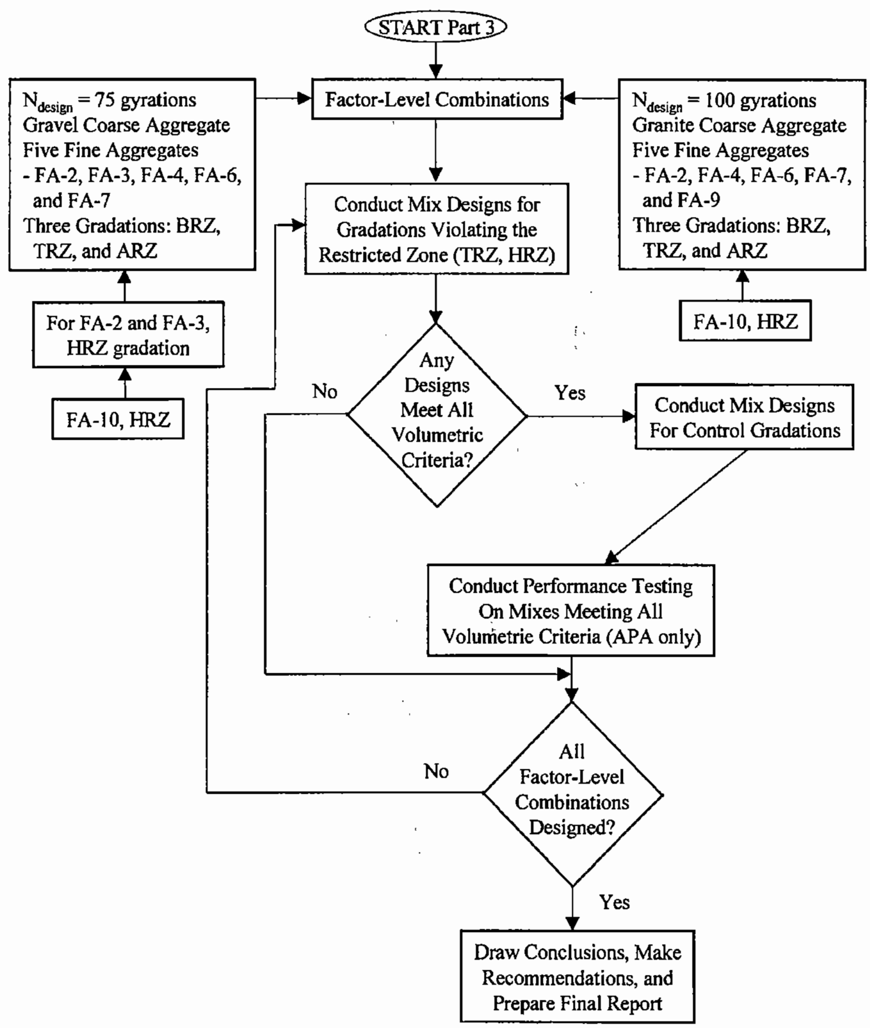

Figure 6. Flow diagram showing work for Part 3.

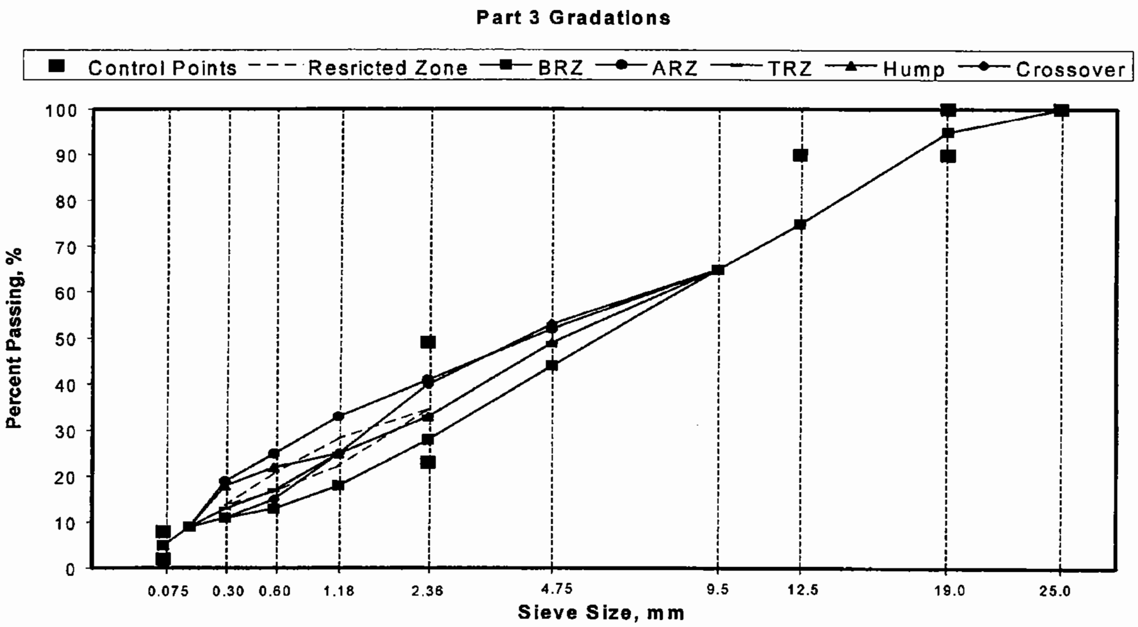

Figure 7. Part 3 gradations.

Response Variables

The performance of mixes with various factor-level combinations meeting Superpave volumetric requirements were evaluated on the basis of performance-related mechanical tests. Because the primary purpose of the restricted zone is to avoid rut-prone mixes, the mixes in this study were evaluated for their rutting potential. This was accomplished by two different types of tests: empirical and fundamental. For the empirical test, the APA was used. The Superpave shear tester and the repeated load confined creep (RLCC) test were used as fundamental tests.

Three tests were included to ensure a satisfactory conclusion of this study. It was not expected that all three permanent deformation tests (i.e., one empirical and two fundamental) will provide exactly similar results. If they did, one mix validation test would be sufficient. However, all three tests might not be equally sensitive to changes in gradation and FAA values. Their relative sensitivity to changes in gradation and FAA values would be evident from the test data. The test that is most sensitive to these two important factors of this research project will be considered the most relevant and significant.

TABLE 7 19.0-mm NMAS gradations used in Part 3

Sieve, mm	BRZ	ARZ	TRZ	HRZ
25.0	100	100	100	100
19.0	95	95	95	95
12.5	75	75	75	75
9.5	65	65	65	65
4.75	44	52	49	49
2.36	28	41	33	33
1.18	18	33	25	25
0.60	13	25	17	22
0.30	11	19	13	18
0.15	9	9	9	9
0.075	5	5	5	5

Asphalt Pavement Analyzer

The APA is an automated, new generation of Georgia Load Wheel Tester (GLWT). The APA (see Figure 8) features controllable wheel load and contact pressure, adjustable temperature inside the test chamber, and the capability to test the samples while they are either dry or submerged in water. This enhanced version of the GLWT gives rutting and moisture susceptibility test environments that are more representative of actual field conditions than were previously provided by the GLWT. The APA test was conducted dry to 8,000 cycles, and rut depths were measured continuously. The APA can test three pairs of gyratory-compacted specimens of 75-mm height. Testing with the APA was conducted at 64°C. The air void content of the different mixtures was 6.0 ± 0.5 percent. The mixture was aged 2 h at the compaction temperature prior to compacting. Hose pressure and wheel load were 690 kPa and 445 N (100 psi and 100 lb), respectively.

Superpave Shear Tester (AASHTO TP7-94)

The Superpave shear tester, shown in Figure 9, is a closed-loop feedback, servohydraulic system that consists of four major components: a testing apparatus, a test control unit, an environmental control chamber, and a hydraulic system. The ability of a pavement structure to resist permanent deformation and fatigue cracking is estimated through the use of the Superpave shear tester. The Superpave shear tester simulates, among other things, the comparatively high shear stresses that exist near the pavement surface at the edge of vehicle tires—stresses that lead to the lateral and vertical deformations associated with permanent deformation in surface layers.

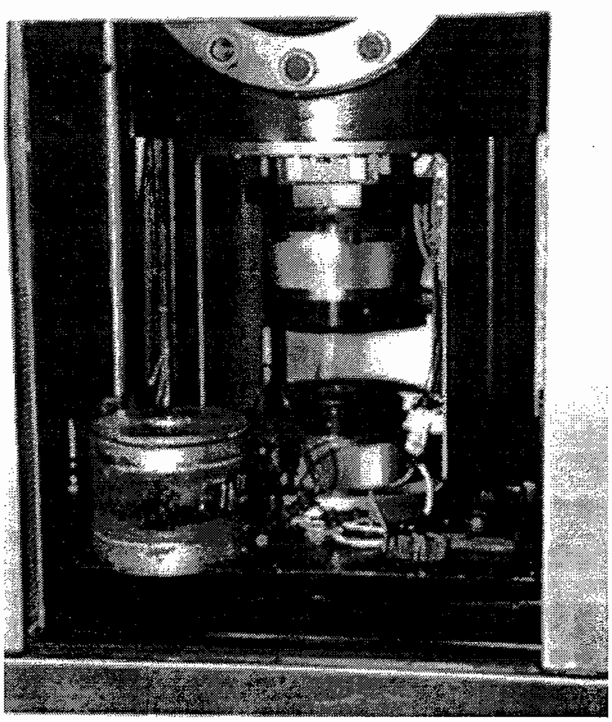

Figure 8. Asphalt Pavement Analyzer.

Figure 9. Superpave shear tester.

The repeated shear at constant height (RSCH) test (AASHTO TP7, Procedure F) was selected to assess the permanent deformation response characteristics of the mixtures. The RSCH test is performed to estimate rut depth. This test operates by applying repeated shear load pulses to an asphalt mixture specimen. As the specimen is being sheared, the constant height prevents specimen dilation, thereby promoting the accumulation of permanent shear strain. The test can be used for comparatively analyzing shear response characteristics of mixtures subjected to similar loading and temperature conditions.

The literature review indicated that this Superpave shear tester has been used successfully by researchers to evaluate the relative rutting potential of HMA mixtures. All specimens for Superpave shear testing were fabricated at 3.0 ± 0.5 percent air voids and tested at 50°C. This test temperature was selected because it is representative of effective temperature for permanent deformation ($T_{eff}[PD]$) as used in Superpave shear test protocol for the southeastern United States and is believed to be critical for inducing rutting in HMA pavements. Prior to compaction, the mixture was aged for 4 h at 135°C.

Repeated Load Confined Creep Test

The RLCC test is considered a fundamental experimental method to characterize the rutting potential of HMA because fundamental creep principles can be applied to deformation of viscoelastic mixes. A material testing system (MTS) was used to conduct this test. A deviator stress, along with a confining stress, is repetitively applied on a HMA sample for 1 h, with a 0.1-s load duration and a 0.9-s rest period. After the 1-h test, the load is removed, and the rebound measured for 15 min. The strain observed at the end of this period is reported as the permanent strain. The permanent strain indicates the rutting potential of the mix. The target air void content for mixtures tested by the RLCC test was 4.0 ± 0.5 percent. Prior to compaction, the mixture was aged for 4 h at 135°C. The test temperature was 60°C. Test loadings consisted of an 138 kPa (20 psi) confining pressure and an 827 kPa (120 psi) normal pressure.

CHAPTER 3

LABORATORY TEST RESULTS AND ANALYSIS

This chapter presents the test results and analysis of the laboratory experiment. The experimental plan was divided into three parts. Experiments in Parts 2 and 3 were guided by the results of Part 1. This chapter is divided into three sections, each providing test results, analysis, and decisions made for subsequent parts.

PART 1 TEST RESULTS AND ANALYSIS

Mix designs for 9.5-mm NMAS mixes were conducted for 80 factor-level combinations during Part 1. As mentioned earlier, the compactive effort used in Part 1 corresponded to a design traffic level of 3 to 30 million ESALs. The initial, design, and maximum number of gyrations were 8, 100, and 160, respectively. The results of these mix designs are presented in Appendix C.

Of the 80 mixes designed, only 9 mixes met all volumetric (i.e., VMA, VFA [voids filled with asphalt], and $\%G_{mm}@N_{initial}$ [the percent of maximum specific gravity at the initial number of gyrations]) and FAA criteria. Of the mixes not meeting criteria, 22 did not meet VMA, 13 did not meet VFA, 6 did not meet $\%G_{mm}@N_{initial}$, 28 did not meet VMA and $\%G_{mm}@N_{initial}$, 1 did not meet $\%G_{mm}@N_{initial}$ and VFA, and 1 did not meet VMA and VFA.

A secondary goal of this research was to evaluate the effect of mix constituent properties on the volumetrics of the 80 designed mixes. Volumetric properties considered included air voids, VMA, VFA, $\%G_{mm}@N_{initial}$, and $\%G_{mm}@N_{maximum}$. Air voids were kept constant at 4 percent as this void level defines optimum asphalt content, so air voids were not analyzed. VFA is a function of VMA and air voids and no mix failed $\%G_{mm}@N_{maximum}$, so neither were included. Therefore, only VMA and $\%G_{mm}@N_{initial}$ were analyzed.

The first step in this analysis was to conduct an analysis of variance (ANOVA) to determine the effect of coarse aggregate, fine aggregate, and gradation on VMA and $\%G_{mm}@N_{initial}$. For these ANOVAs, the calculation of the F-statistics had to be modified. This was because only one response was obtained for each factor-level combination (e.g., there was only one VMA for each mix). To calculate the F-statistic, the degrees of freedom associated with the interactions among the experiment factors were sacrificed. This sacrifice of degrees of freedom for the interactions provided the necessary mean squares of error to calculate the F-statistic without sacrificing the results of the ANOVA.

Results of the ANOVA conducted to evaluate the significance of the experiment's main factors is presented in Table 8. This table shows that all three main factors significantly affect VMA. Based upon the F-statistics, it is seen that the coarse aggregate had the greatest effect on VMA (i.e., it had the largest F-statistic) followed by fine aggregate and gradation, respectively.

Figure 10 illustrates the relative effect of coarse aggregate and gradation on VMA. Each bar on this figure represents the average VMA for mixes having the same coarse aggregate and gradation type—therefore, each bar is the average VMA for all fine aggregates. This figure suggests that mixes containing the more angular coarse aggregate yielded collectively higher VMA values than did mixes containing the crushed gravel fine aggregate. This was true for each gradation. Figure 10 shows that the ARZ and CRZ gradations tended to provide higher VMA values and that the HRZ and TRZ provided the lowest VMA values. Recall that the HRZ gradation was only combined with fine aggregates having an FAA of 45 or lower. Evaluation of the FA-1, FA-2, and FA-3 mix design data indicated that the HRZ gradation provided higher VMA values (an average of 14.4 percent for granite and 13.3 percent for gravel coarse aggregates, respectively) than did the TRZ gradation (an average of 13.8 percent for granite and 12.9 percent for gravel coarse aggregate, respectively). Because the TRZ gradation generally provided the lowest VMA values, it appears that the MDL defined within the Superpave mix design system for 9.5-mm NMAS gradations relatively is in the correct location.

The effect of fine aggregate on the VMA values was evaluated by correlating VMA to FAA. Figures 11 and 12 illustrate the relationship between FAA and VMA for mixes containing granite and gravel coarse aggregates, respectively. Within these figures, the relationship between FAA and VMA is shown for each gradation. Coefficients of determination (R^2) are also shown for each relationship. Table 9 presents the F-statistic and p-value for each regression. Figures 11 and 12 indicate that the relationship between VMA and FAA is poor as R^2 values are typically below 0.25. In fact, the F-statistic and probability values indicate that the relationships are not significant. Although there is no significance to the relationships, there does appear to be a trend that is common to

TABLE 8 Results of ANOVA to determine significance of main factors on VMA

Source	Degrees of Freedom	*F*-Statistic	*P*-Value
Coarse Aggregate	1	156.40	0.000
Fine Aggregate	8	110.85	0.000
Gradation	4	13.99	0.000

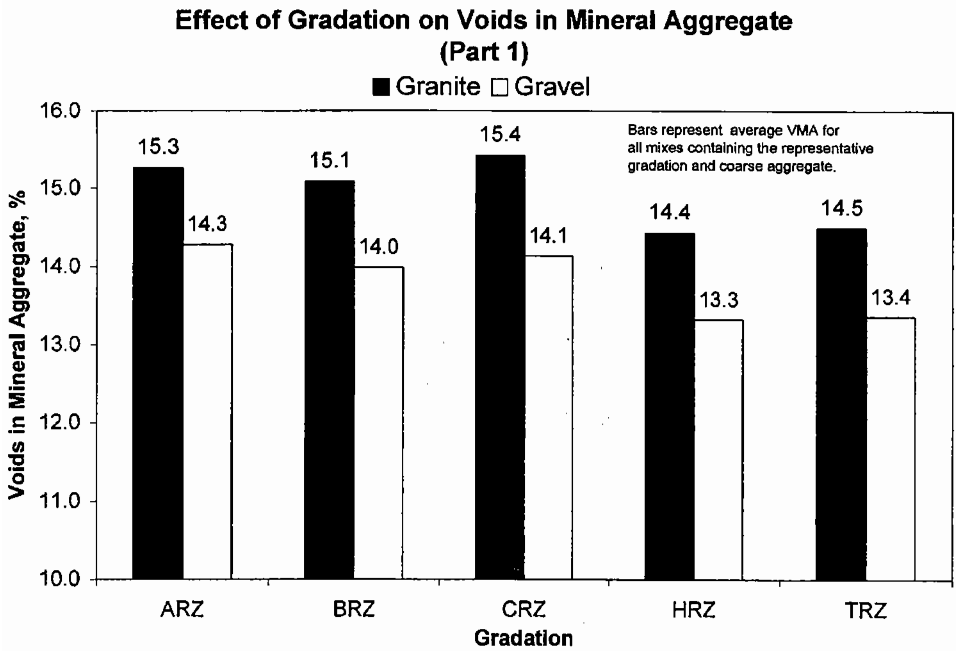

Figure 10. Effect of gradation on VMA.

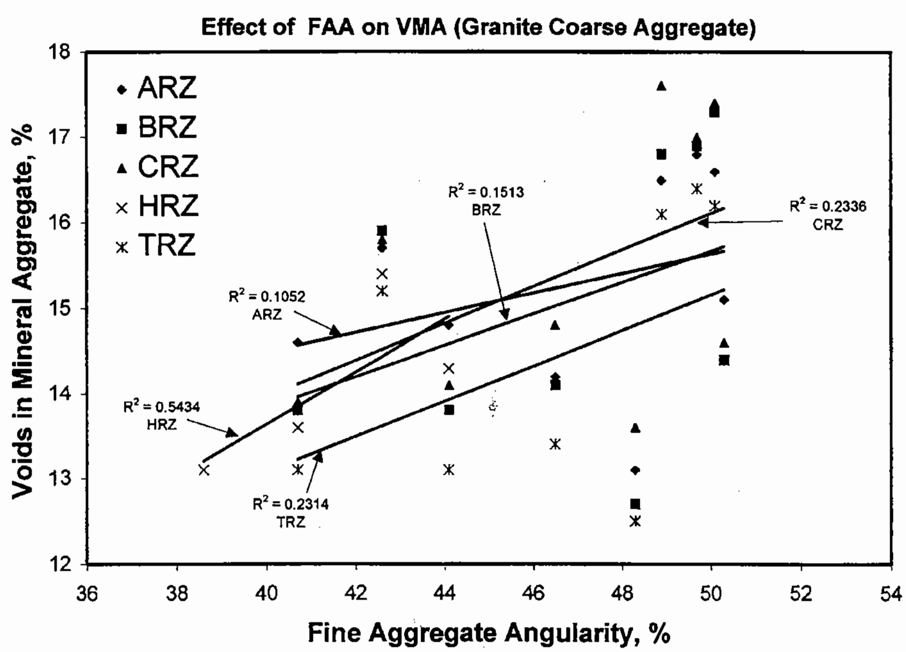

Figure 11. Effect of FAA on VMA (granite coarse aggregate).

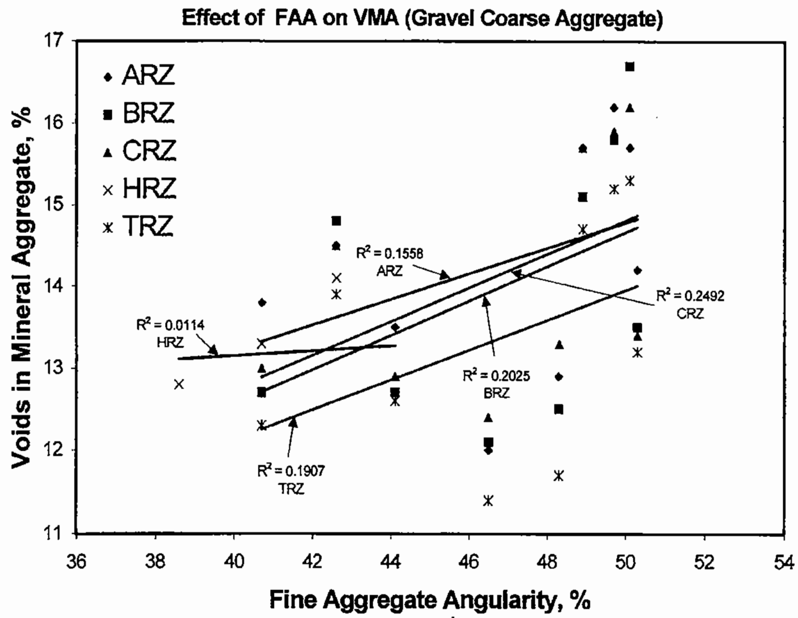

Figure 12. Effect of FAA on VMA (crushed gravel coarse aggregate).

all relationships: increasing VMA values with increasing FAA values. The relative locations of the regression lines are similar for both the granite and gravel coarse aggregate data sets.

Results of the ANOVA conducted to evaluate the significance of coarse aggregate, fine aggregate, and gradation on $\%G_{mm}@N_{initial}$ is presented in Table 10. This table shows that all three main factors significantly affect $\%G_{mm}@N_{initial}$, similar to the VMA analysis. Based upon the F-statistics, the fine aggregate had the greatest effect, followed by gradation and coarse aggregate, respectively.

Figure 13 illustrates the effect of coarse aggregate and gradation on $\%G_{mm}@N_{initial}$. As show by the ANOVA, the effect of coarse-aggregate type seems to be minimal (although significant). This figure suggests that the BRZ gradation provided the lowest $\%G_{mm}@N_{initial}$ values. The CRZ gradation had similar but slightly higher $\%G_{mm}@N_{initial}$ values. Figure 13 suggests that the HRZ gradation provided the highest $\%G_{mm}@N_{initial}$ values. However, similar to the VMA analysis, this conclusion would be misleading. For the three fine aggregates in which both gradations were used, the $\%G_{mm}@N_{initial}$ averaged 91.0 percent for the HRZ gradation and 90.7 percent for the TRZ gradation;

TABLE 9 Regression statistics for FAA versus VMA regressions

Gradation	Granite		Gravel	
	F-statistic	p-value	F-statistic	p-value
ARZ	0.82	0.394	1.29	0.293
BRZ	1.25	0.301	1.78	0.224
CRZ	2.13	0.187	2.32	0.171
HRZ	2.38	0.263	0.02	0.893
TRZ	2.11	0.190	1.65	0.240

TABLE 10 Results of ANOVA to determine significance of main factors on $\%G_{mm}@N_{initial}$

Source	Degrees of Freedom	F-Stastistic	P-Value
Coarse Aggregate	1	7.89	0.007
Fine Aggregate	8	101.85	0.000
Gradation	4	38.31	0.000

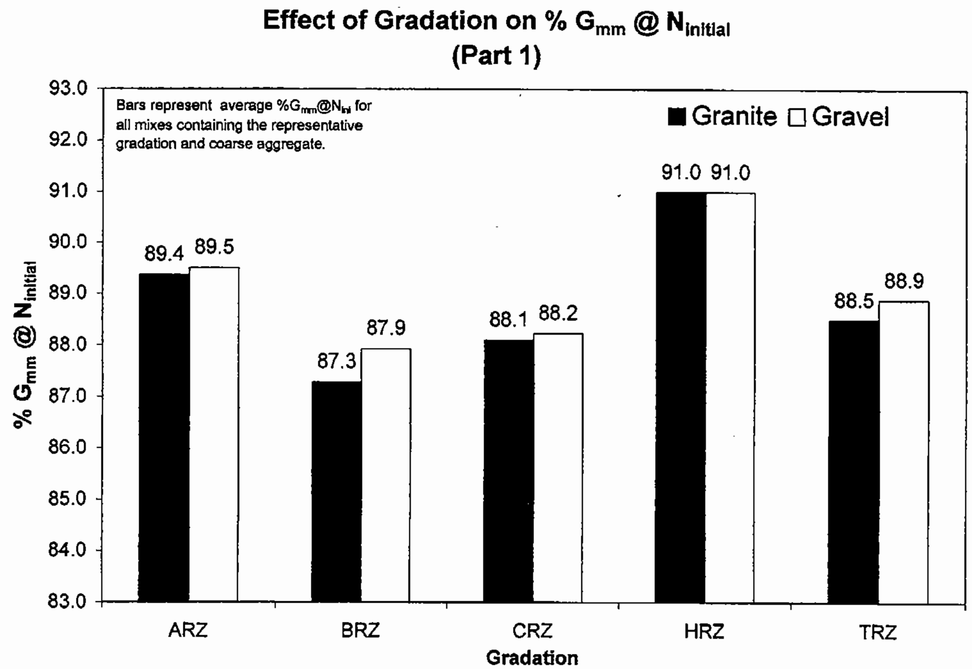

Figure 13. Effect of gradation on %G_{mm}@$N_{initial}$ (Part 1).

therefore, both appear similar and suggest that the ARZ gradation actually provided the highest %G_{mm}@$N_{initial}$ values.

The effect of fine aggregate on %G_{mm}@$N_{initial}$ is illustrated in Figures 14 and 15 for mixes containing granite and gravel coarse aggregate, respectively. These figures illustrate the relationship between FAA and %G_{mm}@$N_{initial}$. R^2 values are also shown for each relationship. The R^2 values indicate a stronger relationship between FAA and %G_{mm}@$N_{initial}$ than for FAA and VMA (see Figures 11 and 12). Table 11 presents the F-statistics and probabilities for each regression shown in Figures 14 and 15.

The regression statistics in Table 11 suggest a significant relationship between FAA and %G_{mm}@$N_{initial}$. The relationships show increasing values of FAA led to decreasing values of %G_{mm}@$N_{initial}$. Furthermore, none of the mixes having an FAA value of 45 or lower met the %G_{mm}@$N_{initial}$ requirement of 89 percent maximum. This was true for both coarse aggregates. Overall, it appears that higher FAA values contribute to a stronger aggregate skeleton (in terms of more resistance to compaction) at initial compaction levels.

Another interesting observation from the Part 1 mix design data was that none of the mixes failed the %G_{mm}@$N_{maximum}$ requirement of 98 percent maximum. This was true even for the worst-case FA-10 mixes with a humped gradation. This observation raises the question of whether the $N_{maximum}$ requirement is necessary or whether the limit of 98 percent needs to be changed.

After completion of all mix designs, performance testing was conducted. Performance testing included the APA, RSCH test with the Superpave shear tester, and the RLCC test as described in Chapter 2. The project statement for this study called for performance testing on mixes that met all volumetric criteria. However, with the concurrence of the project panel, some mixes not meeting VFA requirements were performance tested. This VFA exception was made because of current Superpave VMA requirements for 9.5-mm NMAS mixtures. Optimum asphalt content is defined as the asphalt content that provides 4.0 percent air voids. For 9.5-mm NMAS mixes, the minimum VMA allowed is 15.0 percent. At a VMA of 15.0 percent and an air void content of 4.0 percent, VFA is equal to 73.3 percent. The Superpave requirements for VFA range from 65.0 to 75.0 percent. This VFA range effectively limits VMA to a maximum of 16.0 percent as air voids are set at 4.0 percent at mix design. Only a 1.0-percent range of VMA, therefore, is allowed by the Superpave mix design requirements.

The exception used in this study was based on the findings of the WesTrack Forensic Team (*1*). This report recommended that VMA be restricted to no more than 2.0 percent above the minimum value; therefore, besides mixes meeting all volumetric requirements, performance testing was also conducted on mixtures that failed VFA but that had VMA values below or equal to 17.0 percent. This provided an allowable VFA range in this study of 73.3 to 76.5 percent.

Another exception approved by the project panel was to conduct performance testing on mixtures containing FA-6 (a limestone fine aggregate) and granite coarse aggregate (all gradations) even though these combinations did not meet VMA. The project panel recommended the inclusion

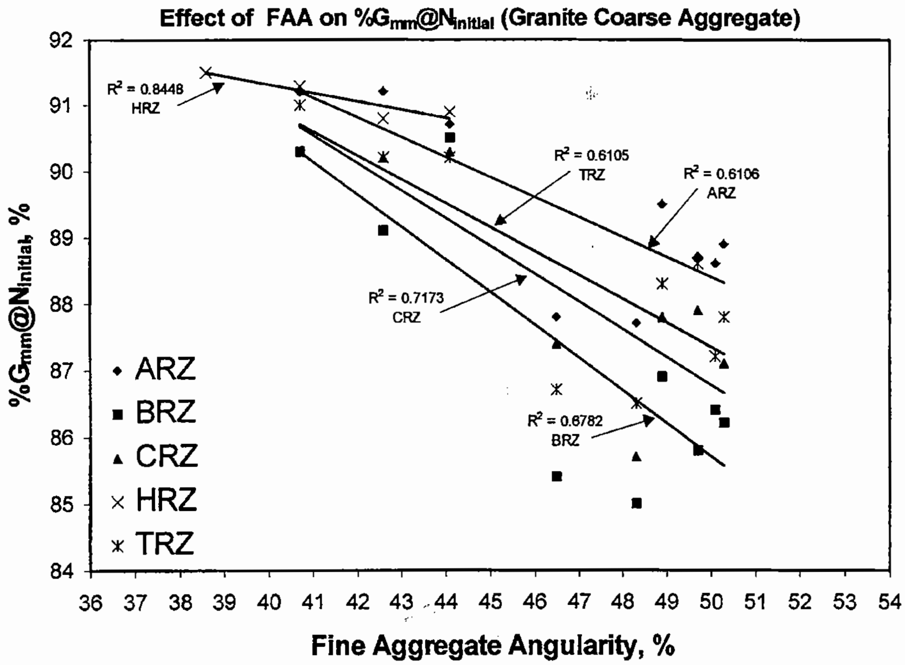

Figure 14. Effect of FAA on $\%G_{mm}@N_{initial}$ *(granite coarse aggregate).*

of these mixes because none of the mixtures meeting all volumetric criteria (and those included with the VFA exception) contained a limestone fine aggregate, which is one of the most common aggregates in the United States. The FA-6/granite mixes were included for informational purposes only.

The fine aggregate FA-10, which had a very low FAA value of 38.6, was used with both granite and gravel coarse aggregates to provide a humped gradation violating the restricted zone (i.e., HRZ). These two mixes did not meet the Superpave requirements for FAA, VMA, or $N_{initial}$. However, these mixes were performance tested to obtain a baseline, worst-case scenario.

Results of Part 1 performance testing for mixes containing FA-10, FA-6, FA-7, FA-4, and FA-9 are presented in

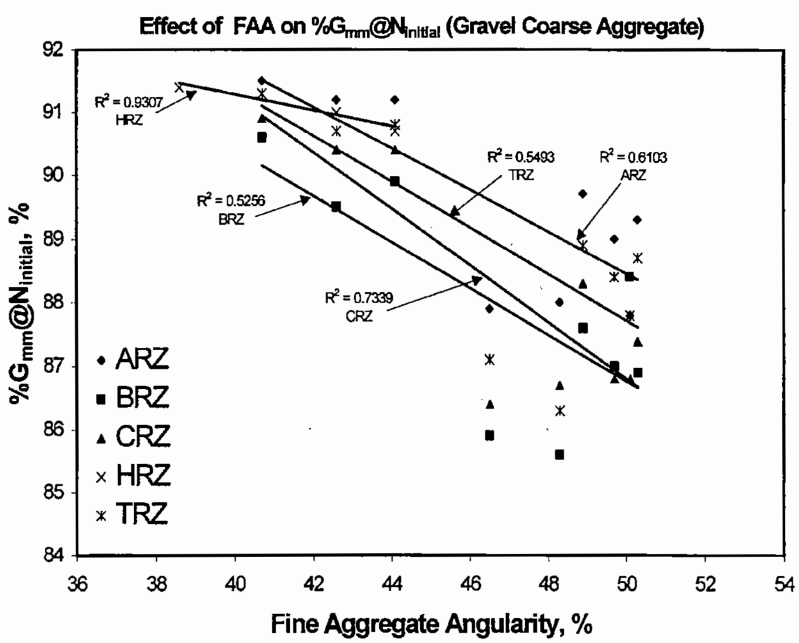

Figure 15. Effect of FAA on $\%G_{mm}@N_{initial}$ *(crushed gravel coarse aggregate).*

TABLE 11 Regression statistics for FAA versus $\%G_{mm}@N_{initial}$ relationships

Gradation	Granite		Gravel	
	F-statistic	*p*-value	*F*-statistic	*p*-value
ARZ	10.98	0.013	10.96	0.013
BRZ	14.76	0.006	7.75	0.027
CRZ	17.76	0.004	19.31	0.003
HRZ	10.89	0.081	26.88	0.035
TRZ	10.97	0.013	8.53	0.022

Appendix C. Results for the APA are presented as the manually measured rut depth after 8,000 cycles. For the RSCH test, results are presented as the plastic strain after 5,000 cycles, expressed as a percentage. Results for the RLCC test are presented as the permanent strain measured after 3,600 load repetitions (applied in 1 h) and a 15-min rebound time, again expressed as a percentage.

Figure 16 illustrates the results of APA testing in the form of a bar chart. Results are shown for the 24 mixes that (1) met all volumetric criteria, (2) met the VFA exception, (3) were recommended by the project panel (e.g., containing FA-6), or (4) was a worst-case scenario (e.g., containing FA-10).

Data within Figure 16 are classified by whether the mixture has a gradation that violates the restricted zone. Solid black bars depict mixes having gradations violating the restricted zone; unshaded bars represent mixes having gradations that do not violate the restricted zone. As can be seen from the figure, the same combination of coarse aggregate and gradation was not tested for all fine aggregates—therefore, performing an analysis of variance was not possible. Duncan's multiple range tests (DMRT) were used to rank the performance of mixes having identical coarse aggregate and fine aggregate (e.g., granite/FA-4). This analysis provided a comparison among gradations for a given coarse aggregate/fine aggregate combination to determine whether gradations violating the restricted zone performed differently than gradations residing outside the restricted zone. Figure 16 shows the results of the DMRT rankings as A, AB, and B. There is no statistically significant difference (at a significance level $\alpha = 0.05$) in performance if two gradations within a coarse aggregate/ fine aggregate combination have the same letter ranking.

Figure 16 shows that all three main factors (i.e., coarse aggregate, fine aggregate, and gradation shape) appear to affect the measured APA rut depths. Collectively, where comparisons are possible, mixes containing the more angular granite coarse aggregate tended to have lower rut depths.

The fine aggregate type also affected the measured rut depths. The FA-10 mixes containing gravel coarse aggregate were the least rut resistant. Also as expected, mixes containing FA-6 were rut resistant. Recall that these four FA-6 mixes were included for informational purposes only because all failed VMA requirements. Because each mix had low VMA,

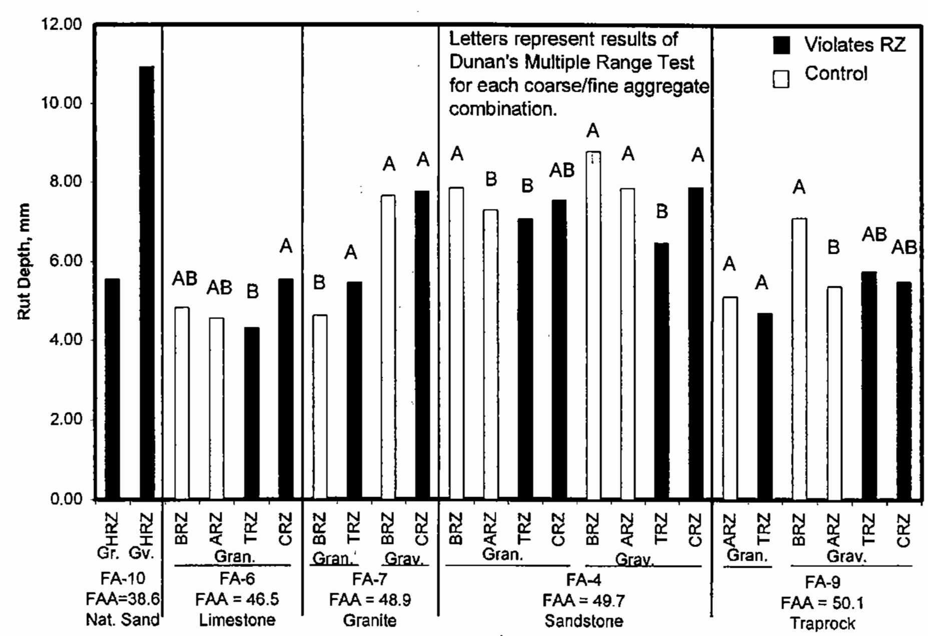

Figure 16. APA rut test data (Part 1).

all four mixes were under-asphalted and, as a result, were rut resistant. However, the FA-6 mixes that violated the restricted zone criteria (i.e., TRZ and CRZ) did perform similarly to the mixes not violating the restricted zone (i.e., BRZ and ARZ).

In all but one case (FA-7/granite mixes) of the seven coarse aggregate/fine aggregate combinations tested, the mixes having gradations that violate the restricted zone performed similarly or better than did the mixes having gradations that did not violate the restricted zone. In this one case, the rut depths for both FA-7/granite/BRZ and FA-7/granite/TRZ were both less than 6 mm. Based upon these Part 1 APA data, it appears that the restricted zone is practically redundant as a requirement to ensure adequate rut resistance if the mix meets all Superpave volumetric and FAA criteria.

No meaningful relationship between FAA values and APA rut depth was obtained, probably because the FAA values of the mixes (which met volumetric requirements) only ranged from 48.9 to 50.1.

Figure 17 illustrates the results of the RLCC test. Results are presented as permanent strain as a percentage. Similar to the APA results, the results show the mixes containing FA-10 had the least resistance to permanent deformation. These FA-10 mixes had considerably higher permanent strain values when compared with the other mixes. The FA-6 limestone mixes collectively had the lowest permanent strain values, similar to the APA rut depths. Again, this was likely due to the low asphalt contents in these mixes (i.e., low VMA).

Similar to the APA analysis, DMRT rankings were conducted on each combination of coarse aggregate/fine aggregate to isolate the effect of gradation. In all but one case (i.e., FA-9/granite) of the seven coarse aggregate/fine aggregate combinations tested, the mixes having gradations violating the restricted zone performed as well or better than did the mixes having gradations complying with the restricted zone requirement. Close inspection of the one exception (i.e., FA-9/granite) shows that both mixes ARZ and TRZ have very low permanent strain values and, therefore, can be considered rut resistant. The RLCC data appears to confirm the APA conclusion that the restricted zone requirement is not needed when the Superpave volumetric and FAA criteria are met.

Figure 18 presents the RSCH test data. Results in this figure are shown as plastic strain expressed as a percentage. Initial observation of Figure 18 indicates little variation in the test results: even the worst-case FA-10 mixes did not have high plastic strain values. All test results were below 2.5 percent plastic strain, which historically suggests adequate rut resistance. Similar to the APA and RLCC test data, DMRT rankings were determined for each fine aggregate/coarse aggregate combination. These rankings also show that not much variation in test results was exhibited. Except for the FA-9/gravel combination, all combinations had similar DMRT rankings. This suggests that the RSCH test was not sensitive enough to identify small changes in gradation or asphalt content, possibly because of test variability. Three replicates were used in this study. Recent research (*2*) has suggested the use of five replicates, discarding the minimum and maximum values and averaging the middle three values to improve the reliability of the RSCH test.

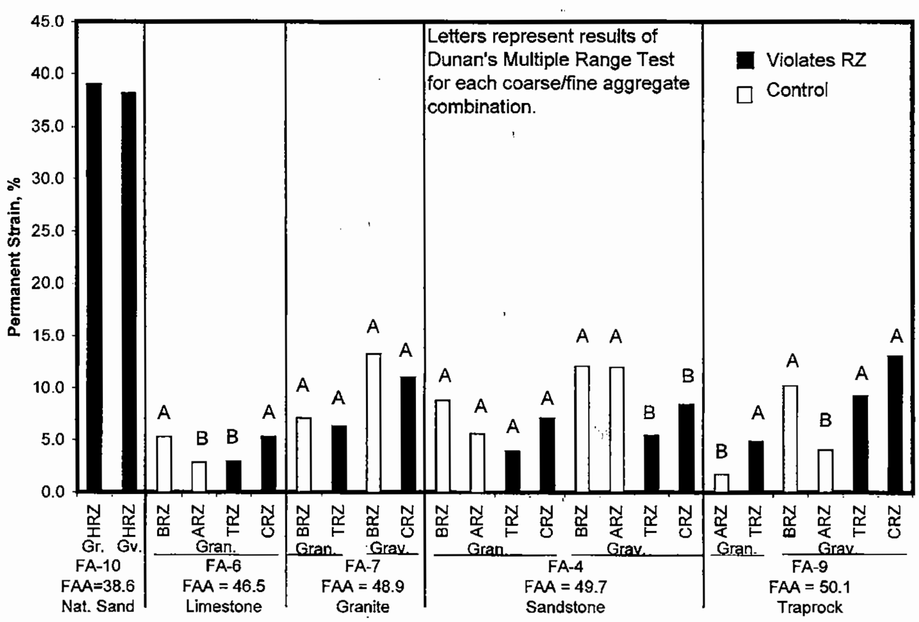

Figure 17. RLCC test data (Part 1).

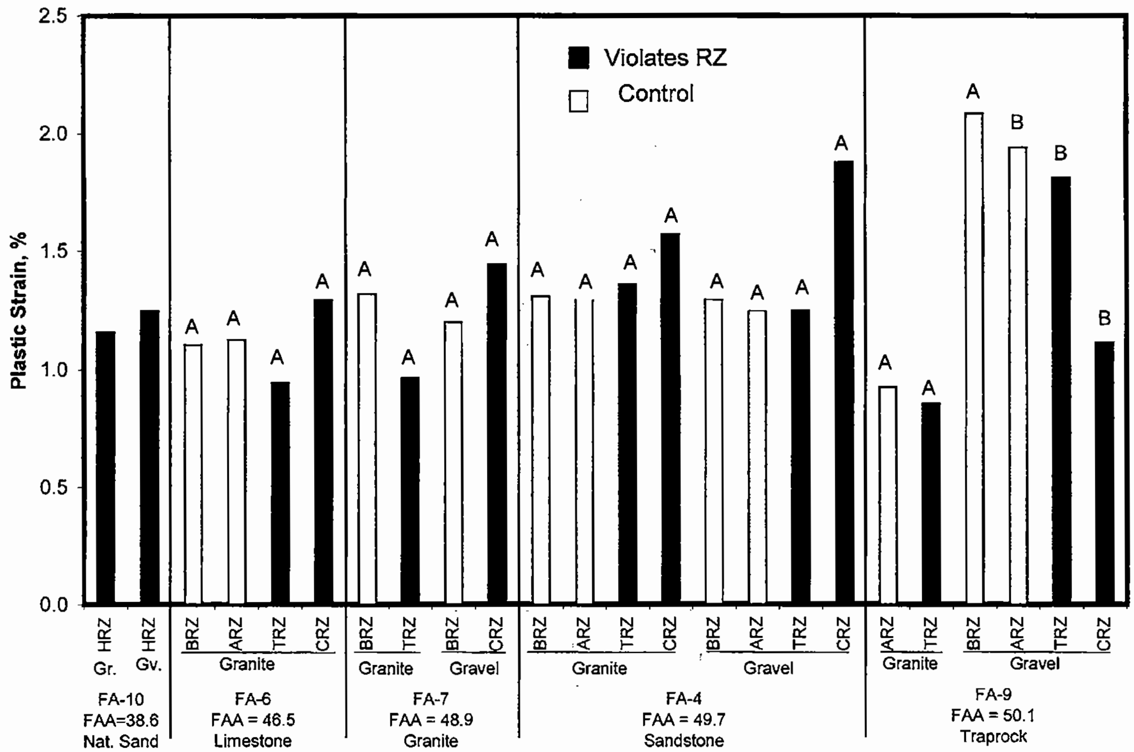

Figure 18. RSCH test data (Part 1).

PART 2 TEST RESULTS AND ANALYSIS

Similar to Part 1, Part 2 involved 9.5-mm NMAS gradations, but included two compactive efforts different than those used in Part 1. The two compactive efforts corresponded to 0.3 to 3 million ESALs (i.e., N_{design} = 75 gyrations) and more than 30.0 million ESALs (i.e., N_{design} = 125 gyrations). Only three gradations were used in all mixes: BRZ, TRZ, and CRZ. Only the granite coarse aggregate was used in Part 2. During Part 1, gravel coarse aggregate produced mixes with low VMA values.

Six fine aggregates—FA-10, FA-2, FA-3, FA-6, FA-7, and FA-4 (in increasing order of FAA values)—were used in mixes designed with an N_{design} of 75 gyrations. Appendix D gives optimum mix design data for mixes with these fine aggregates. Four fine aggregates—FA-10, FA-7, FA-4 and FA-9—were used in mixes compacted with an N_{design} of 125 gyrations. Appendix D also gives optimum mix design data for these fine aggregates. Fine aggregates that had high potential of meeting the minimum VMA requirements (based on mix design data obtained in Part 1) were selected for Part 2. A limestone fine aggregate (i.e., FA-6) was included because limestone is widely used in the United States.

Because each of the mixes studied in Part 2 contained the same coarse aggregate, the factors evaluated were design compactive effort, fine aggregate type (i.e., FAA), and gradation shape. Similar to the analyses conducted in Part 1, the mix design data were analyzed to determine the effect of each factor on volumetric properties. Figures 19 and 20 present the effect of gradation on VMA and $\%G_{mm}@N_{initial}$ for both compactive efforts, respectively. Similar to Part 1, Figure 19 shows that the CRZ gradation produced the highest VMA values for both compactive efforts. This effect is probably caused by the CRZ gradation being somewhat gap-graded. The TRZ and HRZ provided low VMA values. Similar to the Part 1 analyses, in which the TRZ and HRZ gradations were designed for the same fine aggregate (i.e., FA-2 and FA-3 for Part 2), the HRZ gradation provided a slightly higher VMA than did the TRZ gradation. Because the TRZ generally provided the lowest VMA values, these Part 2 data support the finding that for 9.5-mm NMAS gradations, the MDL can be used as a guideline for increasing or decreasing VMA in continuously graded HMA mixes. As expected, the mixes using the CRZ and BRZ gradations had lower VMA values for the higher compactive effort (i.e., N_{design} of 125) although the difference was not as large as would be expected.

Figure 20 illustrates the effect of mix gradation on $\%G_{mm}@N_{initial}$. The effect of design compactive effort is also evident in this figure. Mixes compacted at 125 gyrations had

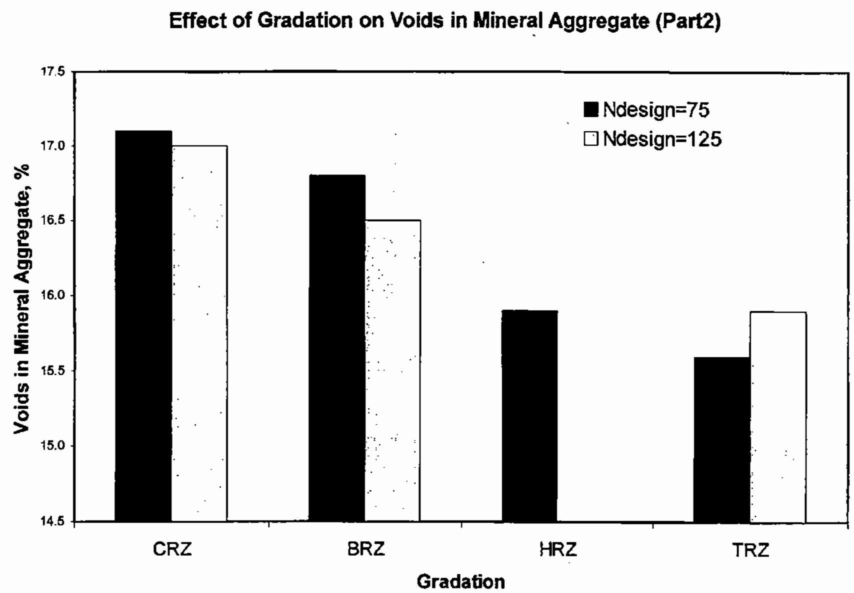

Figure 19. Effect of gradation on VMA for Part 2.

lower $\%G_{mm}@N_{initial}$ values although the initial number of gyrations for the 125 gyration compactive effort was 8 gyrations and the $N_{initial}$ for the 75 gyration compactive effort was 7 gyrations. This is probably due to relatively higher FAA values and lower asphalt contents in high compactive effort mixes compared with low compactive effort mixes, which provided increased initial resistance to compaction. The data also shows a similar effect of gradation on $\%G_{mm}@N_{initial}$ as in Part 1; the mixes using the BRZ and CRZ gradations had similar $\%G_{mm}@N_{initial}$ values and were slightly lower than the values for the TRZ gradation.

As stated previously, during Part 2 the design compactive effort was a factor in the experiment. Figures 21 and 22 present the effect of FAA values on VMA for the $N_{design} = 75$ and $N_{design} = 125$ compactive efforts, respectively. Based upon the regression lines presented in Figure 21, the relationship between FAA and VMA is not significant (i.e., p-values are greater than 0.5). Coefficients of determination ranged from

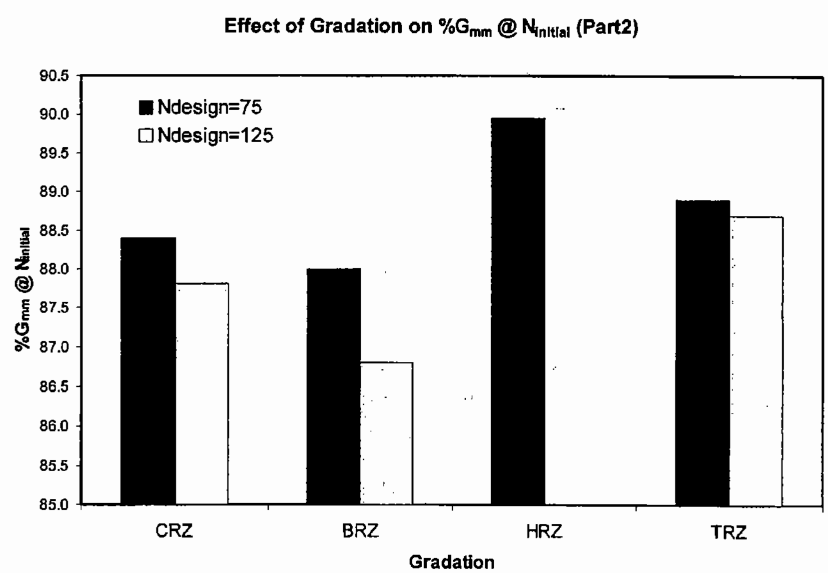

Figure 20. Effect of gradation on $\%G_{mm}@N_{initial}$ for Part 2.

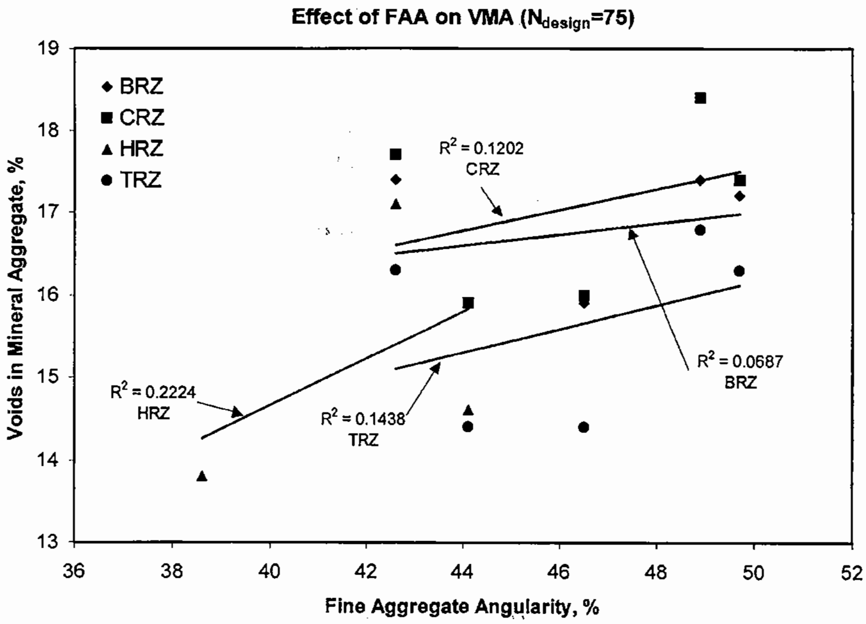

Figure 21. Effect of FAA on VMA for N_{design} = 75, Part 2.

0.06 to 0.22. However, the trend lines do show increasing VMA values with increasing FAA values. This is similar to results in Part 1. Although the results for the N_{design} = 125 mixes did show some higher R^2 values (see Figure 22), the range in FAA values for the N_{design} = 125 mixes was very small (i.e., 48.7 to 50.1). The small range in both FAA and VMA likely resulted in the higher R^2 values for the CRZ and BRZ gradations. The TRZ gradation still had a low R^2 value of 0.01.

Figures 23 and 24 present the relationships between the FAA and $\%G_{mm}@N_{initial}$ for the N_{design} = 75 and 125 mixes, respectively. As shown in the similar Part 1 analyses, the FAA values increase as the $\%G_{mm}@N_{initial}$ values decrease. This relationship suggests that the more angular fine aggregates (i.e., those having higher FAAs) tend to resist early compaction more so than the lower FAA aggregates. For both compactive efforts, the R^2 values were higher than those observed for

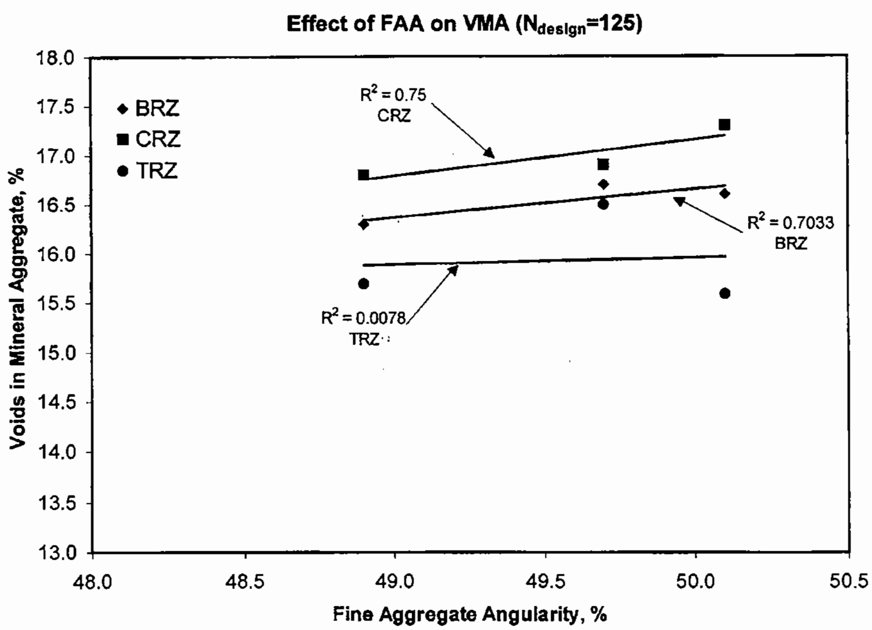

Figure 22. Effect of FAA on VMA for N_{design} = 125 mixes, Part 2.

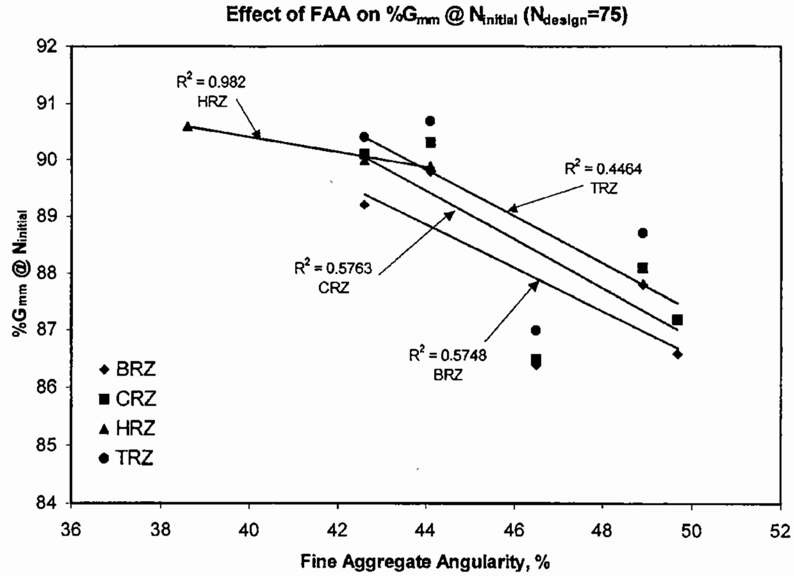

Figure 23. Effect of FAA on $\%G_{mm}@N_{initial}$ for N_{design} = 75 mixes, Part 2.

the FAA–VMA relationships, but the relationships were not significant. However, there was one exception: the TRZ gradation for N_{design} = 125 mixes (see Figure 24). This relationship had an R^2 value of almost zero. The likely reason for this low R^2 value is that the slope of the trend line was basically zero.

Another definite trend can be observed about the relationship between FAA and $\%G_{mm}@N_{initial}$ for the five gradations used in Parts 1, 2, and 3 (see Figures 14, 15, 23, and 24). HRZ and CRZ have the highest correlation in all cases. Also, the order of lines remains the same. That is, the short line of HRZ is followed by ARZ, TRZ, CRZ, and, finally, BRZ. These trends should be helpful to the mix designer to ensure the mix meets the maximum requirement for $\%G_{mm}@N_{initial}$. Thus, it appears that $\%G_{mm}@N_{initial}$ is predominantly controlled by FAA and the fine aggregate content.

After completion of all mix designs, performance testing was conducted. Similar to Part 1, performance testing included testing with the APA, RSCH test with the Superpave shear tester, and RLCC tests. Results of performance testing for both compactive efforts are provided in Appendix D.

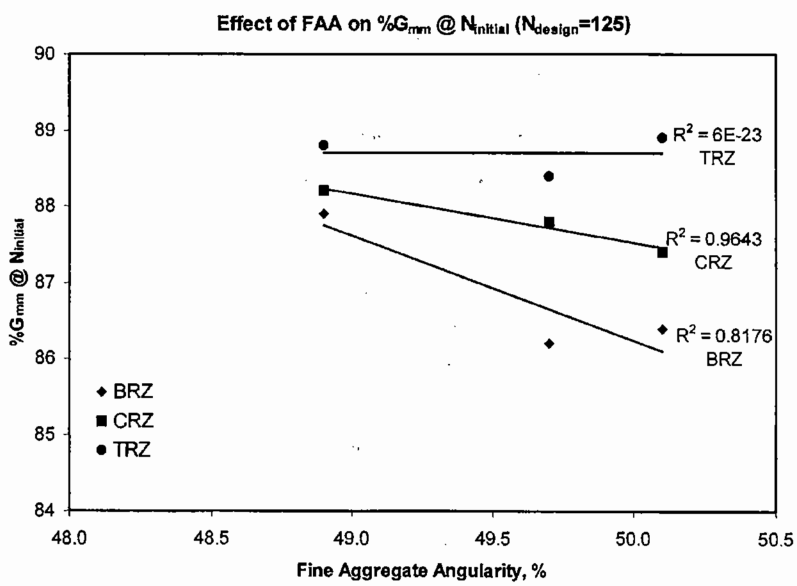

Figure 24. Effect of FAA on $\%G_{mm}@N_{initial}$ for N_{design} = 125 Mixes, Part 2.

A number of mixes in Part 2 failed the VFA requirement with values in excess of the upper limit of 75.0 percent. The VFA exception used in Part 1 was also used in Part 2. This exception called for the performance testing of mixes that failed the upper limit of VFA, but had a VMA value that was no more than 2.0 percent higher than the minimum value (i.e., 17.0 percent or less).

Again, FA-10 was performance tested even though the mixes did not meet volumetric criteria. This was done to provide a baseline of poor performance in the laboratory.

Figure 25 illustrates the results of the APA testing conducted on Part 2 mixes designed at 75 gyrations. Initial observation of this figure suggests that angularity and surface texture of the fine aggregate (i.e., FAA) has a significant effect on measured rut depths. Those mixes containing fine aggregates with FAA values above 46 (i.e., FA-4, FA-6, and FA-7) all had significantly lower rut depths than did the mixes with fine aggregates having FAA values below 46 (i.e., FA-10, FA-2, and FA-3). Also upon initial observation, it is seen that the two FA-3 gradations (i.e., BRZ and CRZ) that met volumetric requirements had rut depths that were slightly higher than did the worst-case baseline FA-10 mix. From a restricted zone standpoint, there was no statistical difference based on DMRT rankings in rut depths between the FA-3 mix that violated the restricted zone (i.e., CRZ) and the control gradation (i.e., BRZ). The only other combination in which a comparison could be made between a gradation violating the restricted zone and a control gradation was FA-6. Again, there was no statistical difference, based on DMRT rankings, in rut depths between the two mixes (i.e., BRZ and CRZ). FA-2, FA-4, and FA-7 had only one gradation that met volumetric requirements (including the VFA exception). Other gradations for these fine aggregates had VMA values in excess of 17.0 percent.

Within the Superpave mix design system, fine aggregates used in mixes designed at 75 gyrations have a requirement for FAA of 40 percent minimum. The data illustrated in Figure 25 suggests that mixes having fine aggregates with FAA values below 46 tend to have more potential for rutting. However, from the standpoint of the restricted zone, there does not seem to be an interaction between the effect of FAA and gradations passing through the restricted zone. This is shown by the data for FA-3 in which the BRZ and CRZ gradations both have similar rut depths. It can be surmised, therefore, that even for this lower compactive effort, the restricted zone is not needed to ensure a rut-resistant mixture. In fact, the data appears to indicate the need for a laboratory "proof" test to be used on designed mixes.

Figure 26 illustrates the APA results of Part 2 mixes designed with 125 gyrations. This figure shows little difference in rut depths among any of the experimental mixes (i.e., FA-4, FA-7, and FA-9 mixes). FA-10 had the highest rut depth, as expected, at approximately 11 mm. The remaining mixes all had rut depths of approximately 8 mm. For each of the fine aggregates (except FA-10), sufficient gradations were available to conduct DMRT rankings to compare the gradations violating the restricted zone (i.e., TRZ and CRZ) and the control gradation (i.e., BRZ). For all three fine aggregates (i.e., FA-4, FA-7, and FA-9), there was no statistical difference among the different gradations. Similar to the Part 1

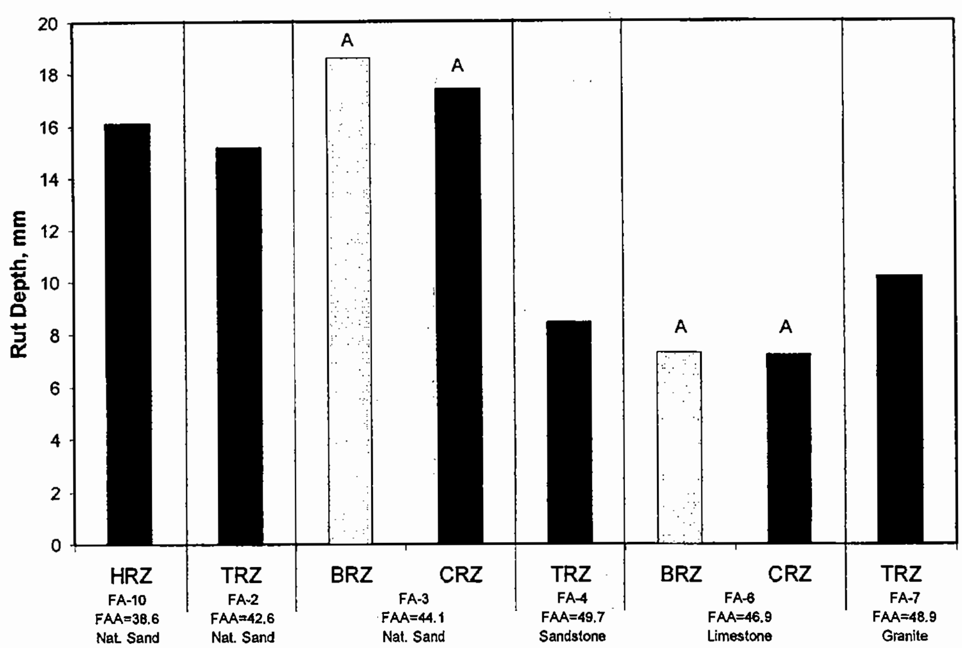

Figure 25. Results of APA testing on mixes designed with 75 gyrations for Part 2.

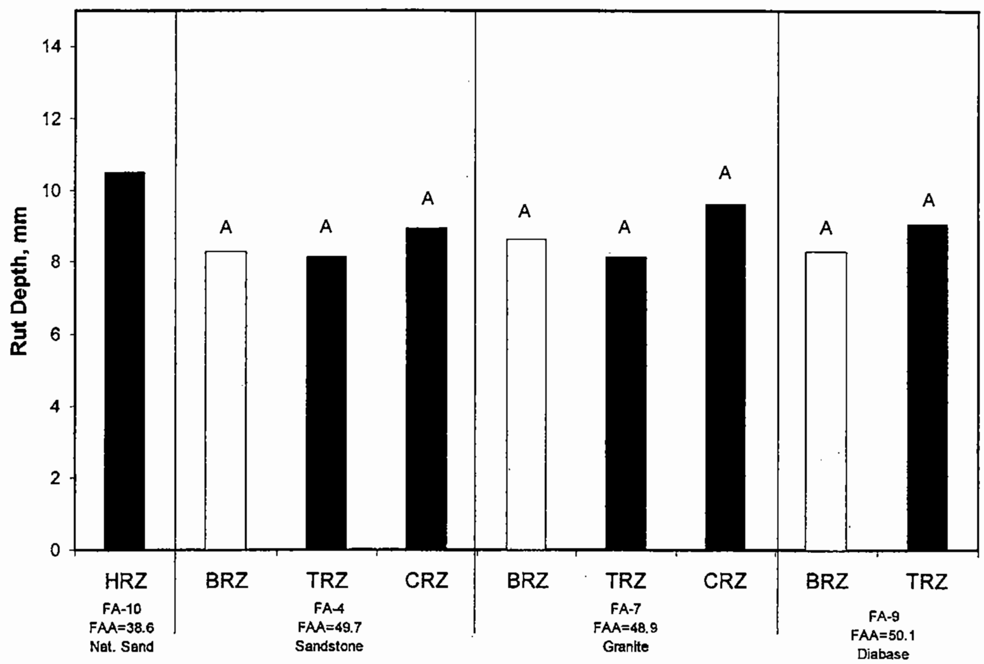

Figure 26. Results of APA testing on mixes designed with 125 gyrations for Part 2.

APA data, Figure 26 suggests that the restricted zone is practically redundant as a requirement to ensure adequate rut resistance if the mix meets all Superpave volumetric and FAA criteria.

Figure 27 illustrates the results of RLCC testing conducted on Part 2 mixes designed with 75 gyrations. This figure does not show the two FA-3 mixes that failed prior to 3,600 load repetitions (i.e., the BRZ and CRZ gradations). As stated previously, the RLCC test uses a confinement pressure on samples. This necessitates the use of a triaxial cell during testing. The premature failure was defined as the point at which the sample within the triaxial cell

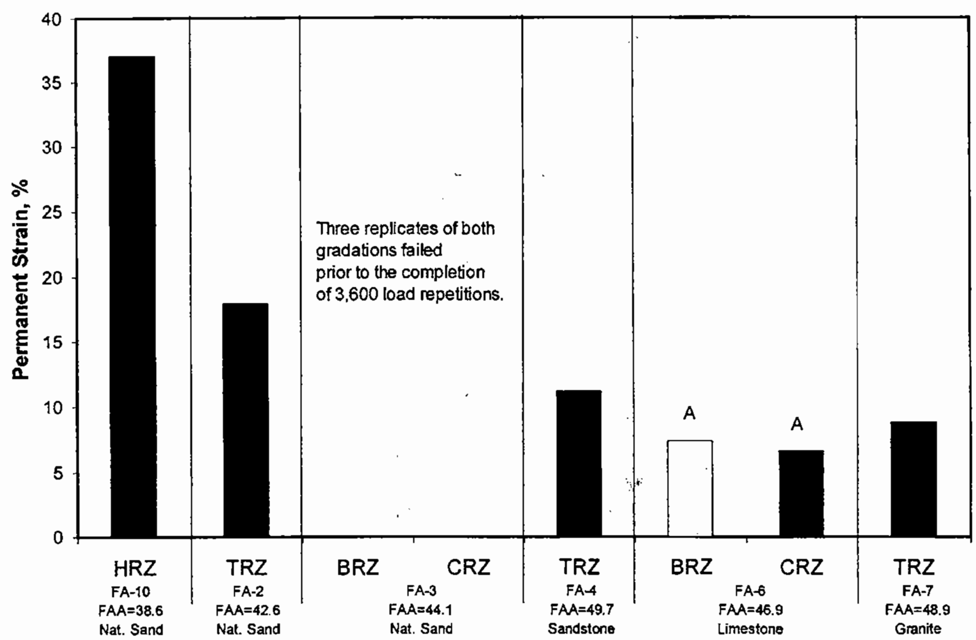

Figure 27. Results of RLCC testing on mixes designed with 75 gyrations for Part 2.

deformed laterally sufficiently to become in contact with the triaxial cell.

The results illustrated in Figure 27 are similar to the APA results shown in Figure 25 in that the mixes containing fine aggregates with FAA values less than 46 (i.e., FA-10, FA-2, and FA-3) all showed significantly less permanent deformation resistance than did the mixes containing fine aggregates with FAA values above 46 (i.e., FA-4, FA-6, and FA-7). Only one fine aggregate had mixes in which gradations violating the restricted zone and a control gradation could be compared (i.e., FA-6). For this fine aggregate, the DMRT rankings indicated that both gradations have similar rut depths.

Based upon both the APA and RLCC performance data for mixes designed with 75 gyrations, it appears that the volumetric and FAA criteria alone do not ensure a rut-resistant mixture. However, gradations passing through the restricted zone do not show more propensity to rut than do gradations residing outside the restricted zone.

Results of RLCC performance testing on Part 2 mixes designed with 125 gyrations are illustrated in Figure 28. Similar to the N_{design} = 125 Part 2 APA testing, all of the mixes except the FA-10 mix had similar laboratory performance. The worst-case FA-10 mix had significantly higher strain values than did the other eight mixes tested. Sufficient data was available to conduct a DMRT ranking within the FA-4, FA-7, and FA-9 mixes. Results of the three DMRT rankings indicate the permanent strain values for each gradation (with a given fine aggregate) are not significantly different. Interestingly, the CRZ gradation did show the highest magnitude permanent strain for both the FA-4 and FA-7 data although it was not significantly different. Based upon these Part 2 N_{design} = 125 performance data, it appears that the restricted zone is redundant with the Superpave volumetric and FAA value.

Figure 29 illustrates the results of RSCH testing on Part 2 mixes design with 75 gyrations. Unlike the Part 1 RSCH data (see Figure 18), there is some variation in test data among the mixes tested. Similar to the APA and RLCC testing conducted on mixes designed with 75 gyrations, the mixes containing fine aggregates with FAA values greater than 46 (i.e., FA-4, FA-6, and FA-7) had significantly less plastic strain than did those mixes using fine aggregates with FAA values less than 46 (i.e., FA-10, FA-2, and FA-3). The FA-10/HRZ, FA-2/TRZ, and FA-3/CRZ mixes had plastic strains approaching the limits measurable by the RSCH test (i.e., approximately 8 percent). The other four mixes—FA-4/TRZ, FA-6/BRZ, FA-6/CRZ, and FA-7/TRZ—all had plastic strains less than 3 percent.

There were sufficient FA-3 and FA-6 mixes to evaluate the restricted zone with the DMRT. Of these two, FA-3 had significant differences in plastic strain between the gradation violating the restricted zone (i.e., CRZ) and the gradation residing outside the zone (i.e., BRZ). The plastic strain for the FA-3/BRZ gradation was approximately 4 percent; the plastic strain for the FA-3/CRZ gradation was approximately 7 percent. Both of these mixes would be considered susceptible to permanent deformation based upon previous research. For the FA-6 combinations (i.e., BRZ and CRZ), results of the DMRT rankings suggested that the plastic strain values were similar.

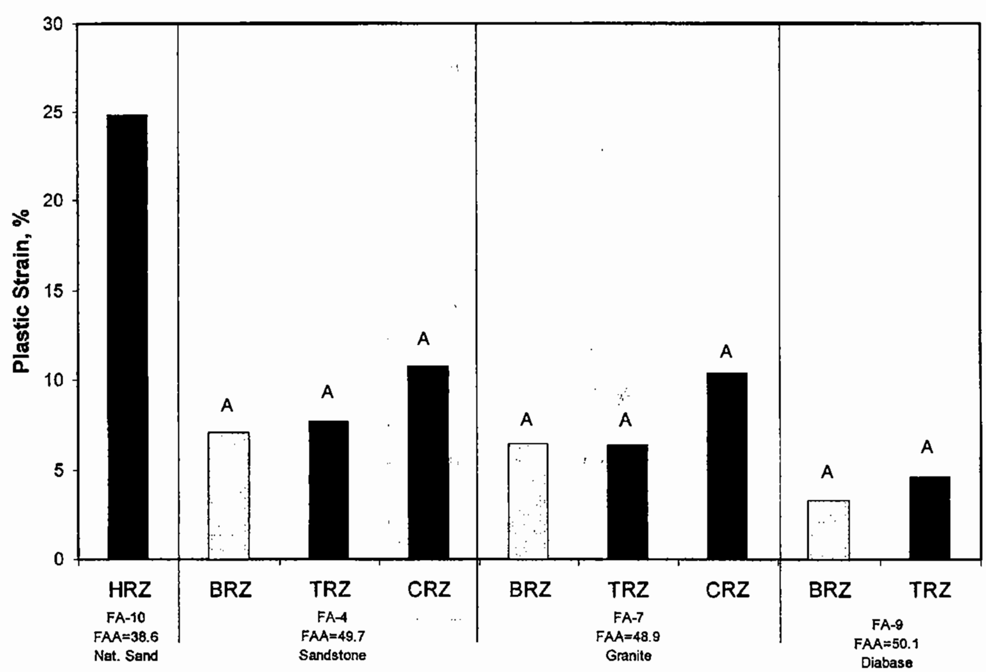

Figure 28. Results of RLCC testing on mixes designed with 125 gyrations for Part 2.

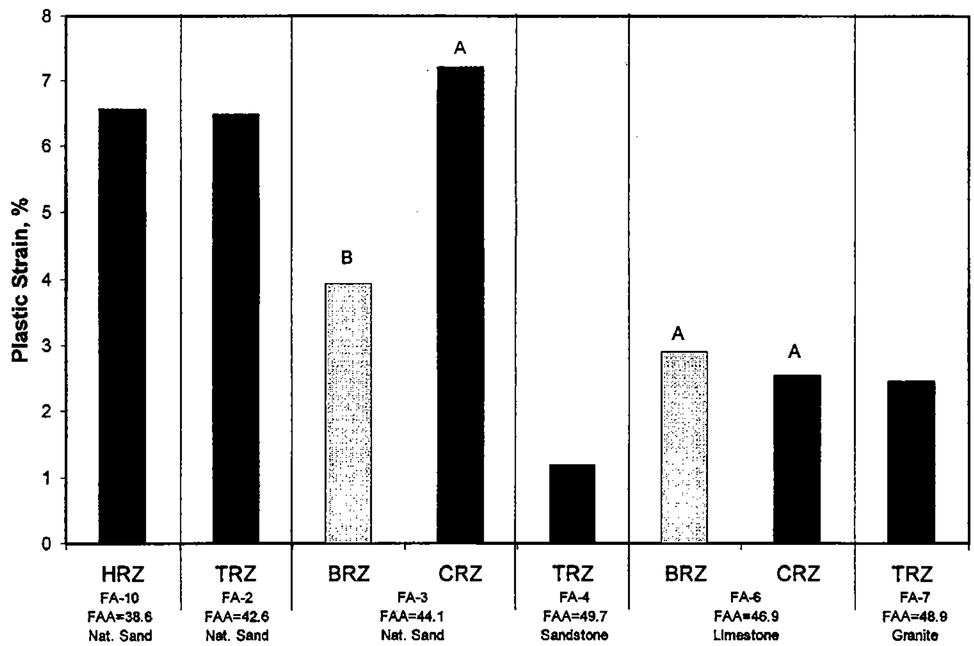

Figure 29. Results of RSCH testing on mixes designed with 75 gyrations for Part 2.

Similar to the APA and RLCC testing, the results shown in Figure 29 suggest that volumetric and FAA criteria are not adequate to ensure rut-resistant mixes when the $N_{design} = 75$ design compactive effort is used. The APA and RLCC test results indicated that the potential for rutting is not enhanced when gradations pass through the restricted zone. However, based upon the FA-3 RSCH data, the CRZ gradation (which violates the restricted zone) did show significantly higher potential for rutting.

Results of the RSCH testing conducted on Part 2 mixes designed with 125 gyrations are illustrated in Figure 30. The data illustrated in Figure 30 is very similar to that shown for

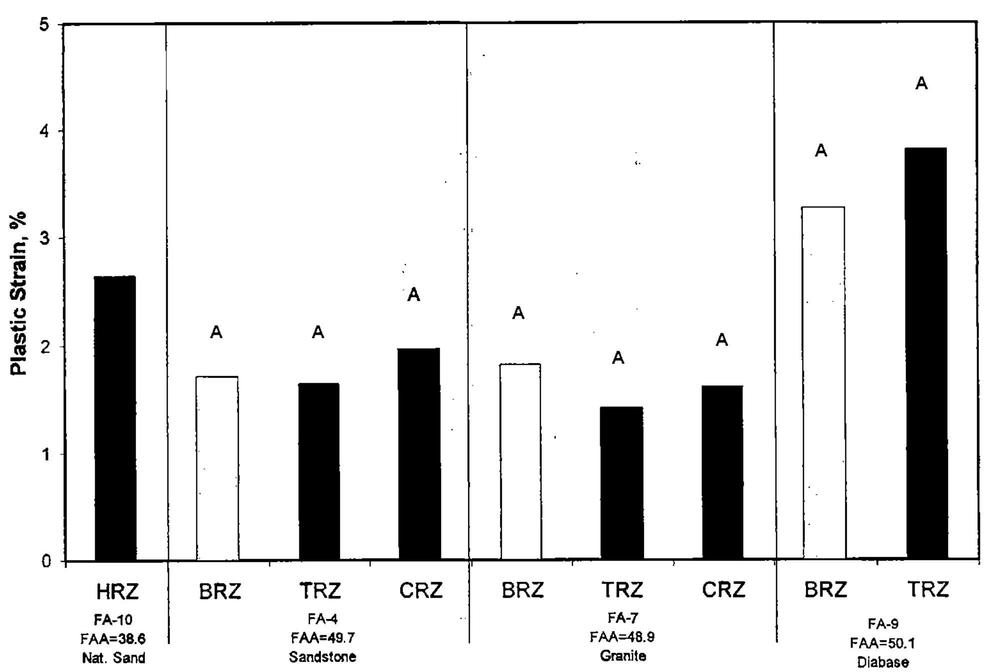

Figure 30. Results of RSCH testing on mixes designed with 125 gyrations for Part 2.

the Part 1 RSCH data (see Figure 18) in that mixes containing FA-9 had higher plastic strain values than did the worst-case FA-10. Besides the FA-9 data, all remaining data appear to be similar (including FA-10). Sufficient mix combinations were available to conduct the DMRT rankings for gradations prepared with FA-4, FA-7, and FA-9. In all instances, no significant differences were shown among the gradations. This suggests that the restricted zone is essentially redundant with the Superpave volumetric and FAA criteria for these high-traffic-volume mixes.

PART 3 TEST RESULTS AND ANALYSIS

As described in Chapter 2, Part 3 was a continuation of Parts 1 and 2, except that 19.0-mm NMAS gradations were used instead of 9.5-mm NMAS gradations. Four 19.0-mm NMAS gradations were included in Part 3: BRZ, TRZ, HRZ, and ARZ. The BRZ, TRZ, and ARZ gradations were used with all fine aggregates; the HRZ gradation was included only with fine aggregates having an FAA value of less than 45 percent. Both the granite and gravel coarse aggregates were included in Part 3. Two design compactive efforts were used, $N_{design} = 75$ and 100. During Parts 1 and 2, a number of mixes had excessive VFA (i.e., above 75 percent because of excessive VMA). In an effort to reduce the number of mixes excluded from performance testing because of excessive VFA, mixes designed with 75 gyrations used the gravel coarse aggregate while mixes designed at 100 gyrations used the granite coarse aggregate. Also different in Part 3 was the method of conducting mix designs. In Parts 1 and 2, mix designs were conducted on all factor-level combinations. During Part 3, for a given coarse aggregate/fine aggregate combination, mix designs were first conducted for the gradation(s) violating the restricted zone. If these mixes met all volumetric criteria, then mix designs were conducted for the control gradations.

A total of six fine aggregates were investigated for the 75-gyration design compactive effort and included FA-10, FA-2, FA-3, FA-4, FA-6, and FA-7. Results of these mix designs are presented in Appendix E. Six fine aggregates were also investigated for mixes designed with 100 gyrations and included FA-10, FA-2, FA-4, FA-6, FA-7, and FA-9. Results of these mix designs are also presented in Appendix E. Similar to Parts 1 and 2, in Part 3 the FA-10 fine aggregate was included as a worst-case baseline on performance.

Of the five experimental fine aggregates used with the 75-gyration design effort (excluding FA-10), three had gradations violating the restricted zone that met volumetric criteria (i.e., FA-2, FA-4, and FA-7). For the two fine aggregates not meeting volumetric criteria (i.e., FA-3 and FA-6), the VMA values were below the 13-percent minimum. Similar to the analysis in Parts 1 and 2, the effect of gradation on VMA and $\%G_{mm}@N_{initial}$ was evaluated for the 75-gyration design effort mixes. Included in this analysis were the fine aggregates in which all gradations were investigated (i.e., FA-2, FA-4, and FA-7). Because only three fine aggregates were included in this analysis, no comparisons were made between VMA or $\%G_{mm}@N_{initial}$ and FAA values.

Figure 31 illustrates the effect of gradation on VMA. This figure shows that the BRZ gradation provided much higher VMA values than did the TRZ, ARZ, or HRZ gradations. The TRZ and ARZ gradations provided somewhat

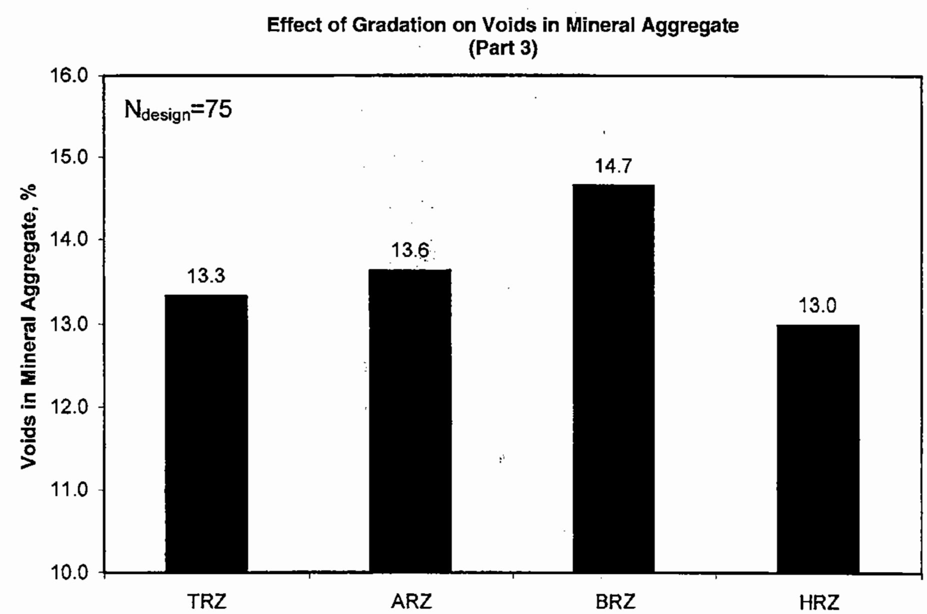

Figure 31. Effect of gradation on VMA ($N_{design} = 75$), Part 3.

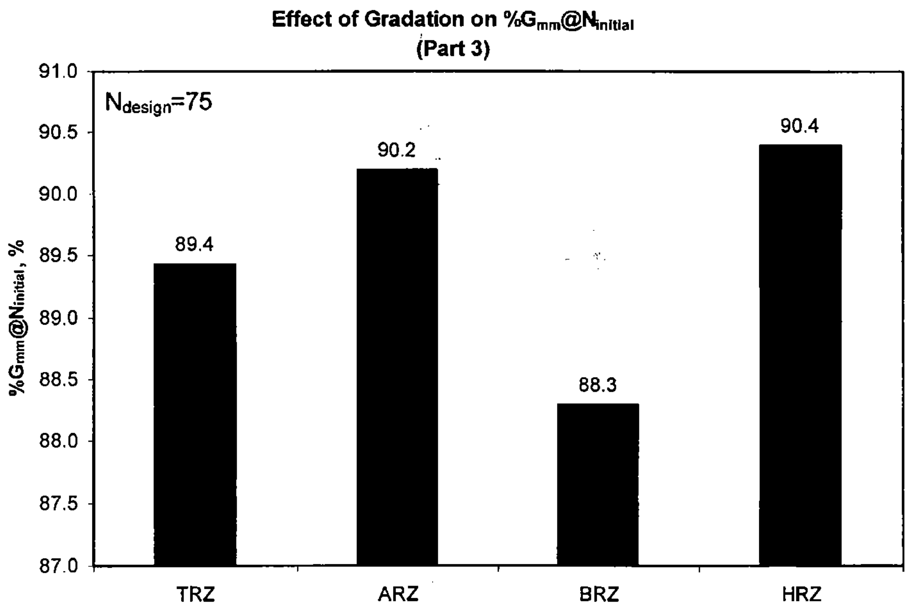

Figure 32. Effect of gradation on $\%G_{mm}@N_{initial}$ *(*$N_{design} = 75$*), Part 3.*

similar VMAs. Figure 31 suggests that the HRZ gradation provided the lowest VMA value; however, the HRZ gradation was only included with FA-2 (which had an FAA of less than 45 percent). For FA-2, the HRZ gradation provided approximately the same VMA (i.e., 13.0 percent) as the TRZ and ARZ gradations (i.e., 12.9 and 12.8 percent, respectively). These results are similar to those presented in Parts 1 and 2.

The effect of gradation on $\%G_{mm}@N_{initial}$ is illustrated in Figure 32. This figure shows that as the gradation becomes coarser, $\%G_{mm}@N_{initial}$ values decrease. The BRZ gradation had the lowest $\%G_{mm}@N_{initial}$, and the ARZ had the highest. These results are very similar to the results in Parts 1 and 2. The HRZ gradation did have a high $\%G_{mm}@N_{initial}$ value; however, a comparison of the FA-2 data suggests that the HRZ gradation had a similar $\%G_{mm}@N_{initial}$ value as did the TRZ gradation.

For the experimental fine aggregates designed at 100 gyrations, only two had gradations violating the restricted zone that met volumetric criteria: FA-7 and FA-9. Only the TRZ, ARZ, and BRZ gradations were included with these fine aggregates. The ARZ gradation used with FA-7 failed to meet the $\%G_{mm}@N_{initial}$ criteria of 89.0 percent maximum. Trends between VMA and gradation shape were similar for these $N_{design} = 100$ mixes to those for Parts 1 and 2 and the lower compactive effort mixes used in Part 3. The BRZ gradation provided the highest average VMA value at 15.1 percent followed by the ARZ gradation (14.2 percent) and TRZ gradation (13.9 percent). Trends between $\%G_{mm}@N_{initial}$ and gradation shape were also similar to previous analyses in that the coarser the gradation, the lower the $\%G_{mm}@N_{initial}$ value. BRZ had the lowest average $\%G_{mm}@N_{initial}$ value at 87.1 percent, and ARZ had the highest at 89.1 percent; at 87.6 percent, the TRZ gradation fell between the BRZ and the ARZ.

Results of performance testing conducted in Part 3 are also presented in Appendix E. For Part 3, the APA was used as the only performance test because in Parts 1 and 2 the APA appeared to be more sensitive to changes in gradation. APA results for mixes designed with 75 gyrations in Part 3 are illustrated in Figure 33. Rut depths for gradations that violate the restricted zone are shown with solid black bars; rut depths for control gradations are shown as unshaded bars. As expected, the mix containing FA-10 had a high rut depth. However, the FA-2/BRZ gradation had a slightly higher rut depth. The remaining mixes shown in Figure 33 had similar rut depths. Sufficient data was available for FA-2, FA-4, and FA-7 to conduct DMRT rankings. For FA-4 and FA-7, all of the gradations had similar rankings, which suggests the gradations violating the restricted zone did not result in mixes more susceptible to rutting. The FA-2 mixes did show significantly different rut depths for the two mixes tested. The control gradation (i.e., BRZ) had a significantly higher rut depth than did the gradation violating the restricted zone (i.e., HRZ). Based upon these data for 19.0-mm NMAS designed with 75 gyrations, it appears that gradations passing through the restricted zone will provide comparable, if not better, rut resistance when compared with gradations passing outside the restricted zone.

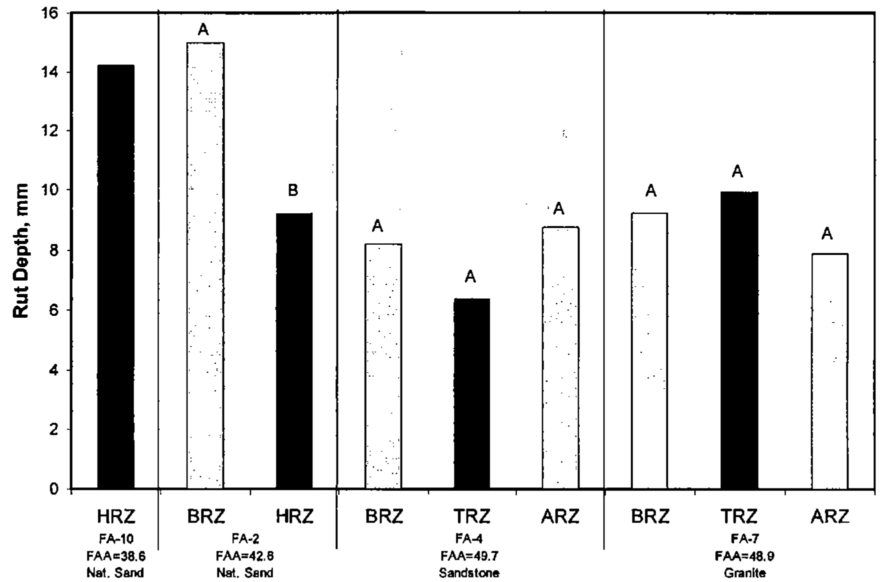

Figure 33. Results of APA testing conducted on mixes designed with 75 gyrations, Part 3.

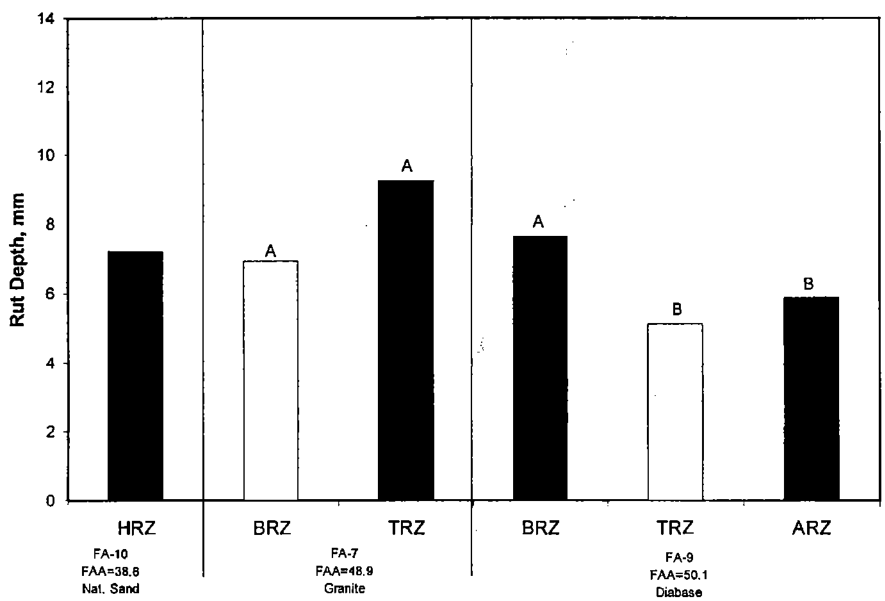

Figure 34. Results of APA testing conducted on mixes designed with 100 gyrations in Part 3.

Results of APA testing conducted on mixes designed with 100 gyrations for Part 3 are illustrated in Figure 34. Sufficient data was available to conduct DMRT rankings for mixes containing FA-7 and FA-9. Mixes containing FA-7 (i.e., BRZ and TRZ) had similar rut depths based upon the DMRT rankings. For the FA-9 mixes, the BRZ gradation (i.e., the control) had a significantly higher rut depth than did the TRZ and ARZ gradations. This data supports the previous analyses in Parts 1 and 2 and the analysis of the lower design compactive effort work in Part 3. Mixes having gradations passing through the restricted zone perform similarly or better than mixes having gradations passing outside the restricted zone.

CHAPTER 4

CONCLUSIONS, RECOMMENDATIONS, AND SUGGESTED RESEARCH

CONCLUSIONS

The following conclusions are drawn from the analysis of data presented in Chapter 3.

1. Mixes meeting Superpave and FAA requirements with gradations that violated the restricted zone performed similarly to or better than the mixes with gradations passing outside the restricted zone. This conclusion is drawn from the results of experiments with 9.5- and 19-mm NMAS gradations at N_{design} values of 75, 100, and 125 gyrations and is supported by extensive, independent results from the literature.
2. The restricted zone requirement is redundant for mixes meeting all Superpave volumetric parameters and the required FAA. References to the restricted zone, as either a requirement or a guideline, should be deleted from the AASHTO specifications and practice for Superpave volumetric design for HMA, regardless of NMAS or traffic level. Some agencies have used the restricted zone to differentiate between coarse- and fine-graded Superpave mixtures. Because the term "restricted zone" will be deleted, research needs to be done to differentiate and define coarse- and fine-gradations, if desired.
3. Although not germane to the primary objective of this project, the following observations were made:
   - Coarse-aggregate type has a significant effect on the VMA of mixes. Coarse, angular granite aggregate generally produced a higher VMA than did the coarse, crushed gravel aggregate.
   - Coarse-aggregate type has a significant effect on $\%G_{mm}@N_{initial}$ values. However, fine-aggregate type and gradation appear to have more significant effects.
   - ARZ and CRZ gradations tend to provide higher VMA values; the TRZ gradation provided the lowest VMA values.
   - The TRZ gradations generally provide the lowest VMA values for both the 9.5- and 19.0-mm NMAS mixes. This result suggests that the MDL drawn according to the Superpave guidelines (connecting the origin of the 0.45 power chart to the 100-percent passing the maximum aggregate size) is located reasonably on the gradation chart.
   - Relatively finer gradation mixes (such as ARZ and HRZ) tend to have higher $\%G_{mm}@N_{initial}$ values compared with the values of TRZ, CRZ, and BRZ mixes.
   - High FAA values do not necessarily produce high VMA in mixes although there was a general trend of increasing VMA values for increasing FAA.
   - Higher FAA values generally produced lower $\%G_{mm}$ $@N_{initial}$ values. None of the mixes having an FAA value lower than 45 met the $\%G_{mm}@N_{initial}$ requirements of 89 percent and lower for the mixes prepared at N_{design} =100 and 125. This indicates that high FAA values contribute to a stiffer fine aggregate/asphalt component in HMA at initial compaction levels.
   - None of the mixes failed the $\%G_{mm}@N_{maximum}$ requirement of 98 percent maximum. In the future, the validity of this requirement should be examined.
   - Numerous mix designs in this study exceeded the maximum VFA requirement of 75 percent. The Superpave requirement of 65.0 to 75.0 percent for VFA effectively limits the VMA of 9.5-mm NMAS mixes to a narrow range. Both VMA and VFA requirements for 9.5-mm NMAS Superpave mix design need to be evaluated.
   - The potential of mixes failing because of excessive VMA (i.e., more than 2 percent above the minimum specified value) increases with a lower design compactive effort, angular coarse aggregate content, and high FAA values.
   - Both the APA and the RLCC test were reasonably sensitive to the gradation of mixes. The RSCH test conducted with the Superpave shear tester was not found to be as sensitive to changes in gradation.

RECOMMENDATIONS

The primary objective of this research project was to determine under what conditions, if any, compliance with the restricted zone requirement is necessary when an asphalt paving mix meets all other Superpave requirements such as FAA and volumetric mix criteria (such as VMA) for a project. The results of the study demonstrated that the restricted zone is redundant in all conditions (such as NMAS and traffic levels) when all other relevant Superpave volumetric mix and FAA requirements are satisfied. Therefore, all reference to the restricted zone in AASHTO MP2-00 and AASHTO PP28-00 should be deleted thoroughly to avoid any confusion in implementation.

The following specific revisions to AASHTO MP2-00, "Standard Specification for Superpave Volumetric Mix Design," are recommended:

- Delete Section 6.1.3, which reads: "Gradation Restricted Zones—It is recommended that the selected combined aggregate gradation does not pass through the restricted zones specified in Table 3. See Figure 1 for an example of a graph showing the gradation control points and the restricted zone."
- Delete Table 3: Boundaries of Aggregate Restricted Zone.
- Renumber Table 4 as Table 3, and Table 5 as Table 4.
- Sections 6.2, 6.3, 6.4, and 6.5: change "Table 4" to "Table 3."
- Section 7.2: change "Table 5" to "Table 4."
- Figure 1: delete the words "and Restricted Zone" from the title. Erase or remove the illustration of the restricted zone from the figure.

The following revisions to AASHTO PP28-00, "Standard Practice for Superpave Volumetric Design for Hot-Mix Asphalt (HMA)," are recommended:

- Section 6.8: revise "confirm that each trial blend meets MP2 gradation control (see Tables 2 and 3 of MP2)" to read as follows: "confirm that each trial blend meets MP2 gradation control (see Table 2 of MP2)."
- Figure 1: remove the illustration of the restricted zone from the figure.

SUGGESTED RESEARCH

Table 3 of AASHTO MP2-00 presents gradation restricted zones for five NMAS mixtures: 9.5-mm, 12.5-mm, 19.0-mm, 25.0-mm, and 37.5-mm. Section 6.1.3 states "It is recommended that the selected combined aggregate gradation does not pass through the restricted zones specified in Table 3."

AASHTO PP28-00 specifies four design compaction levels (N_{design}) of 50, 75, 100, and 125 gyrations corresponding to four design ESALs of < 0.3 million, 0.3 to < 3 million, 3 to < 30 million, and ≥ 30 million, respectively.

Ideally, then, the necessity of the restricted zone for five NMAS and four traffic levels (i.e., $5 \times 4 = 20$ combinations) should be evaluated. This would be a monumental task and is considered unnecessary by the research team. Besides this project (NCHRP Project 9-14), various researchers have already evaluated the restricted zone in NMAS ranging from 9.5 mm to 37.5 mm and N_{design} ranging from 75 to 152 gyrations. Table 12 gives this information; the work is reviewed in detail in Appendix A. This body of research clearly shows the redundancy of the restricted zone for various NMAS and traffic levels listed in Table 12.

There does not appear any need for conducting additional research pertaining to the design compaction level of 50 gyrations because those mixes are used for light-traffic-volume roads. This leaves N_{design} of 75, 100, and 125 gyrations to be researched. Table 13 presents the NMAS mixes that have been evaluated at compactive efforts of 75 gyrations and higher.

TABLE 12 NMAS and compactive efforts evaluated by researchers

Researchers*	NMAS	N_{design} (Gyrations)
NCHRP Project 9-14	9.5 mm	75, 100, and 125
	19.0 mm	75 and 100
McGennis (1997)	19.0 mm	96
Anderson and Bahia (1997)	19.0 mm	109
Sebaaly et al. (1997)	19.0 mm	Hveem design
Van de Ven et al. (1997)	9.5 mm	142
	12.5 mm	142
El-Basyouny and Mamlouk (1999)	19.0 mm	113
	37.5 mm	113
Kandhal and Mallick (2001)	12.5 mm	76
	19.0 mm	76
Chowdhury et al. (2001)	19.0 mm	86 and 96
Hand et al. (2001)	9.5 mm	76, 109, and 152
	19.0 mm	76, 109, and 152

* See Reviews, Appendix A.

TABLE 13 Evaluations by researchers

NMAS	N_{design} Gyrations
9.5 mm	75, 76, 100, 109, 125, and 142
12.5 mm	76, 142
19.0 mm	75, 76, 86, 96, 100, 109, 113, and 152
25.0 mm	None
37.5 mm	113

Table 13 shows that all NMAS mixes except 25.0-mm, which is used primarily in HMA base courses, have been evaluated. If the restricted zone is redundant for 19.0-mm and 37.5-mm NMAS mixes, it is probable it will also be redundant for the intervening 25.0-mm NMAS mix as well. The Georgia DOT's 25.0-mm NMAS base mix has gradation that overlaps a small portion of the Superpave restricted zone. According to Watson et al. (*3*) the average rut depth (measured by the GLWT) obtained on the base mix was 2.6 mm, the lowest of all the mixes used by Georgia DOT; this indicates the redundancy of the restricted zone for 25.0-mm NMAS mixes. The research team is of the opinion that no further research work on the restricted zone is necessary and that the zone should be considered redundant for all NMAS mixes. However, if it is strongly believed that the research team should fill the research gaps, the following combinations of NMAS and N_{design} are suggested:

NMAS	N_{design} Gyrations
12.5 mm	100
25.0 mm	75 and 100

It is recommended to use the APA only for performance testing because it was observed to be the most sensitive to change in gradation of the three test procedures used in NCHRP Project 9-14. At least six fine aggregates covering a wide range of FAA values should be used. It is recommended to use crushed gravel coarse aggregate for an N_{design} of 75 gyrations and granite coarse aggregate for an N_{design} of 100 gyrations similar to the work plan for Part 3. This will increase the potential of obtaining HMA mixes that will meet the minimum VMA requirements. The cost of this additional research work is estimated to be $200,000.

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FOREWORD

By Edward T. Harrigan
Senior Program Officer
Transportation Research Board

This report presents recommended guidelines for hot mix asphalt pavement construction to achieve satisfactory levels of in-place air voids and permeability. These guidelines were developed from the findings of a research project that examined the relationship of air voids content to permeability and hot mix asphalt lift thickness. The report will be of particular interest to materials and construction engineers in state highway agencies, as well as to materials supplier and paving contractor personnel responsible for the production and placement of hot mix asphalt.

For satisfactory performance, hot mix asphalt (HMA) pavements must be constructed with adequate field density and impermeability to moisture. During the transition to the use of the Superpave mix design method since 1994, several states reported problems with greater than expected permeability associated with the use of coarse-graded mixes. In addition, there has been ongoing debate over the in-place air voids content and layer thickness needed to ensure an impermeable pavement. Some state highway agencies have addressed these issues by increasing their field density requirements, lift thickness requirements, or both, when coarse-graded mixes are used. Such changes, however, entail increased expense. So other states have elected (1) to reduce the nominal maximum aggregate size of given lifts (e.g., use of a 19.0-mm in place of a 25.0-mm mix) or (2) to eliminate pavement layers (such as a binder layer) and increase the thickness of the remaining layers to keep the total pavement thickness at typically used levels. However, many agencies are reluctant to adopt any such change without the support of specific research results that justify the increased cost or provide evidence of satisfactory long-term performance.

Under NCHRP Project 9-27, "Relationships of HMA In-Place Air Voids, Lift Thickness, and Permeability," the National Center for Asphalt Technology (NCAT) at Auburn University was assigned the tasks of (1) determining the minimum ratio of layer thickness, t, to nominal maximum aggregate size, NMAS, needed to achieve desirable pavement density levels, and thus impermeable pavements; (2) evaluating the permeability characteristics of different thicknesses of compacted HMA; and (3) assessing factors affecting the relationship between in-place air voids, permeability, and lift thickness. To accomplish these tasks, the research team (1) conducted a critical review of the literature on the relationship of HMA lift thicknesses to in-place air voids, the relationship of in-place air voids to permeability, and their effects on pavement performance; (2) evaluated current state DOT guidelines and requirements for minimum lift thickness and minimum in-place density; and (3) designed and carried out coordinated laboratory and field experiments to establish relationships among air voids, lift thickness, and permeability from which to develop practical field compaction guidelines.

The NCAT project team found that the HMA pavement density that can be obtained under normal rolling conditions is clearly related to the ratio t/NMAS of the

HMA. For improved compactibility, the agency recommended that t/NMAS be at least 3 for fine-graded mixes and at least 4 for coarse-graded mixes. The data for SMA mixes indicate that the ratio should also be at least 4. Ratios less than these suggested values can be used but a greater than normal compactive effort will generally be required in these situations to obtain the desired in-place density.

The results of an experiment to evaluate the effect of mix temperature on the relationship between pavement density and t/NMAS found that the more rapid cooling of the HMA is a key reason for low density in thinner sections (lower t/NMAS). Hence, for thin HMA layers NCAT emphasized the importance of paving rollers staying very close to the paving machine so that rolling can be accomplished prior to excessive cooling.

The project team further identified the in-place air voids content as the most significant factor impacting permeability of HMA mixtures, followed by coarse aggregate ratio and VMA. As the coarse aggregate ratio increases, permeability increases, but it decreases as VMA increases at constant air voids content. The variability of permeability between various mixtures is very high; some mixtures are permeable in the range of 8 to 10 percent air voids while others are not. However, to ensure that permeability is not a problem NCAT recommends an in-place air voids content between 6 and 7 percent or lower. This appears to be true for a wide range of mixtures regardless of NMAS and aggregate gradation.

The project final report presents detailed descriptions of the coordinated laboratory (Task 3) and field (Task 5) experiments; a discussion of the research results from both experiments; and the project findings, conclusions, and recommendations in five volumes:

- Volume I: Task 3—Parts 1 and 2;
- Volume II: Task 3—Part 3;
- Volume III: Task 5;
- Volume IV: Appendices for Volumes I, II, and III; and
- Volume V: Executive Summary.

This report includes Volume V only; Volumes I through IV will be available online at http://www4.trb.org/trb/onlinepubs.nsf/web/nchrp_web_documents as NCHRP Web Document 68.

The recommended guidelines from Project 9-27 have been referred to the TRB Mixtures and Aggregate Expert Task Group for its review and possible recommendation to the AASHTO Highway Subcommittees on Materials and Construction for revision of appropriate specifications and recommended practices.

CONTENTS

CHAPTER 1

INTRODUCTION AND PROBLEM STATEMENT

Proper compaction of hot mix asphalt (HMA) mixtures is vital to ensure that a stable and durable pavement is built. For dense-graded mixes, numerous studies have shown that initial in-place air voids should not be below approximately 3 percent nor above approximately 8 percent (*1*). Lower percentages of in-place air voids can result in rutting and shoving, while higher percentages allow water and air to penetrate into the pavement, leading to an increased potential for water damage, oxidation, raveling, and cracking. Low in-place air voids are generally the result of a mix problem while high in-place voids are generally caused by inadequate compaction.

Many researchers have shown that increases in in-place air void contents have meant increases in pavement permeability. Zube (*2*) showed in the 1960s that dense-graded pavements become excessively permeable when in-place air voids exceed 8 percent. Brown et al. (*3*) later confirmed this value during the 1980s. However, due to problems associated with coarse-graded mixes (those with a gradation passing below the maximum density line), the size and interconnectivity of air voids have been shown to greatly influence permeability. A study conducted by the Florida Department of Transportation (FDOT) (*4*) indicated that coarse-graded Superpave mixes can sometimes be excessively permeable to water even when in-place air voids are less than 8 percent.

Permeability is also a major concern in stone matrix asphalt (SMA) mixes that utilize a gap-graded coarse gradation. Data have shown that SMA mixes tend to become permeable when air voids are above approximately 6 percent.

Numerous factors can potentially affect the permeability of HMA pavements. In a study by Ford and McWilliams (*5*), it was suggested that particle size distribution, particle shape, and density (air voids or percent compaction) affect permeability. Hudson and Davis (*6*) concluded that permeability is dependent on the size of air voids within a pavement, not just the percentage of voids. Research by Mallick et al. (*7*) has also shown that the nominal maximum aggregate size (NMAS) and lift thickness for a given NMAS affect permeability.

Work by FDOT indicated that lift thickness can have an influence on density and hence permeability (*8*). FDOT constructed numerous pavement test sections on Interstate 75 that included mixes of different NMAS and lift thicknesses. Results of this experiment suggested that increased lift thicknesses could lead to better pavement density and hence lower permeability.

Thus permeability, lift thickness, and air voids are all interrelated. Permeability has been shown to be related to pavement density (in-place air voids). Increased lift thickness has been shown to allow desirable density levels to be more easily achieved. Westerman (*9*), Choubane et al. (*4*), and Musselman et al. (*8*) have suggested that a thickness to NMAS ratio (t/NMAS) of 4.0 is preferred. Most guidance recommends that a minimum t/NMAS of 3.0 be used (*10*). However, due to the potential problems of achieving the desired density, it is believed that this ratio should be further evaluated based on NMAS, gradation, and mix type (Superpave and SMA).

CHAPTER 2

OBJECTIVE

The objectives of NCHRP 9-27 were to (1) determine the minimum t/NMAS needed for desirable impermeable pavement density levels to be achievable, (2) evaluate the permeability characteristics of compacted samples at different thicknesses, and (3) evaluate factors affecting the relationship among in-place air voids, permeability, and lift thickness.

CHAPTER 3

RESEARCH APPROACH

The laboratory evaluation of the relationship between thickness, density, and permeability was divided into two parts. Part 1 evaluated the relationship of lift thickness, air voids, and permeability in a controlled, statistically designed experiment. This part looked at varying the lift thickness in the gyratory compactor and determining density; the experimental variables included three aggregates, four gradations, three nominal aggregate sizes for Superpave mixes, and three nominal aggregate sizes for SMA mixes. The aggregate properties are shown in Table 1. Only one asphalt binder was used for this study, a PG 64-22. After the mix designs were performed for these mixes, they were compacted in the Superpave gyratory compactor (100 gyrations) to heights of 2.0, 3.0, and 4.0 times the t/NMAS. The effect of t/NMAS on density was then determined. The plan was to select the t/NMAS that gave optimum density; but, as will be shown later, the results from the Superpave gyratory compactor data did not provide a conclusive answer; hence, additional work was needed to better establish the appropriate ratio.

It was then decided to look at many of the same mixes with a vibratory compactor, to establish whether the vibratory compactor would better simulate field compaction and would provide more conclusive results The experimental variables included two aggregates, three gradations, two nominal aggregate sizes for Superpave, and three nominal aggregate sizes for SMA. These mixtures, which had already been designed in the first part, were compacted at three thicknesses using three compactive efforts with the vibratory compactor. The density results were determined, and again the results did not identify a definitive minimum ratio. It was then decided that additional work was needed if an acceptable answer was to be obtained.

The third attempt at the effect of t/NMAS on compaction was to look at a field study during the rebuilding of the National Center for Asphalt Technology (NCAT) test track. During this work, the layer thicknesses were varied and compacted under similar conditions. Seven mixes from the track were constructed on a paved surface adjacent to the track to look at the effect of layer thickness on density. A general description of these seven mixtures is provided in Table 2. For this part of the study, seven mixes were compacted at layer thicknesses varying from two to five times the t/NMAS. For some of these seven mixes, one side was compacted with a vibratory roller and the other sided was compacted with vibratory and rubber tire rollers. The test data were evaluated, as shown later, and provided reasonable results.

Another part of the study for Part 1 looked at the effect of lift thickness on permeability. The air voids were controlled at 7 percent and the thickness varied. The permeability results were then determined. These variables were evaluated: two aggregate types, three gradations, two Superpave NMAS, three SMA NMAS, and three t/NMAS.

Part 2 of Task 3 looked at the permeability of cores obtained from the NCHRP 9-9 project. This project contained 40 sections with varying aggregate types, NMASs, thicknesses, and design gyrations. The results were evaluated to determine the effect of gradation, NMAS, thickness, and design gyration on permeability. It was assumed that this information would help to determine the in-place air voids at which permeability would become a problem. Both field and lab permeability were measured.

TABLE 1 Physical properties of aggregate

Property		Test Method	Aggregate Type		
			Granite	Limestone	Crushed Gravel
Coarse Aggregate					
Bulk Specific Gravity		AASHTO T-85	2.654	2.725	2.585
Apparent Specific Gravity		AASHTO T-85	2.704	2.758	2.642
Absorption (%)		AASHTO T-85	0.7	0.4	0.9
Flat and Elongated (%), 3:1, 5:1	19.0 mm	ASTM D4791	14, 0	10, 0	4, 0
	12.5 mm		16, 0	6, 0	16, 2
	9.0 mm		9, 1	16, 3	19, 2
Los Angeles Abrasion (%)		AASHTO T-96	37	35	31
Coarse Aggregate Angularity (%)		AASHTO TP56-99	42.9	43.0	44.0
Percent Crushed (%)		ASTM D5821	100	100	80
Fine Aggregate					
Bulk Specific Gravity		AASHTO T-84	2.678	2.689	2.610
Apparent Specific Gravity		AASHTO T-84	2.700	2.752	2.645
Absorption (%)		AASHTO T-84	0.3	0.9	0.5
Fine Aggregate Angularity (%)		AASHTO T-33 (Method A)	49.4	45.7	48.8
Sand Equivalency (%)		AASHTO T-176	92	93	94

TABLE 2 Mix information for field density study

Section	NMAS	Gradation	Asphalt Type	Aggregate Type
1	9.5 mm	Fine-Graded Superpave	Unmodified	Granite and Limestone
2	9.5 mm	Coarse-Graded Superpave	Unmodified	Limestone
3	9.5 mm	SMA	Modified	Granite
4	12.5 mm	SMA	Modified	Limestone
5	19.0 mm	Fine-Graded Superpave	Unmodified	Granite and Limestone
6	19.0 mm	Coarse-Graded Superpave	Unmodified	Granite
7	19.0 mm	Coarse-Graded Superpave	Modified	Limestone

CHAPTER **4**

TEST RESULTS AND ANALYSIS

4.1 PART 1—MIX DESIGNS FOR SPECIMENS TO STUDY THE EFFECT OF t/NMAS ON DENSITY

Of the 36 mix designs, 27 were Superpave-designed mixes and 9 were SMA mixes. The Superpave mixes were classified according to three gradations: above the restricted zone (ARZ), through the restricted zone (TRZ), and below the restricted zone (BRZ). The optimum asphalt content, the effective asphalt content (P_{be}), voids in mineral aggregate (VMA), voids filled with asphalt (VFA), percent theoretical maximum density at $N_{initial}$ (% G_{mm} at N_{ini}), and ratio of dust to effective asphalt content ($P_{0.075}/P_{be}$) for the Superpave mixes are summarized in Table 3. Data for SMA mixes are shown in Table 4. The mix design information for both mix types is presented in Appendix A. Optimum asphalt binder content was chosen to provide 4 percent air voids at the design number of gyrations. However, for the 19-mm NMAS limestone SMA mix, 4 percent air voids could be achieved with 5.7 percent asphalt content, which did not meet the minimum asphalt content requirement in accordance with the "Standard Practice for Designing SMA," AASHTO PP44-01. Therefore, the minimum asphalt content of 6.0 percent was chosen, which resulted in 3.7 percent air voids at the design number of gyrations. Some designs did not meet the requirements of VMA, VFA, % G_{mm} at N_{ini}, and/or dust/P_{be}. Efforts were made to redesign the respective mixes by changing the gradation until the requirements were met or closely approximated. This is important in that the mixes used in this project were intended to duplicate mixes utilized in the field. No modification was made for the TRZ mixes that did not meet the requirements, as little could be done to modify these gradations and still pass through the restricted zone.

4.2 EVALUATION OF EFFECT OF t/NMAS ON DENSITY USING GYRATORY COMPACTOR

Before the evaluation was done, two methods of measuring density, or bulk specific gravity, were compared: the AASHTO T166 (SSD) and the vacuum sealing (ASTM D6752-02a) methods. All samples were measured using both methods. Figures 1 through 4 present these measurements for the three gradations of Superpave mixes and the SMA mixes.

As shown in Figure 1, the air voids for ARZ mixes as measured by the two methods are approximately equal at low air voids and deviate by approximately 0.5 percent at the highest air void level. This figure indicates that for ARZ mixes, the two methods provide similar results. For the TRZ, BRZ, and SMA mixes, Figures 2 through 4 suggest that the bulk specific gravity measurements derived from the two methods moved farther apart as density decreased. The results also indicate that, as the gradation became coarser, the difference in the test results for the two test methods increased. This finding agrees with the research by Cooley et al. (*11*).

The apparent reasons for the different results according to the two test methods is loss of water during density measurement when using the T-166 method and the effect of surface texture. The loss of water when blotting in the T-166 method causes a test error resulting in higher measured density. The surface texture can result in the vacuum seal device measuring a lower density than the actual density. Because the vacuum seal device is more accurate in measuring the density of porous samples, it was used to determine density for this research project.

The main objective of this part of the study was to determine the minimum t/NMAS. To achieve this objective, relationships of average air voids for the three aggregate types versus t/NMAS with respect to NMAS and gradation were evaluated; the results are illustrated in Figures 5 through 10. Originally it was intended to determine the t/NMAS at which the air voids began to level out and to pick that t/NMAS level as the minimum level recommended to achieve optimum compaction. However much of the data in Figures 5 through 10 indicate that the air voids continue to drop with increasing t/NMAS past typical t/NMAS values. These data therefore did not provide reasonable guidance for selecting a minimum t/NMAS. Hence an air void content of 7.0 percent was selected as the criteria to determine the minimum t/NMAS. This level of air voids was selected because compaction of most pavements in the field is targeted at 92.0 to 94.0 percent of theoretical maximum density. Because of the uncertainty in the relationship of average air voids to t/NMAS, as indicated by the data, it was determined to compact some laboratory samples with a vibratory compactor and also to compact some mixes in the field during reconstruction of the NCAT test track. These two efforts, which are discussed later in the report, should provide sufficient information to make reasonable conclusions concerning desired t/NMAS levels.

One potential problem with the Superpave gyratory compactor is that it applies a constant strain to the mix during compaction and the force required varies as necessary to provide the desired strain. This is not the approach that is observed in

TABLE 3 Summary of mix design results for Superpave mixes

Aggregate	NMAS, mm	Gradation	Optimum Asphalt, %	P_{be}, %	VMA %	VFA %	% G_{mm} at N_{ini}	$P_{0.075}/P_{be}$
Granite	9.5	ARZ	6.7	6.2	18.4	76	89.0	0.8
	9.5	BRZ	5.3	4.9	15.7	73	86.7	1.0
	9.5	TRZ	5.4	5.0	15.6	75	88.9	1.0
	19.0	ARZ	4.7	4.3	14.1	72	89.5*	1.2
	19.0	BRZ	4.4	3.9	13.3	68	86.0	1.0
	19.0	TRZ	4.0	3.6	12.5*	68	88.8	1.4*
	37.5	ARZ	4.2	4.0	13.7	69	89.8*	0.8
	37.5	BRZ	3.3	3.0	11.3	64	86.8	1.0
	37.5	TRZ	3.6	3.3	12.0	65	88.1	0.9
Gravel	9.5	ARZ	6.7	6.5	18.3	78*	88.4	0.8
	9.5	BRZ	6.2	5.6	16.7	75	86.5	0.8
	9.5	TRZ	6.0	5.4	16.3	75	87.7	0.9
	19.0	ARZ	4.9	4.4	14.0	72	88.5	1.1
	19.0	BRZ	4.5	3.9	12.9*	69	86.3	1.3*
	19.0	TRZ	4.4	3.8	12.8*	69	88.0	1.3*
	37.5	ARZ	4.4	3.9	13.0	70	89.7*	0.8
	37.5	BRZ	3.6	3.2	11.7	63	85.5	1.0
	37.5	TRZ	3.9	3.5	12.0	66	85.6	0.9
Limestone	9.5	ARZ	6.0	5.7	17.4	76	87.8	0.7
	9.5	BRZ	5.0	4.6	15.3	72*	85.5	0.9
	9.5	TRZ	4.4	4.2	14.4	70*	86	1.2
	19.0	ARZ	4.1	3.5	12.6*	66	88.3	1.4*
	19.0	BRZ	4.7	4.4	14.3	71	85.5	0.7
	19.0	TRZ	3.3	2.8	11.0*	62*	85.7	1.8*
	37.5	ARZ	3.2	3.1	11.8	64	88.8	1.0
	37.5	BRZ	2.7	2.6	10.6*	60*	86.0	1.2
	37.5	TRZ	2.8	2.6	10.6*	61*	87.7	1.1

* Did not meet Superpave Design Requirements

the field where the stress is constant and the strain varies. Hence, the Superpave gyratory compactor likely does not provide a reasonable answer because the compaction provided by this device is different from the field. The big problem with using this concept to establish a minimum t/NMAS is that the voids continue to increase significantly as the t/NMAS increases, making it impossible to select an optimum value.

The optimum t/NMASs established using the Superpave gyratory compactor vary from less than 2.5 up to approximately 8. This wide range of numbers did not allow specific criteria to be established. Hence, additional testing was performed using the laboratory vibratory compactor and field test section.

4.3 EVALUATION OF EFFECT OF t/NMAS ON DENSITY USING VIBRATORY COMPACTOR

After obtaining the results for the Superpave gyratory compactor, it was concluded that more tests needed to be conducted to better simulate compaction in the field. The air voids determined from the vacuum seal device were utilized in the analysis. To further evaluate the relationship between density and lift thickness, a similar study was conducted, but on a smaller scale, using the vibratory compactor as the compaction mode. This was not part of the original proposed work, but it was believed that the vibratory compactor might provide compaction that has more typical of in-place compaction.

TABLE 4 Summary of mix design results for SMA mixes

Aggregate	NMAS, mm	Optimum Asphalt, %	P_{be}, %	VMA, %	VFA, %	VCA_{mix},[a] %	VCA_{drc},[b] %
Granite	9.5	7.2	6.6	18.7	78	30.9	41.9
	12.5	6.6	6.4	18.8	77	30.3	42.7
	19.0	6.4	5.9	17.6	77	29.6	42.0
Gravel	9.5	7.3	6.5	18.6	77	30.4	41.8
	12.5	6.8	6.1	17.7	77	31.1	42.1
	19.0	6.7	6.2	17.8	76	29.3	42.0
Limestone	9.5	6.2	5.8	17.4	76	30.7	38.4
	12.5	7.4	7.0	19.6	80	31.1	38.9
	19.0	6.0	5.6	16.8[c]	77	29.8	40.3

[a]VCA = Voids in Compacted Aggregate
[b]drc = dry-rodded compacted
[c]Did not meet SMA Design Requirements

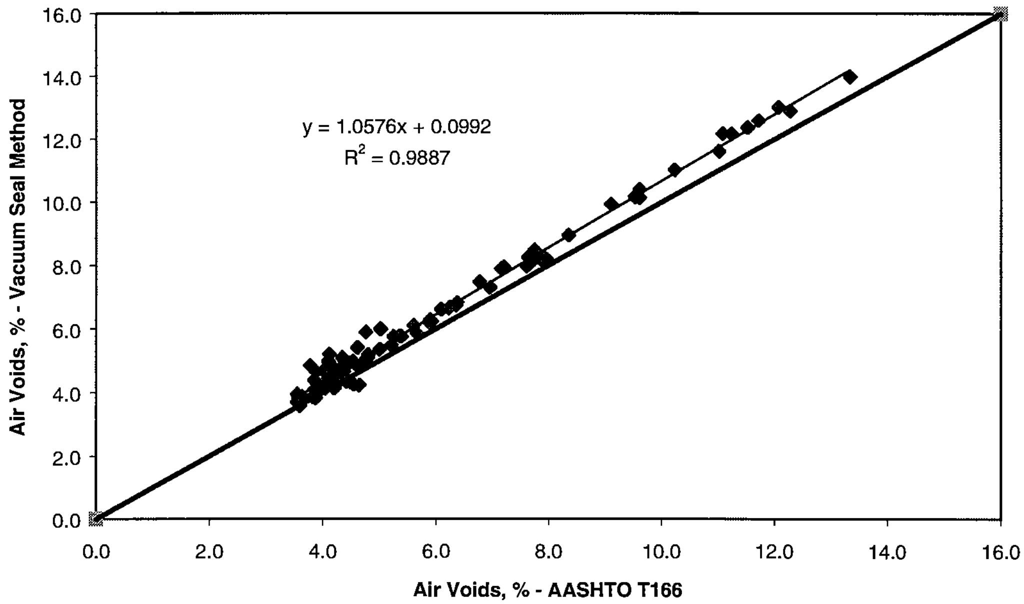

Figure 1. Relationship between air voids for ARZ mixes.

The vibratory compactor used compacted beam samples for the wheel-tracking device.

Of the 36 mix designs analyzed for Part 1, 14 mixes were selected for further study. Two types of aggregates, granite and limestone were used. For Superpave designed mixes, two gradations were utilized (ARZ and BRZ) along with two NMASs (9.5 mm and 19.0 mm). The 37.5-mm NMAS mix was excluded from the study because the maximum thickness of the vibratory specimen that could be obtained was 75.0 mm, which would only be 2.0 t/NMAS. For the SMA mixes, three NMASs were selected (9.5 mm, 12.5 mm, and 19 mm). The t/NMAS ratios utilized were 2.0, 3.0, and 4.0. The compactive effort for each t/NMAS was varied over a range including 30 sec, 60 sec, and 90 sec of compaction. The range of compactive efforts was selected for two reasons: (1) there is no standard compactive effort for the vibratory compactor and

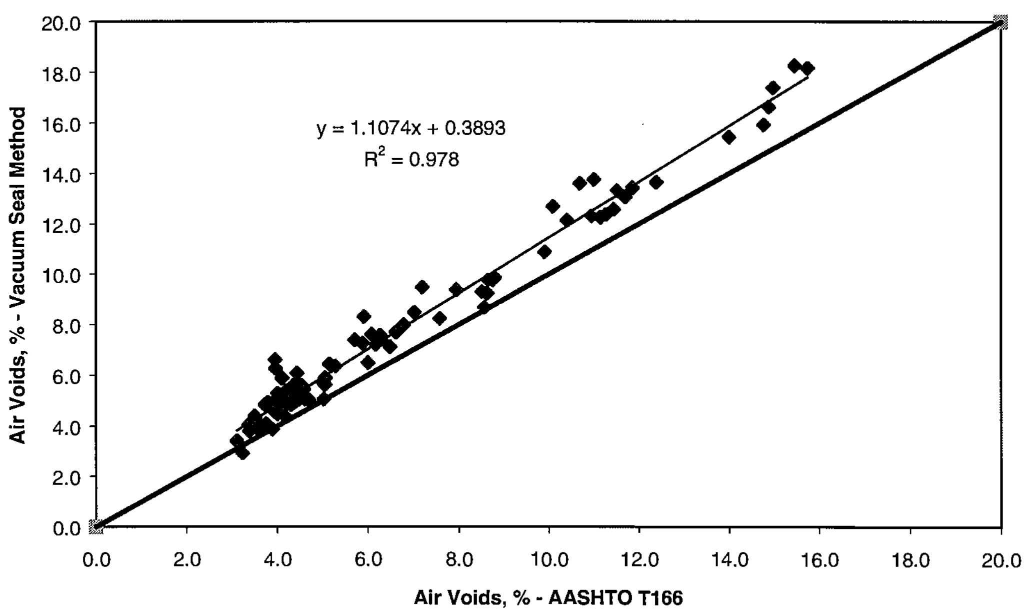

Figure 2. Relationship between air voids for TRZ mixes.

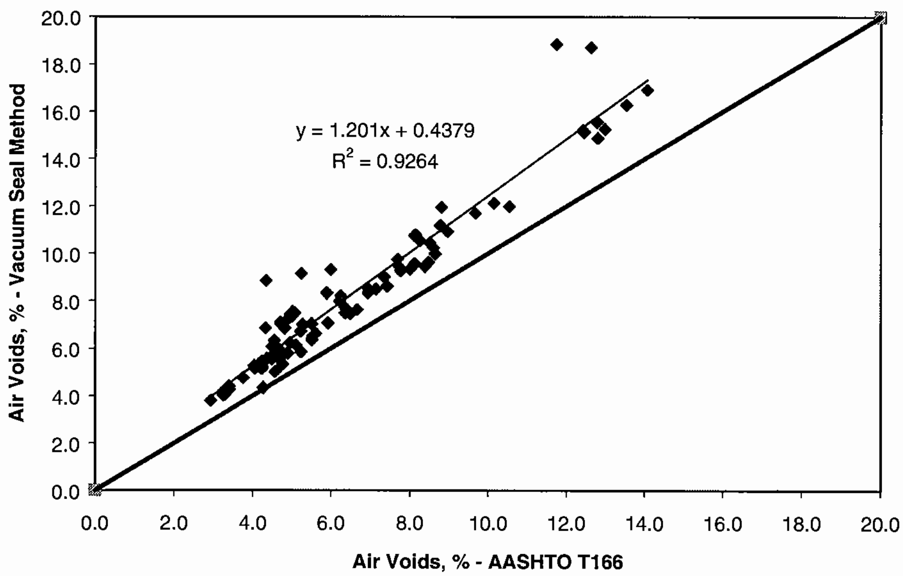

Figure 3. Relationship between air voids for BRZ Mixes.

(2) the effects of compactive effort on density at different thicknesses could be evaluated. After compaction, the bulk specific gravity was measured and the data were analyzed to provide recommendations concerning the minimum t/NMAS.

To determine the minimum t/NMAS, relationships between average air voids for the two types of aggregates and t/NMAS were plotted for each NMAS, compaction time, and gradation, as shown in Figures 11 through 17. In many cases there was very little difference between the densities for the different t/NMAS values. However, in a few cases there was a difference. Also, in many cases the best t/NMAS was 2.0, which is significantly lower than that observed on field projects. Typically, it was assumed that coarse graded mixes would have a desired t/NMAS greater than fine-graded mixes. The results did not always follow that trend. It was judged that some fieldwork was necessary to validate the results with the Superpave gyratory compactor and with the vibratory compactor.

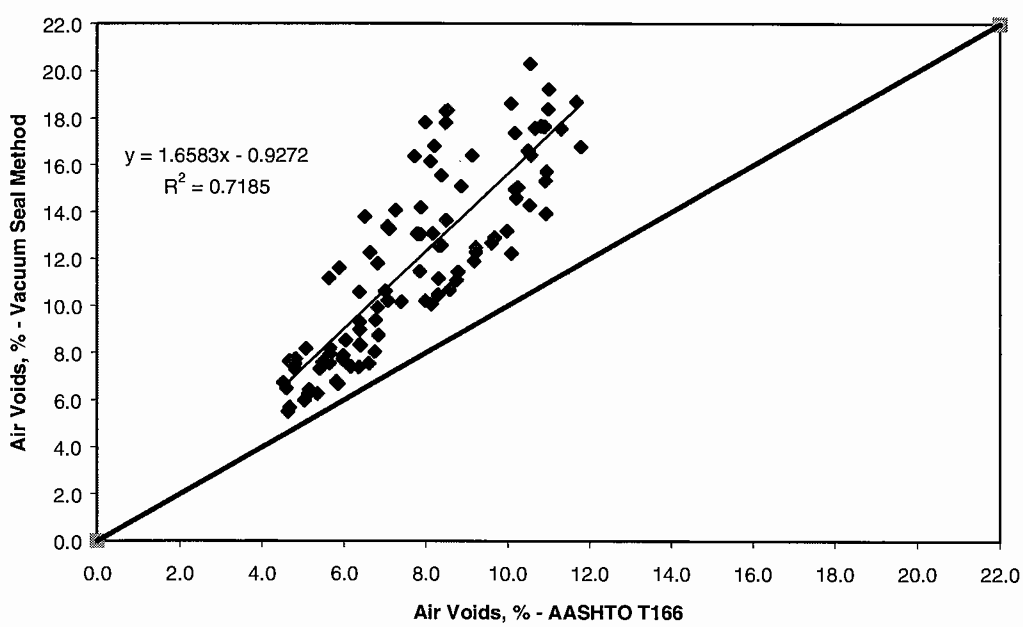

Figure 4. Relationship between air voids for SMA mixes.

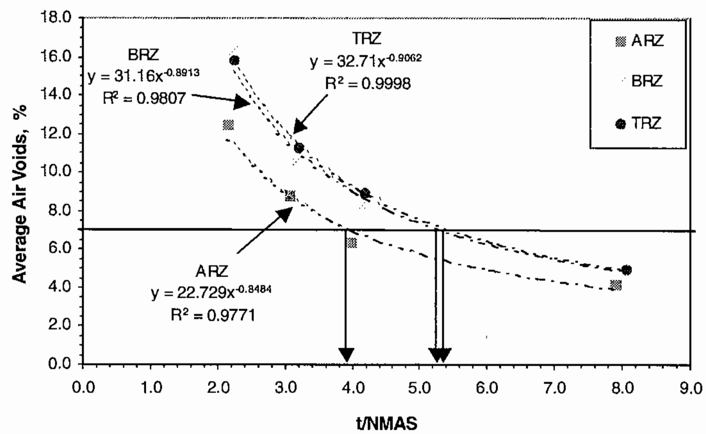

Figure 5. Relationships between air voids and t/NMAS for 9.5-mm Superpave mixes.

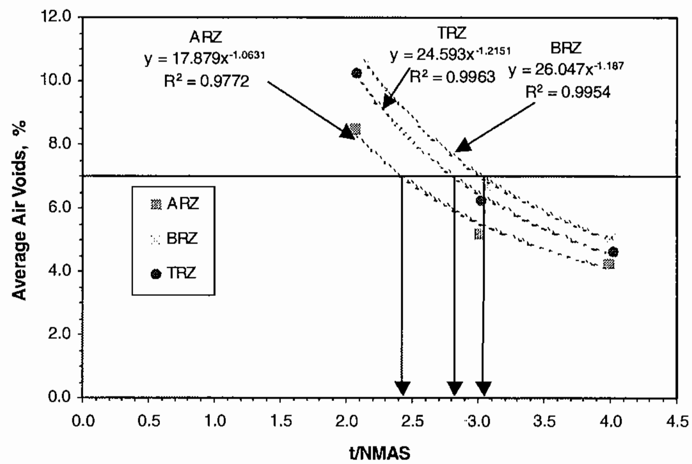

Figure 6. Relationships between air voids and t/NMAS for 19.0-mm Superpave mixes.

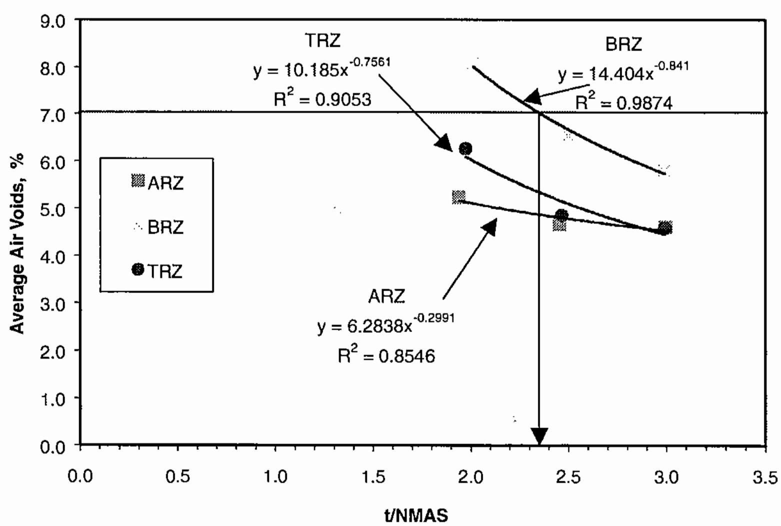

Figure 7. Relationships between air voids and t/NMAS for 37.5-mm Superpave mixes.

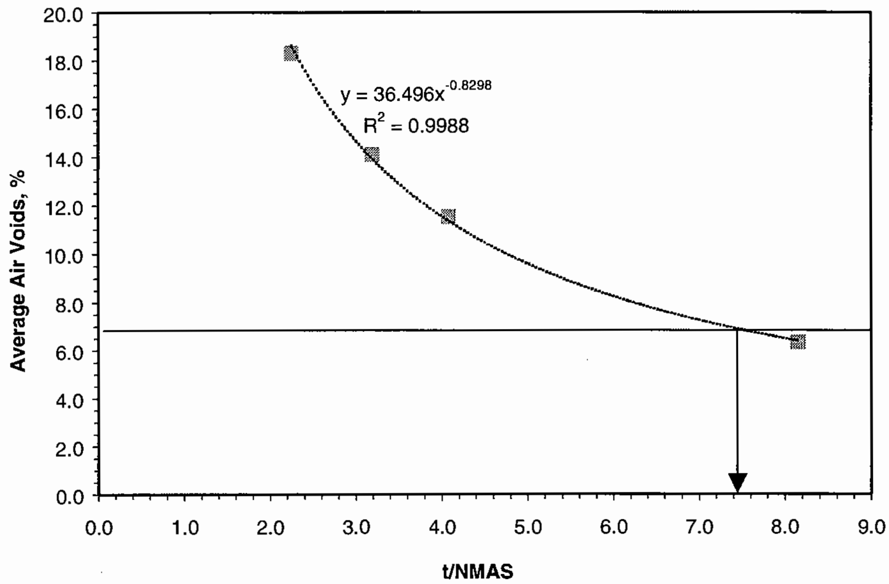

Figure 8. Relationships between air voids and t/NMAS for 9.5-mm SMA mixes.

4.4 EVALUATION OF EFFECT OF t/NMAS ON DENSITY FROM FIELD STUDY

The field test sections consisted of 7 mixes that were to be placed on the test track. These mixes had to be verified before placing on the track; hence, these mixes could be placed and tested without significant costs. Some of the mixes did not meet volumetrics and other requirements, but they were judged sufficient for this part of the study because determining the desired thickness range was a relative value based on t/NMAS.

4.4.1 Section 1

Section 1 was constructed on July 18, 2003, and consisted of a 2.0 to 5.0 t/NMAS overlay of an existing HMA layer. This construction was performed adjacent to the NCAT Test Track. The mix was a 9.5-mm NMAS fine-graded mixture. The length of the section was about 40 m, and the width was about 3.5 m. On some of the sections the placement began on the thick side and in some cases the placement began on the thin side. This technique was used so that there would be no bias due to the placement of the HMA. On this sec-

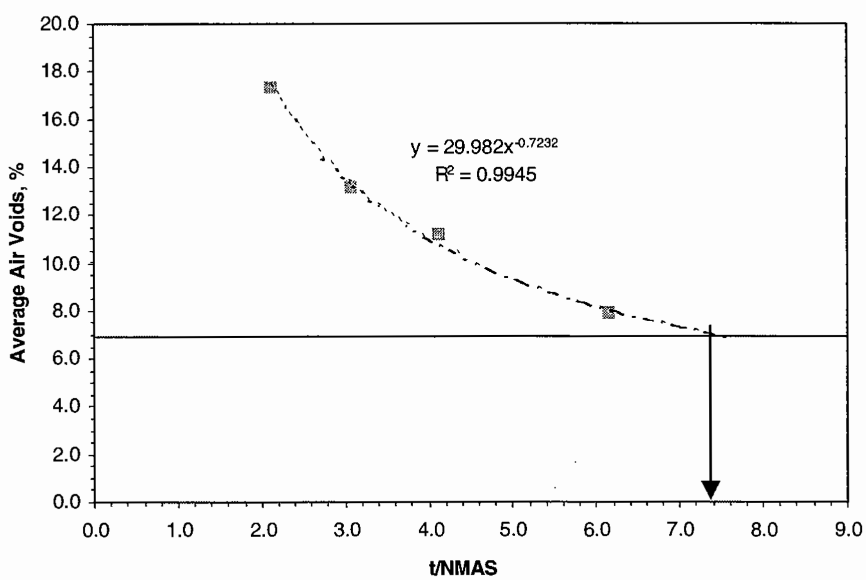

Figure 9. Relationships between air voids and t/NMAS for 12.5-mm SMA mixes.

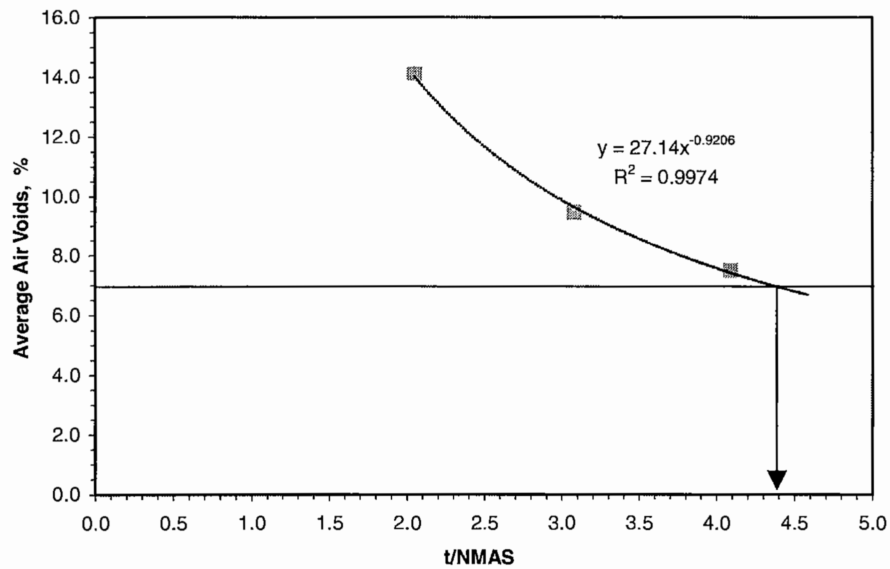

Figure 10. Relationships between air voids and t/NMAS for 19.0-mm SMA mixes.

tion the paving began with the thicker portion of the section and the thickness was slowly decreased as the paver moved down the test lane. The desired mat thickness was achieved by gradually adjusting the screed depth crank of the paver during the paving operation. The weather conditions during the paving were 84°F, overcast, with calm wind. The existing surface temperature prior to overlay was also 84°F.

The roller utilized in this section was an 11-ton steel roller HYPAC C778B with a 78-in. wide drum that could operate in vibratory or static mode. The rubber tire roller available did not meet desired requirements for weight and tire pressure, and thus the data generated for the rubber tire roller compacted mixture were omitted from the analysis for this section. The breakdown rolling was performed with one pass in the static mode on the mat at a temperature of about 300°F. This was followed by three passes in the vibratory mode at low amplitude and high frequency (3800 vibrations per minute [vpm]) and one pass in the static mode. It was determined that this compaction effort reached the peak density; hence, additional rolling was not performed.

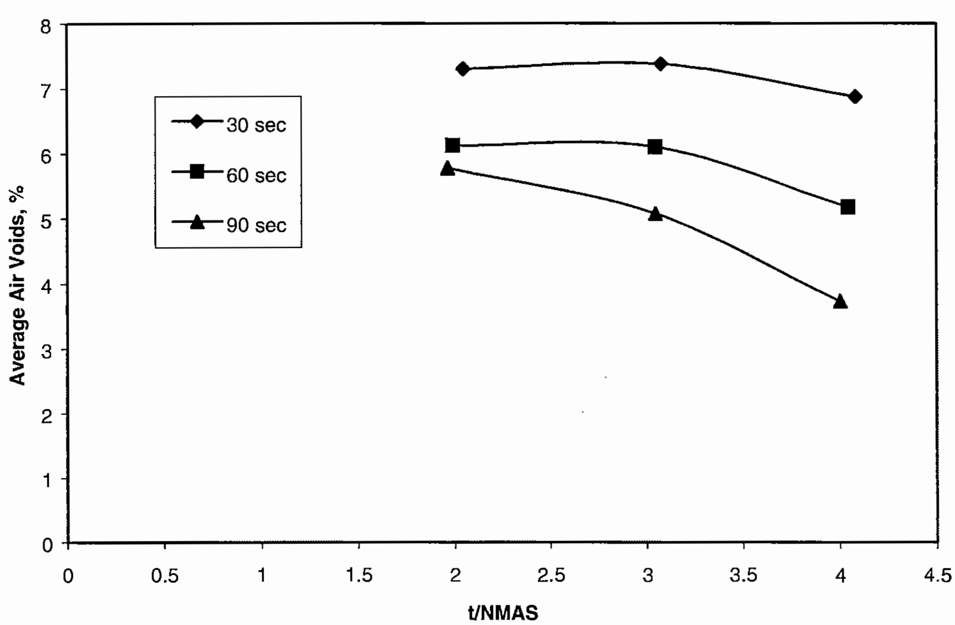

Figure 11. Relationships between air voids and t/NMAS for 9.5-mm ARZ mixes.

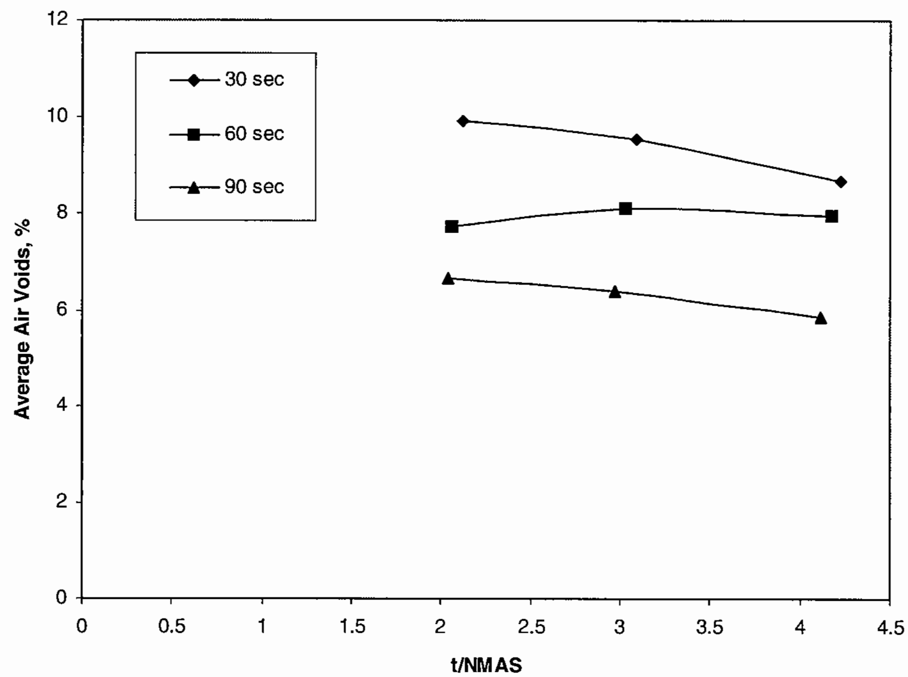

Figure 12. Relationships between air voids and t/NMAS for 9.5-mm BRZ mixes.

A total of 16 cores were obtained from this section and the test results of the cores are presented in Figure 18. The results include the thickness of cores, t/NMAS, and the air voids determined from the vacuum seal device.

A review of the data indicated that a polynomial function provided the best fit line. The best-fit line indicates that the air voids decreased as the t/NMAS increased to a point where additional thickness resulted in increased air voids. The recommended thickness range was selected as the point(s) where the air voids increased by 0.5 percent (less than 0.5 percent were considered insignificant). This number is somewhat arbitrary, but it is realistic. Therefore, as shown in Figure 18, the recommended t/NMAS range for 9.5-mm fine-graded mix was 3.4 to 5.8. This does not mean that satisfactory compaction cannot be obtained outside of these limits, but it does indicate that more compactive effort would be needed. So this recommended range should only be used as a guide and should not be a rigid requirement. The effect of t/NMAS on

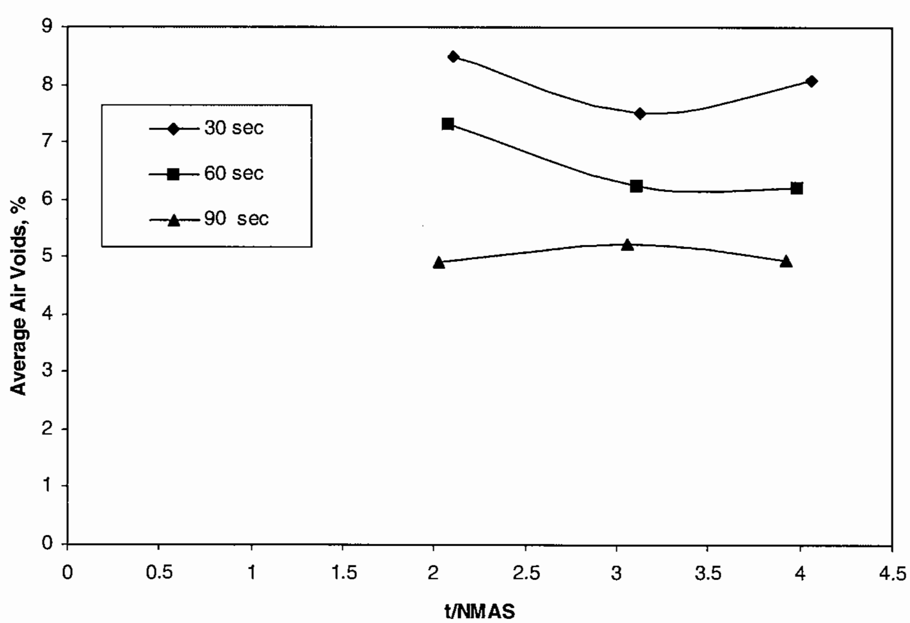

Figure 13. Relationships between air voids and t/NMAS for 19.0-mm ARZ mixes.

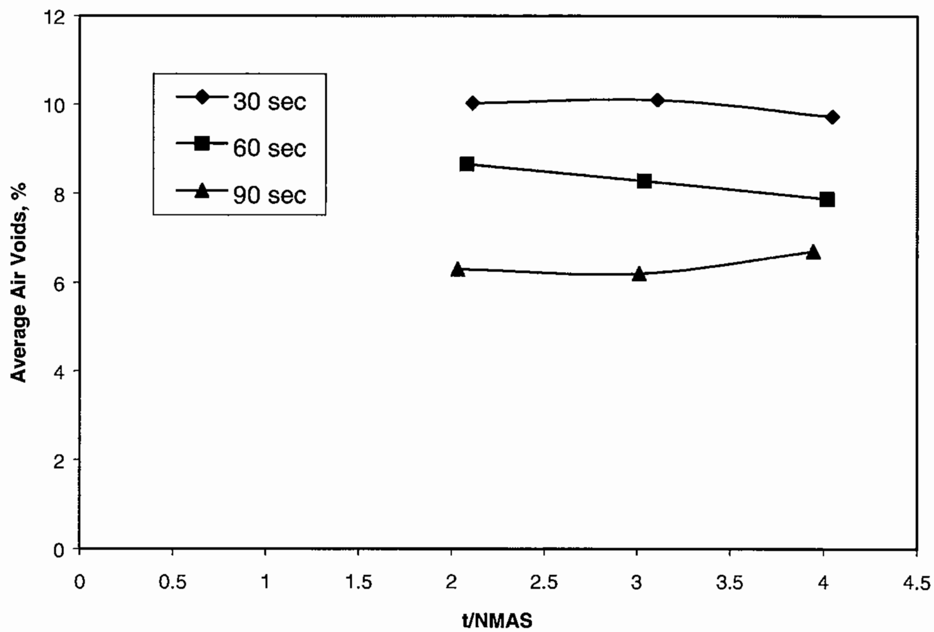

Figure 14. Relationships between air voids and t/NMAS for 19.0-mm BRZ mixes.

the measured density was determined from Figure 18. Data in the figure indicate that the lowest air voids (7.0 percent air voids) occurred at t/NMA 4.4. Table 5 shows the air voids at various t/NMAs as related to this minimum.

4.4.2 Section 2

Section 2 was constructed on August 7, 2003, and the t/NMAS for this overlay ranged from 2.0 to 5.0. The mixture was a 9.5-mm NMAS coarse-graded mixture. The length of the section was about 40 m, and the width was about 3.5 m. The paving started from the thick portion of the mat and progressed toward the thinner portion. The weather conditions during the paving were 82°F, overcast, with calm wind. The existing surface temperature was 96°F.

The roller utilized in this section was an 11-ton steel drum roller HYPAC C778B with a 78-in. wide drum that could operate in vibratory or static mode. The rubber tire roller was a 15-ton HYPAC C560B with a tire pressure of 90 psi. For the side of the mat utilizing only the steel drum roller, the initial rolling was performed with four passes in the vibratory mode at low amplitude and high frequency (3800 vpm) at a mix tem-

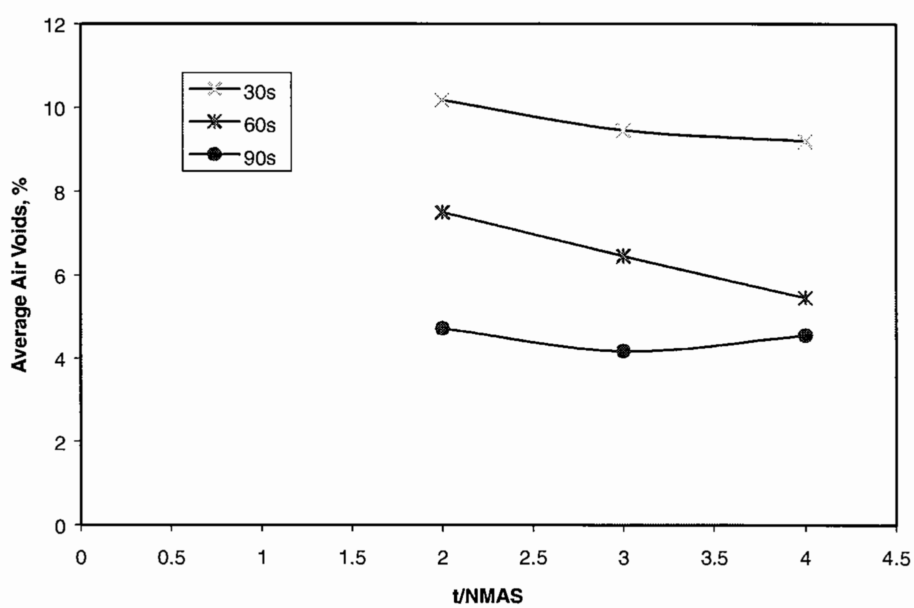

Figure 15. Relationships between air voids and t/NMAS for 9.5-mm SMA mixes.

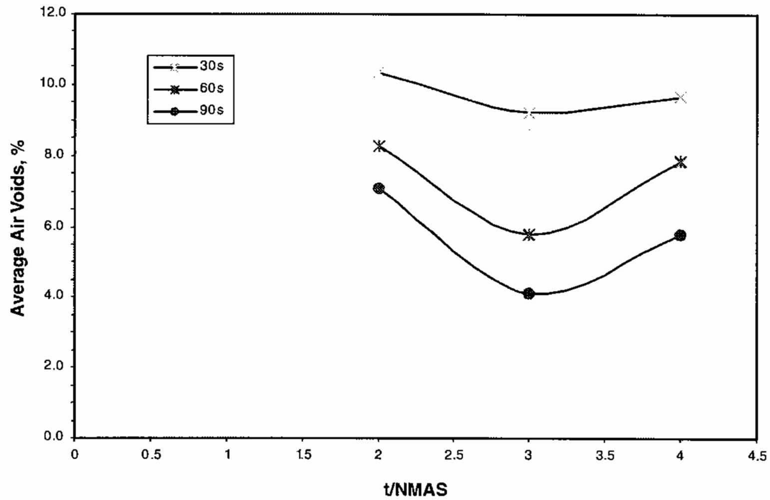

Figure 16. Relationships between air voids and t/NMAS for 12.5-mm SMA mixes.

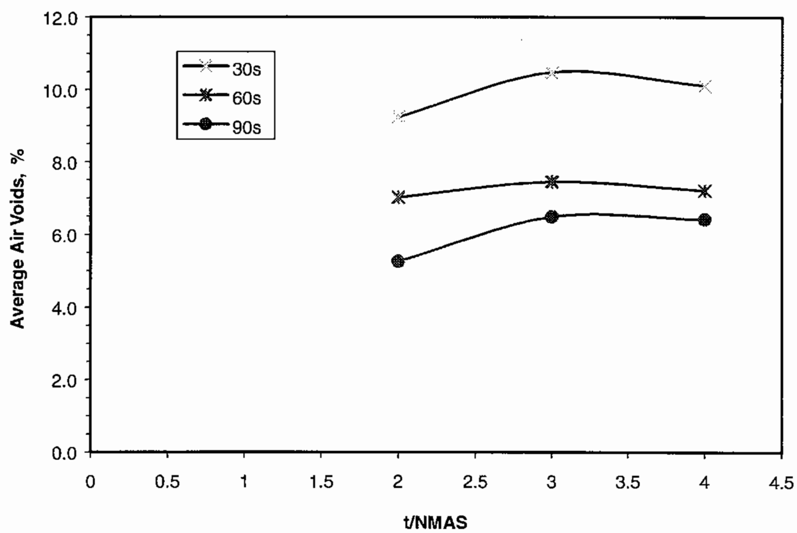

Figure 17. Relationships between air voids and t/NMAS for 19.0-mm SMA mixes.

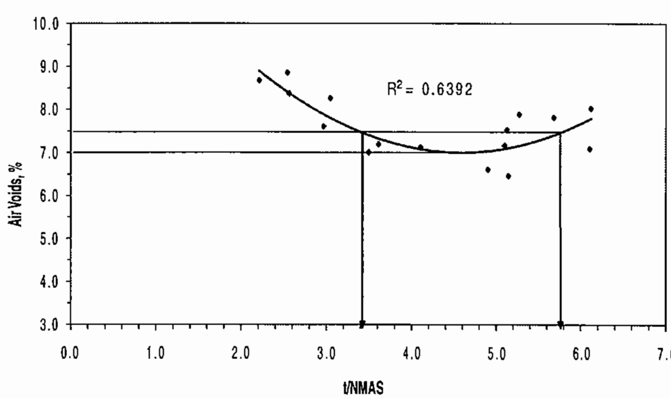

Figure 18. Relationships of air voids and t/NMAS for 9.5-mm fine-graded mix.

TABLE 5 Relationship of air voids and t/NMAS for 9.5-mm fine-graded HMA compacted with steel roller

t/NMA	Percentage points above lowest
4.4 (lowest air voids, 7.0 %)	0.0
2	2.5
3	1.0
4	0.1
5	0.1

perature of about 300°F. This was followed with four passes in the static mode. For the side of the mat that used a rubber tire roller as an intermediate roller, the breakdown rolling was performed with four passes in the vibratory mode operated at low amplitude and high frequency (3800 vpm). This was followed with five passes of the rubber tire roller and one pass of the steel roller in the static mode.

A total of 15 cores were obtained from the side that utilized only a steel drum roller and 16 cores from the side that used the rubber tire roller. The relationship of air voids measured from the vacuum seal device and t/NMAS was evaluated for each rolling pattern. The results are illustrated in Figure 19.

A review of the data indicated that a polynomial function provided the best fit. As the thickness increased, the air voids decreased until a point where additional thickness resulted in increased air voids. The plots also suggest that the side utilizing only a steel drum compactor had better compaction. To determine the desired thickness, it was decided to use air voids 0.5 percent larger (a void level less than 0.5 percent different was not considered significantly different) than the minimum air voids from the best-fit line. Therefore, as shown in Figure 19, the desired t/NMAS range for 9.5-mm coarse-graded mix was 3.5 to 5.9 for compaction with a steel wheel roller and 2.9 to 4.6 for compaction with the steel and rubber tire roller. The effect of t/NMAS on the measured density was determined from Figure 19. Data in the figure indicate that the lowest in-place air voids (10 percent air voids for the steel wheel roller only and 10.5 percent air voids for the steel and rubber tire rollers) occurred at t/NMAS of 4.7 for the steel wheel roller and 3.8 for the rubber and steel wheel roller. Table 6 shows the air voids at various t/NMAs as related to this minimum.

4.4.3 Section 3

Section 3 was constructed on July 25, 2003, and consisted of a 2.0 to 5.0 t/NMAS overlay of an existing HMA layer. The mix was a 9.5-mm NMAS SMA. The length of the section was about 40 m, and the width was about 3.5 m. The paving started from the thick portion of the mat and progressed to the thinner portion. The desired mat thickness was achieved by gradually adjusting the screed depth crank of the paver during the operation. The weather conditions during the paving were 95°F, partly cloudy, with calm wind. The existing surface temperature was 115°F.

The roller utilized in this section was an 11-ton steel drum roller HYPAC C778B with a 78-in. wide drum that could operate in vibratory or static mode. The rubber tire roller was a 15-ton HYPAC C560B with a tire pressure of 90 psi. For the side of the mat utilizing only the steel drum roller, the initial rolling was performed with one pass in the static mode followed by five passes in the vibratory mode operated in low amplitude and high frequency (3800 vpm) on the mat having a mix temperature of about 320°F. This was followed with two passes in the static mode for the finish rolling. For the side of the mat that used a rubber tire roller as an intermediate roller, the breakdown rolling was performed with one pass in the static mode and four passes in the vibratory mode operated in low amplitude and high frequency (3800 vpm). This

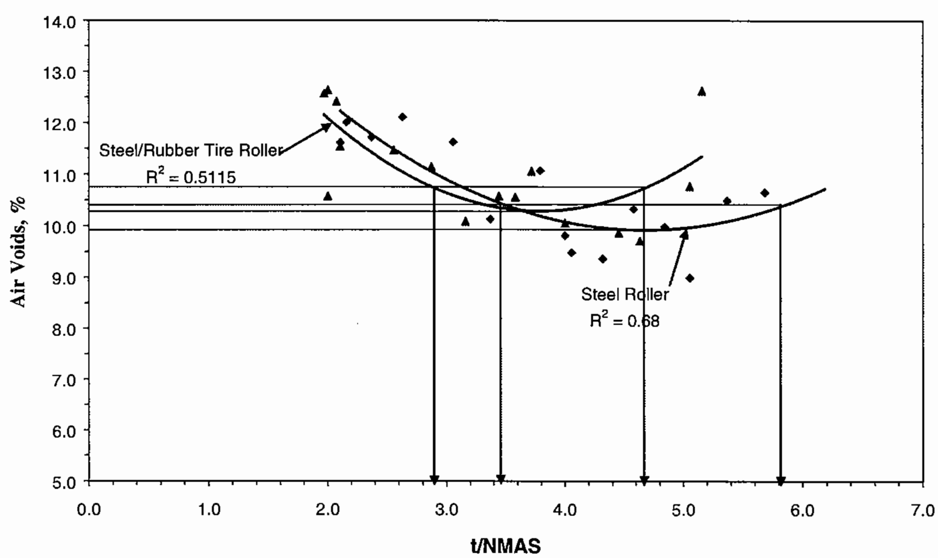

Figure 19. Relationships of air voids and t/NMAS for 9.5-mm coarse-graded mix.

TABLE 6 Relationship of air voids and t/NMAS for 9.5-mm coarse-graded HMA compacted with steel roller and with steel and rubber tire rollers

Steel roller		Steel and rubber tire rollers	
t/NMA	Percentage points above lowest	t/NMA	Percentage points above lowest
4.7 (lowest air voids, 10.0 %)	0.0	3.8 (lowest air voids, 10.5 %)	0.0
2	2.5	2	2.0
3	1.0	3	0.5
4	0.5	4	0.0
5	0.0	5	1.0

was followed with eight passes of the rubber tire roller and two passes of the steel wheel roller in the static mode.

A total of 12 cores were obtained from the side that utilized only the steel drum roller and another 12 cores from the side that used the rubber tire roller. To determine the range of recommended t/NMAS for this mix, the relationship of air voids from the vacuum seal device and t/NMAS was evaluated for each rolling pattern. The results are illustrated in Figure 20.

The best-fit lines indicate that the air voids decreased as the thickness increased to a point where additional thickness resulted in increased air voids. The plots also suggest that the side utilizing only the steel drum compactor had higher density. Rubber tire rollers are not used on SMA mixtures and these data confirm that there is no need to use the rubber tire roller. As shown in Figure 20, the recommended range for t/NMAS for the 9.5-mm SMA mix is 3.8 to 5.3 for the compaction with a steel wheel roller and 2.6 to 5.1 for compaction with a steel and rubber tire roller. The effect of t/NMAS on the measured density was determined from Figure 20. Data in the figure indicate that the lowest in-place air voids (8.5 percent air voids for the steel wheel roller only and 10.3 percent air voids for the steel and rubber tire rollers) occurred at t/NMAS of 4.5 for the steel wheel roller and 3.8 for the rubber and steel wheel roller. Table 7 shows the air voids at various t/NMAs as related to this minimum.

4.4.4 Section 4

Section 4 was constructed on August 12, 2003, and consisted of a 2.0 to 5.0 t/NMAS overlay of an existing HMA layer. The mix was a 12.5-mm NMAS SMA. The length of the section was about 40 m, and the width was about 3.5 m. The paving started from the thinner portion and proceeded toward the thicker portion of the mat. The weather conditions during the paving were 80°F, overcast, with calm wind. The existing surface temperature was 85°F.

The roller utilized in this section was an 11-ton steel drum roller HYPAC C778B with a 78-in. wide drum that could operate in vibratory and static modes. The rubber tire roller was a 15-ton HYPAC C560B with a tire pressure of 90 psi. For the side of the mat utilizing only the steel drum roller, the initial rolling was performed with four passes in the vibratory mode operated at low amplitude and high frequency (3800 vpm). The mat temperature was approximately 320°F. This was followed with three passes in the static mode including finish rolling. For the side of the mat that used a rubber tire roller as an intermediate roller, the initial rolling was performed with four passes in the vibratory mode operated at low amplitude and high frequency (3800 vpm). This was followed with four passes of the rubber tire roller and one pass of the steel roller in the static mode.

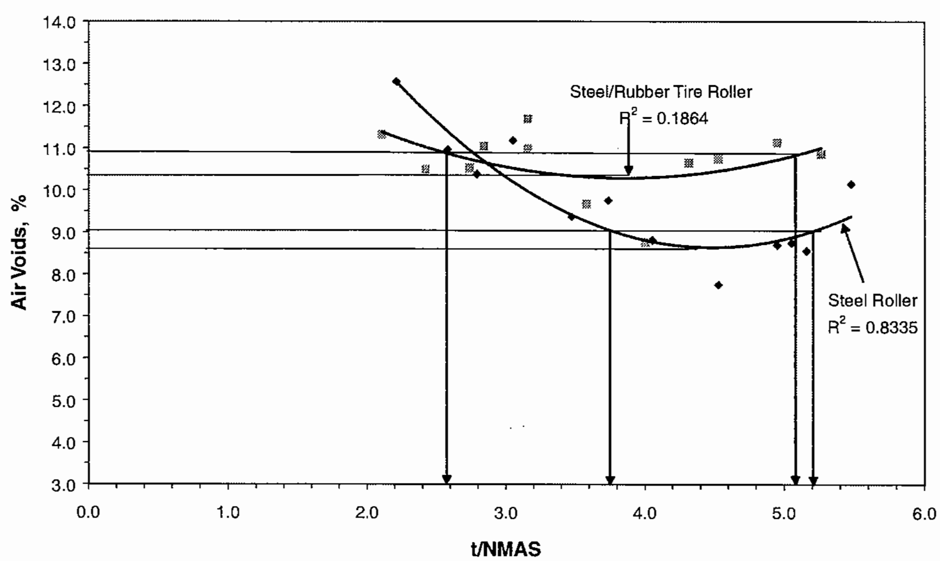

Figure 20. Relationships of air voids and t/NMAS for 9.5-mm SMA mix.

TABLE 7 Relationship of air voids and t/NMAS for 9.5-mm SMA mix compacted with steel roller and with steel and rubber tire rollers

Steel roller		Steel and rubber tire rollers	
t/NMA	Percentage points above lowest	t/NMA	Percentage points above lowest
4.5 (lowest air voids, 8.5 %)	0.0	3.8 (lowest air voids, 10.3 %)	0.0
2	5.5	2	1.2
3	2.0	3	0.2
4	0.2	4	0.0
5	0.2	5	0.5

A total of 21 cores were obtained from the side that utilized only a steel drum roller and 21 cores from the side that used the rubber tire roller. To determine the recommended t/NMASs for this mix, the relationship of air voids from the vacuum seal device and t/NMAS was evaluated for each rolling pattern. The results are illustrated in Figure 21.

The best-fit lines indicate that the air voids decreased as the thickness increased to a point where additional thickness resulted in increased air voids. The plots also suggest that the side utilizing only the steel drum compactor had higher density. As shown in Figure 21, the suggested minimum t/NMAS for 12.5-mm SMA mix is 3.8 for compaction with steel wheel roller and 4.6 for compaction with steel and rubber tire rollers. For these mixes, the density increased as the t/NMAS increased even at the thicker portions. Also the curve did not fit the data as well as desired, so the data points were actually used to select the suggested t/NMAS number. Note in the plots that the data points continue downward with increasing t/NMAS to a point and then the air voids remain relatively constant as the t/NMAS increased.

The effect of t/NMAS on the measured density was determined from Figure 21. Data in the figure indicate that the lowest in-place air voids (4.7 percent air voids for the steel wheel roller only and 7.5 percent air voids for the steel and rubber tire rollers) occurred at t/NMAS of 4.5 for the steel wheel roller and 4.8 for the rubber and steel wheel rollers. Table 8 shows the air voids at various t/NMAs as related to this minimum.

4.4.5 Section 5

Section 5 was constructed on July 16, 2003, and consisted of a 2.0 to 5.0 t/NMAS overlay of an existing HMA. The mix consisted of a 19.0-mm NMAS fine-graded HMA. The length of the section was about 40 m, and the width was about 3.5 m. The paving started on the thin end of the section and proceeded to the thicker portion. The desired mat thickness was achieved by gradually adjusting the screed depth crank of the paver during the operation. The weather conditions during the paving were 90°F, clear, with calm wind. The existing surface temperature was 96°F.

The roller utilized in this section was an 11-ton steel roller HYPAC C778B with a 78-in. wide drum that operated in vibratory and static modes. The rubber tire roller used did not meet the tire pressure requirements and the results were omitted from the analysis for this section. The breakdown rolling was performed with four passes in the vibratory mode operated in low amplitude and high frequency (3800 vpm). The

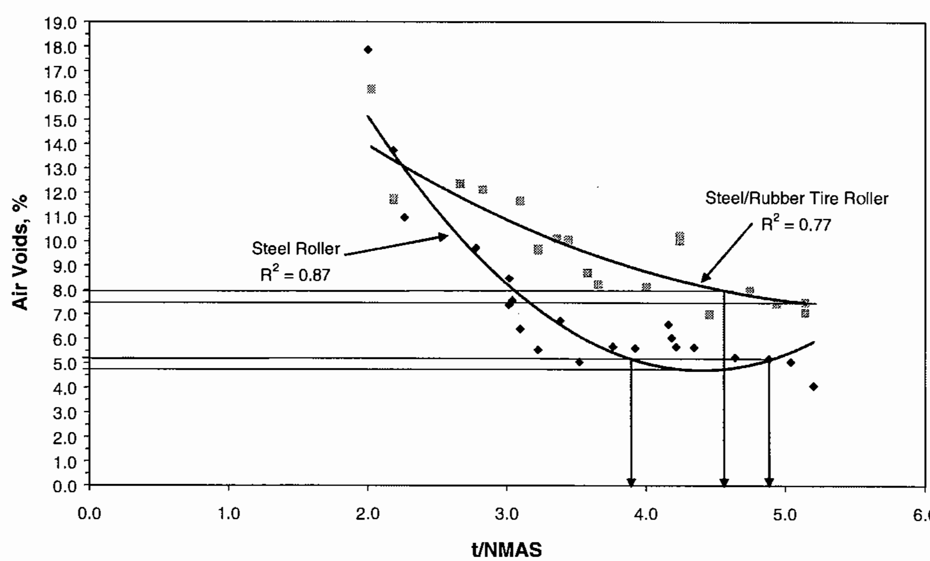

Figure 21. Relationships of air voids and t/NMAS for 12.5-mm SMA Mix.

TABLE 8 Relationship of air voids and t/NMAS for 12.5-mm SMA mix compacted with steel roller and with steel and rubber tire rollers

Steel roller		Steel and rubber tire rollers	
t/NMA	Percentage points above lowest	t/NMA	Percentage points above lowest
4.5 (lowest air voids, 4.7 %)	0.0	4.8 (lowest air voids, 7.5 %)	0.0
2	11.3	2	6.5
3	3.3	3	3.5
4	0.3	4	0.5
5	0.5	5	0.0

mat temperature was approximately 300°F. Three passes in the static mode and one pass for finish rolling followed this initial rolling.

A total of 20 cores were obtained from this section. To determine the minimum t/NMAS for this mix, the relationship between air voids (from the vacuum seal device) and thickness was evaluated. The results are illustrated in Figure 22.

The best-fit line indicated that the air voids decreased as the thickness increased to a point where additional thickness resulted in increased air voids. As shown in Figure 22, the recommended t/NMAS range for the 19.0-mm fine-graded mix was 3.1 to 4.6. The effect of t/NMAS on the measured density was determined from the figure. Data in the figure indicate that the lowest in-place air voids (6.2 percent air voids) occurred at t/NMAS of 3.8. Table 9 shows the air voids at various t/NMAs as related to this minimum.

4.4.6 Section 6

Section 6 was constructed on August 6, 2003, and consisted of a range of 2.0 to 5.0 t/NMAS overlay of an existing HMA. The mix was a 19.0-mm NMAS coarse-graded HMA. The length of the section was about 40 m, and the width was about 3.5 m. The paving started from the thinner portion of the mat and proceeded to the thicker portion. The weather conditions during the paving were 79°F, cloudy, with calm wind. The existing surface temperature was 84°F.

The roller utilized in this section was an 11-ton steel drum roller HYPAC C778B with a 78-in. wide drum that could operate in vibratory and static mode. The rubber tire roller was a 15-ton HYPAC C560B with a tire pressure of 90 psi. For the side of the mat utilizing only the steel drum roller, the initial rolling was performed with four passes in the vibratory mode operated at low amplitude and high frequency (3800 vpm). The mat temperature was approximately 300°F. This initial rolling was followed with six passes in the static mode. For the side of the mat that used a rubber tire roller as the intermediate roller, the initial rolling was performed with four passes in the vibratory mode operated in low amplitude and high frequency (3800 vpm). This initial rolling was followed with four passes of the rubber tire roller and two passes with a steel wheel roller in the static mode.

A total of 22 cores were obtained from the side that utilized only a steel drum roller and 16 cores from the side that used the rubber tire roller. To determine the minimum t/NMAS for this mix, the relationship between air voids from vacuum seal

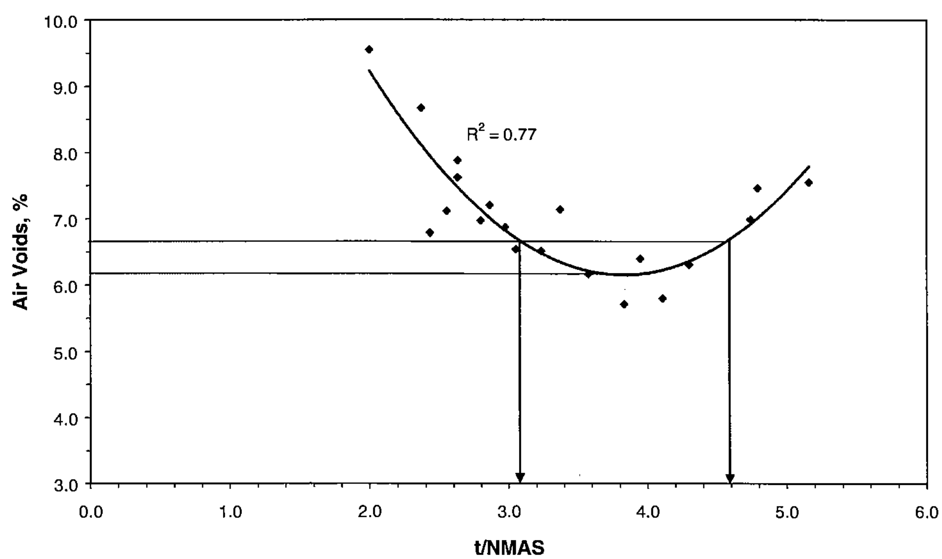

Figure 22. Relationships of air voids and t/NMAS for 19.0-mm fine-graded mix.

TABLE 9 Relationship of air voids and t/NMAS for 19.0-mm fine-graded mix compacted with steel roller

t/NMA	Percentage points above lowest
3.8 (lowest air voids, 6.2 %)	0.0
2	3.1
3	0.6
4	0.0
5	1.3

Table 10 Relationship of air voids and t/NMAS for 19.0-mm coarse-graded mix compacted with steel and rubber tire roller*

t/NMA	Percentage points above lowest
4.5 (lowest air voids, 5.7 %)	0.0
2	1.8
3	0.6
4	0.1
5	0.1

*The steel wheel roller alone was not used because it produced too much scatter in the data

device and thickness was evaluated for each rolling pattern. The results are illustrated in Figure 23. The best-fit lines indicate that the air voids decreased as the thickness increased to a point where additional thickness resulted in increased air voids. The plots also suggest that the side utilizing the rubber tire roller had higher density. As shown in Figure 23, the recommended minimum thickness for 19.0-mm coarse-graded mix was 3.0 for compaction with the steel and rubber tire rollers. There is too much scatter in the data to make a good selection of a recommended value for compaction with a steel wheel roller.

The effect of t/NMAS on the measured density was determined from Figure 23. Data in the figure indicate that the lowest in-place air voids (5.7 percent for the steel and rubber tire roller, the steel wheel roller alone was not used because it produced too much scatter in the data) occurred at t/NMAS of 4.5. Table 10 shows the air voids at various t/NMAs as related to this minimum.

4.4.7 Section 7

Section 7 was constructed on August 14, 2003, and consisted of a range of 2.0 to 5.0 t/NMAS overlay of an existing HMA. The mix consisted of a 19.0-mm NMAS coarse-graded HMA and utilized a modified asphalt. The length of the section was about 40 m, and the width was about 3.5 m. The paving started from the thicker portion of the mat and proceeded to the thinner portion. The weather conditions during the paving were 90°F, clear, with calm wind. The existing surface temperature was 120°F.

The roller utilized in this section was an 11-ton steel drum roller HYPAC C778B with a 78-in. wide drum that could operate in the vibratory and static modes. The rubber tire roller was a 15-ton HYPAC C560B with a tire pressure of 90 psi. For the side of the mat utilizing only the steel drum roller, the initial rolling was performed with four passes in the vibratory mode operated in low amplitude and high frequency (3800 vpm). The mat temperature was about 330°F. This was followed with another five passes in the vibratory mode operated at low amplitude and high frequency (3800 vpm). There was one additional pass with the steel wheel roller in the static mode to finish the mat. For the side of the mat that used a rubber tire roller as an intermediate roller, the initial rolling was performed with two passes in the vibratory mode operated at low amplitude and high frequency (3800 vpm). This was followed with ten passes with

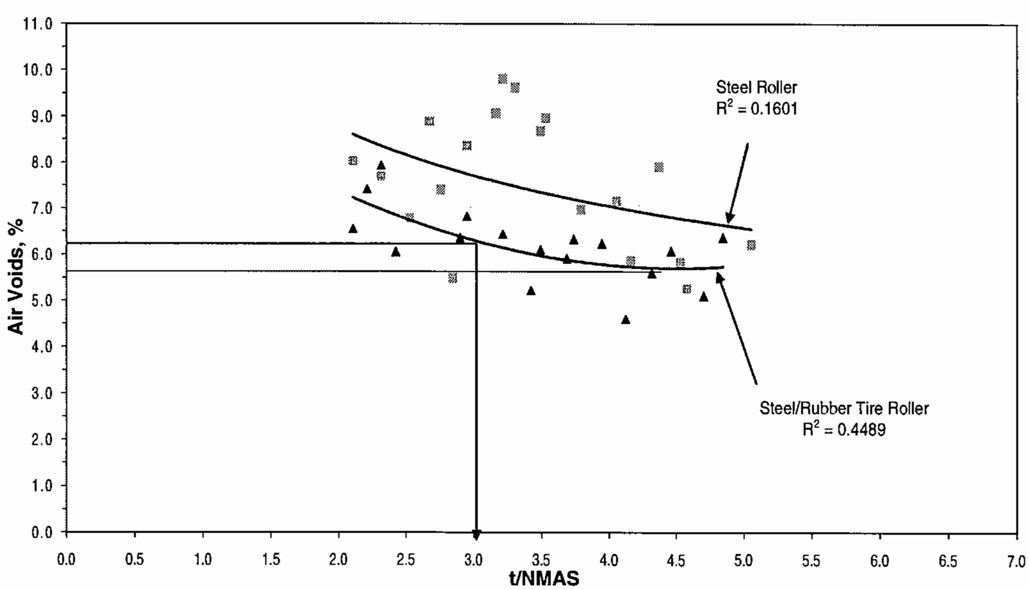

Figure 23. Relationships of air voids and t/NMAS for 19.0-mm coarse-graded mix.

the rubber tire roller and two passes of the steel wheel roller in the static mode.

A total of 23 cores were obtained from the side that utilized only the steel drum roller and 26 cores from the side that used the rubber tire roller. To determine the minimum t/NMAS for this mix, the relationship of air voids from the vacuum seal device and t/NMAS was evaluated for each rolling pattern. The results are illustrated in Figure 24.

The best-fit lines indicate that the air voids decreased as the thickness increased to a point where additional thickness resulted in increased air voids. The plots also suggested that the side utilizing only the steel drum compactor had higher density. As shown in Figure 24, the minimum t/NMAS range for 19.0-mm coarse-graded with modified asphalt mix was 3.4 to 4.8. The effect of t/NMAS on the measured density was determined from Figure 24. Data in the figure indicate that the lowest in-place air voids (5.6 percent air voids for the steel wheel roller only and 7.4 percent air voids for the steel and rubber tire rollers) occurred at t/NMAS of 4.2 for the steel wheel roller and 5.3 for the rubber and steel wheel roller. Table 11 shows the air voids at various t/NMAs as related to this minimum.

4.4.8 Summary

In summary, the data for the seven sections appear to be reasonable and to match past experience. A summary of the results compared to the t/NMAS for lowest voids is provided in Table 12. These results indicate that the t/NMAS should be somewhere between 3 and 5 for best results. Based on the limited data, a t/NMAS of 3 is probably reasonable for fine-graded mixes, because there is less than 1 percentage point change in density when the t/NMAS is reduced from optimum to 3.0. The t/NMAS should be set at 4.0 for coarse-graded mixes due to the significant increase in voids when reducing the t/NMAS from optimum down to 3.0.

4.5 EVALUATION OF THE EFFECT OF TEMPERATURE ON THE RELATIONSHIP BETWEEN DENSITY AND t/NMAS

Three locations were selected for temperature measurements for each section in the field experiment; one near the beginning of the section, one near the middle, and one near the end of the section. To determine the effect of mix temperature on the density, the temperature at 20 minutes after placement of the mix at each location was selected because this provides a reasonable compaction time. Because the mixes in this study used two different types of asphalt binder, PG 67-22 and PG 76-22, the temperatures at 20 minutes were normalized by subtracting the high temperature grade of the asphalt type from the temperatures at 20 minutes. Table 13 presents the t/NMAS, the average temperature readings at 20 minutes, the asphalt high temperature grade, and the difference between mix temperature and high temperature grade. The differences in temperature were plotted against the t/NMAS together with the core densities for each section, as shown in Figures 25 through 31.

The relationship between density and t/NMAS for all sections is shown in Figure 32. The best-fit line has an R^2 of 0.26 and indicates that the density increased as the thickness increased to a point where additional thickness resulted in a decrease in density. The effect of the layer thickness and cooling time on mix temperature is provided in Figure 33. The data were obtained from the thermocouples installed in the pavement. This plot indicates that, during hot weather,

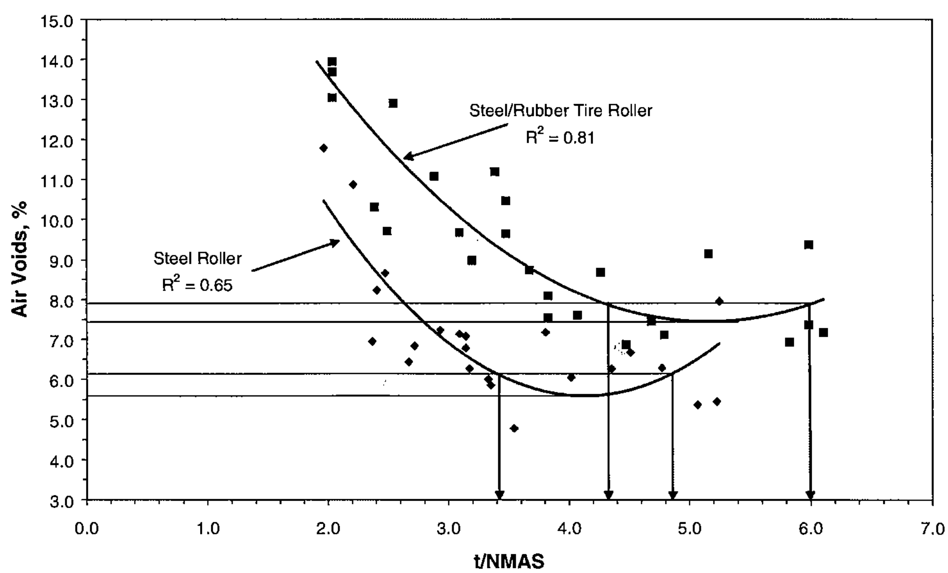

Figure 24. Relationships of air voids and t/NMAS for 19.0-mm coarse-graded mix with modified asphalt.

TABLE 11 Relationship of air voids and t/NMAS for 19.0-mm coarse-graded mix with modified asphalt compacted with steel roller and with steel and rubber tire rollers

Steel roller		Steel and rubber tire rollers	
t/NMA	Percentage points above lowest	t/NMA	Percentage points above lowest
4.2 (lowest air voids, 5.6 %)	0.0	5.3 (lowest air voids, 7.4 %)	0.0
2	4.9	2	6.1
3	1.3	3	3.4
4	0.0	4	0.8
5	0.8	5	0.0

compaction time for a layer thickness of 1.5 in. is approximately twice that for a 1-in. layer. This clearly shows that one of the problems in obtaining density is layer thickness regardless of the t/NMAS. If the amount of compaction time is reduced by 50 percent, it may be very difficult to compact the mixture to an adequate density. To place the same amount of compactive effort on an HMA mixture prior to cooling to some defined temperature will take twice as many rollers at a 1-in. thickness as that required for a 1.5-in. surface. It is likely to be significantly more difficult to compact a 1-in. layer than to compact a 1.5-in. layer simply because of the cooling rate.

TABLE 12 Effect of t/NMAS on compactibility of HMA

Description of Mix	Increase in Air Voids for t/NMAS=2	Increase in Air Voids for t/NMAS=3	Increase in Air Voids for t/NMAS=4	Increase in Air Voids for t/NMAS=5
Section 1-9.5mm Fine Graded—Steel Roller	2.5%	1.0%	0.1%	0.1%
Section 2-9.5mm Coarse Graded-Steel Roller	2.5%	1.0%	0.5%	0.0%
Section 2-9.5mm Coarse Graded-Steel and Rubber Roller	2.0%	0.5%	0.0%	1.0%
Section 3-9.5mm SMA(mod AC) Steel Roller	5.5%	2.0%	0.2%	0.2%
Section 3-9.5mm SMA(Mod AC) Steel & Rubber Roller	1.2%	0.2%	0.0%	0.5%
Section 4-12.5mm SMA (mod AC) Steel Roller	11.3%	3.3%	0.3%	0.5%
Section 4-12.5mm SMA (mod AC) Steel & Rubber Roller	6.5%	3.5%	0.5%	0.0%
Section 5-19mm Fine Graded Steel Roller	3.1%	0.6%	0.0%	1.3%
Section 6-19mm Coarse Graded Steel and Rubber Roller	1.8%	0.6%	0.1%	0.1%
Section 7-19mm Coarse Graded (mod AC) Steel Roller	4.9%	1.3%	0.0%	0.8%
Section 7-19mm Coarse Graded (mod AC) Steel & Rubber Roller	6.1%	3.4%	0.8%	0.0%

TABLE 13 t/NMAS, temperature in °C at 20 min., asphalt high temperature grade, and difference in temperature

Section/Mix		Temp. at 20 min., °C	Asphalt Grade, PG	Difference
1	2.5	60	67	-7
9.5mmFG	3.6	82	67	15
	5.1	95	67	28
2	2.1	64	67	-3
9.5mmCG	2.4	72	67	5
	5.1	105	67	38
3	2.2	65	76	-11
9.5mmSMA	3.7	100	76	24
	5.2	112	76	36
4	2.2	72	76	-4
12.5mmSMA	3.1	118	76	42
	3.8	120	76	44
5	2.6	124	67	57
19mmFG	3.0	122	67	55
	5.2	130	67	63
6	2.1	82	67	15
19mmCG	3.2	120	67	53
	5.1	118	67	51
7	2.7	86	76	10
19mmCG	3.8	120	76	44
	5.2	142	76	66

4.6 EVALUATION OF EFFECT OF t/NMAS ON PERMEABILITY USING GYRATORY COMPACTOR

Specimens were compacted to 7.0 ± 1.0 percent air void content at t/NMAS of 2.0, 3.0, and 4.0. For most mixes, specimens could not achieve the target air voids even when the gyrations were increased up to 300 gyrations. This shows the difficulty of compacting mixes at thinner lifts in the gyratory mold. Permeability testing was only performed on specimens that met the desired air voids. The results were very limited, but, did show that generally the coarser mixes (larger maximum aggregate size or higher percentage of coarse aggregate) had higher permeabilities.

4.7 EVALUATION OF EFFECT OF t/NMAS ON PERMEABILITY USING VIBRATORY COMPACTOR

All specimens compacted at t/NMAS of 2.0, 3.0, and 4.0 did achieve the target air void content, which was 7 ± 1.0 percent. Figure 34 shows the relationship between average permeability for the two aggregate types and t/NMAS. In general, the permeability decreased as t/NMAS increased. Most of the mixes had permeability values fewer than 50 × 10^{-5} cm/sec. However, at t/NMAS equal to 2.0, the 9.5-mm and 12.5-mm NMAS SMA mixes had average permeability values of 173 × 10^{-5} cm/sec and 196 × 10^{-5} cm/sec, respectively. These values for the SMA exceed the recommended maximum permeability value of 125 × 10^{-5} cm/sec. It appears from these data that a specification requirement of 7 percent air voids would be acceptable for all of the mixes if the t/NMAS is 3 or greater. The likely reason that the thinner samples have high permeability is that the voids are more likely to be interconnected all the way through the samples when the samples are thinner. Hence when mixes are placed thin, in this case

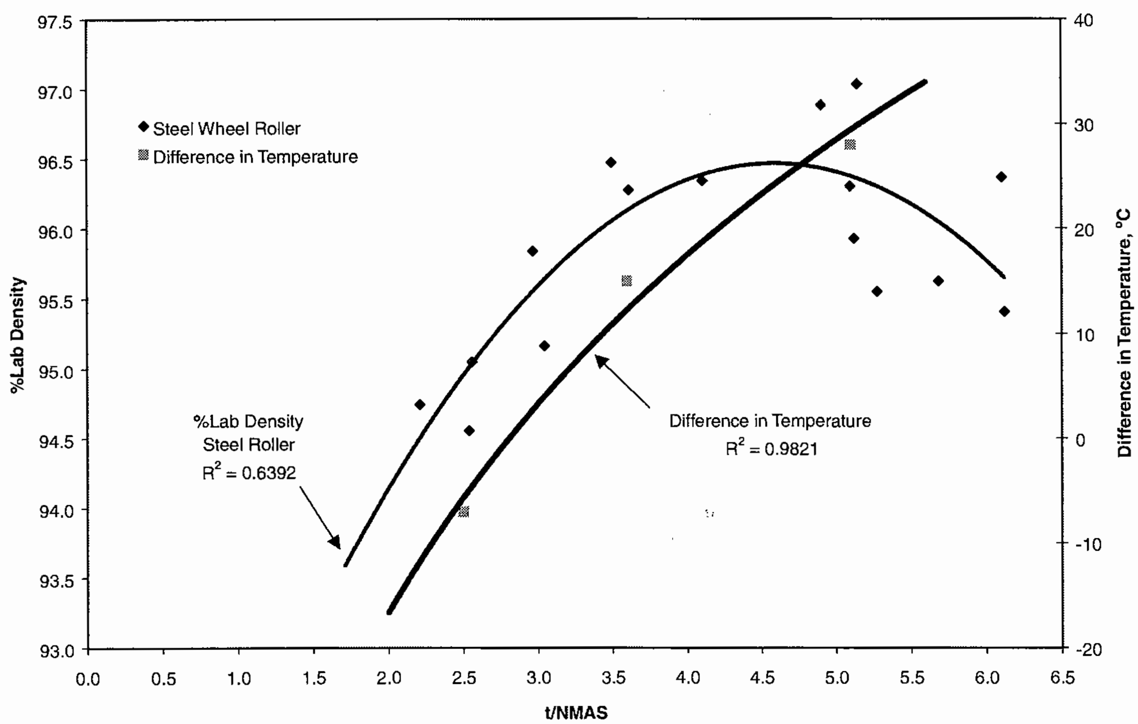

Figure 25. Relationships between density, t/NMAS, and temperature for Section 1.

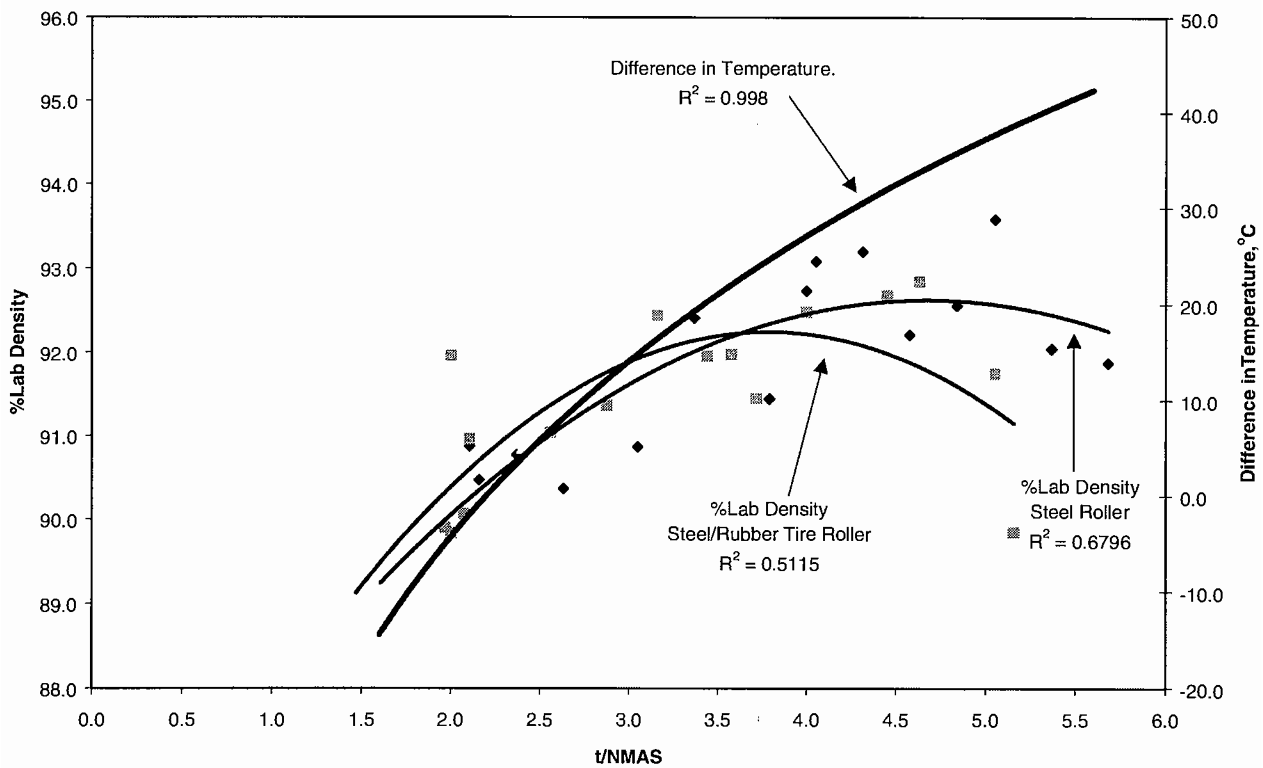

Figure 26. Relationships between density, t/NMAS, and temperature for Section 2.

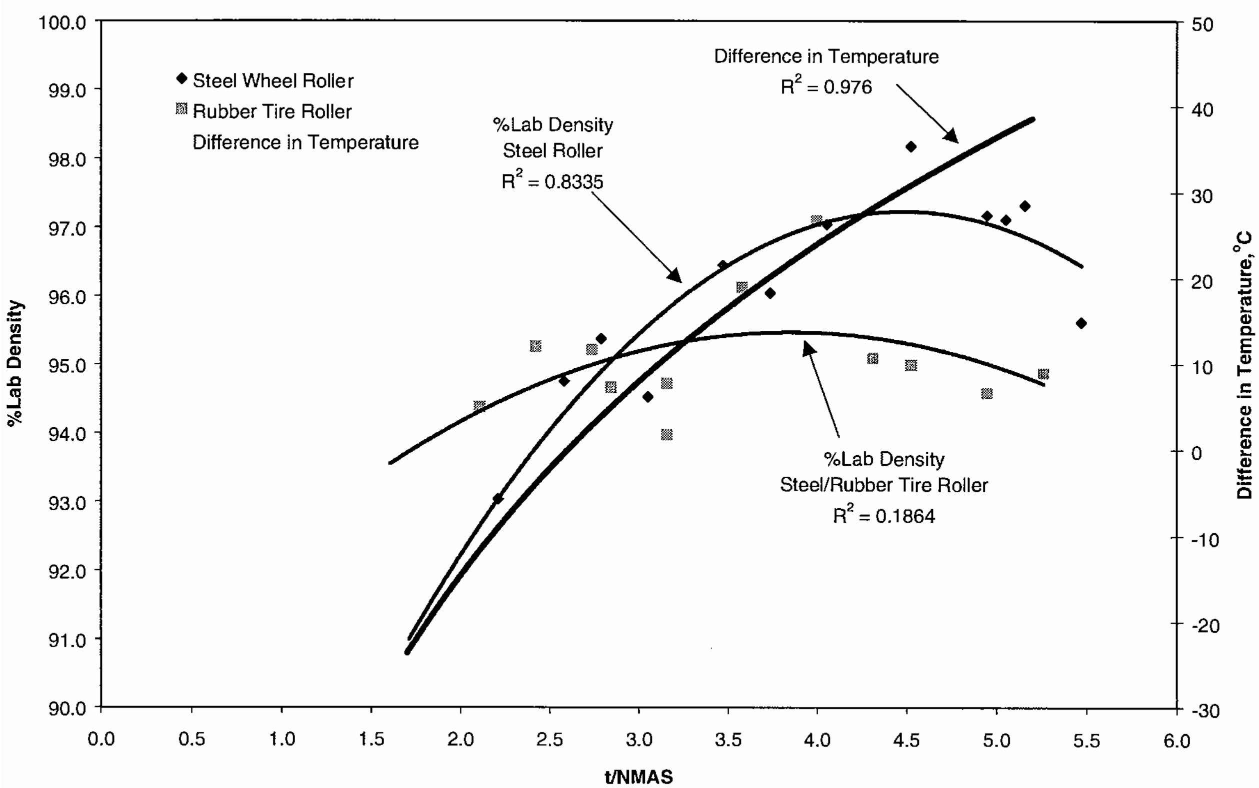

Figure 27. Relationships between density, t/NMAS, and temperature for Section 3.

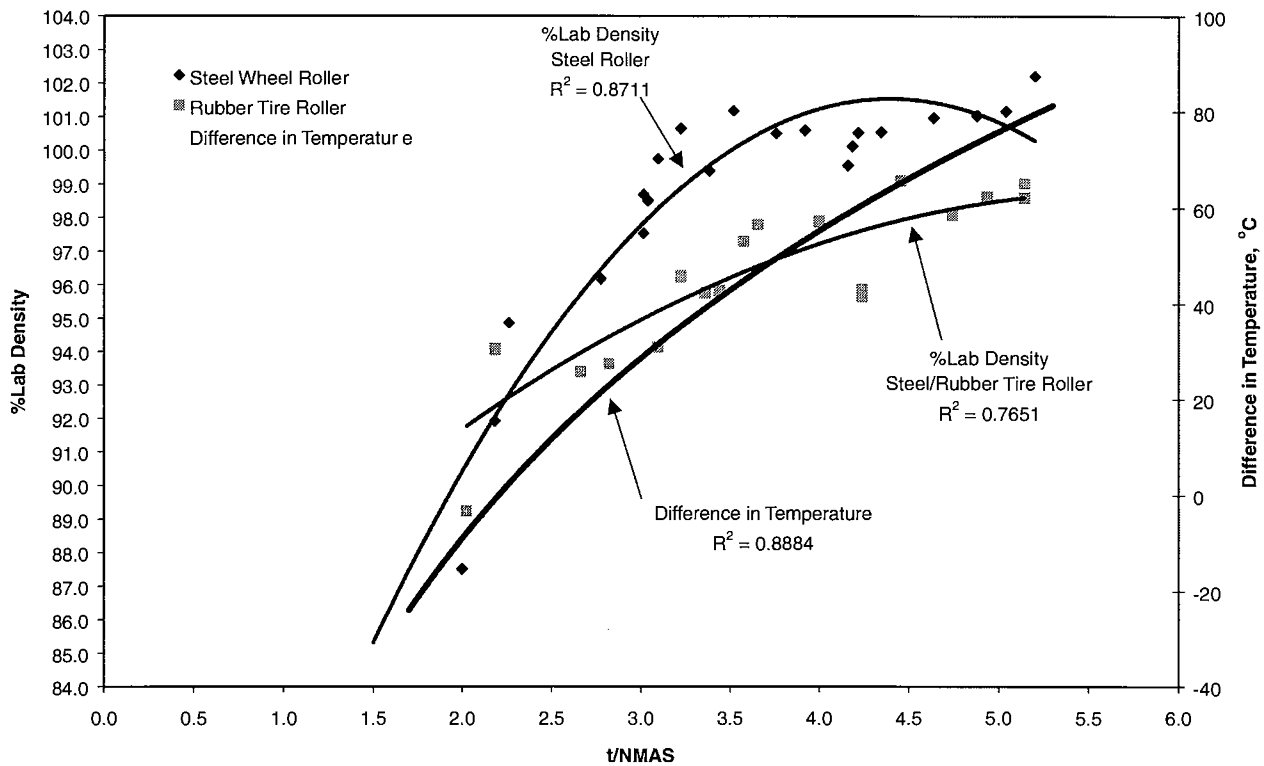

Figure 28. Relationships between density, t/NMAS, and temperature for Section 4.

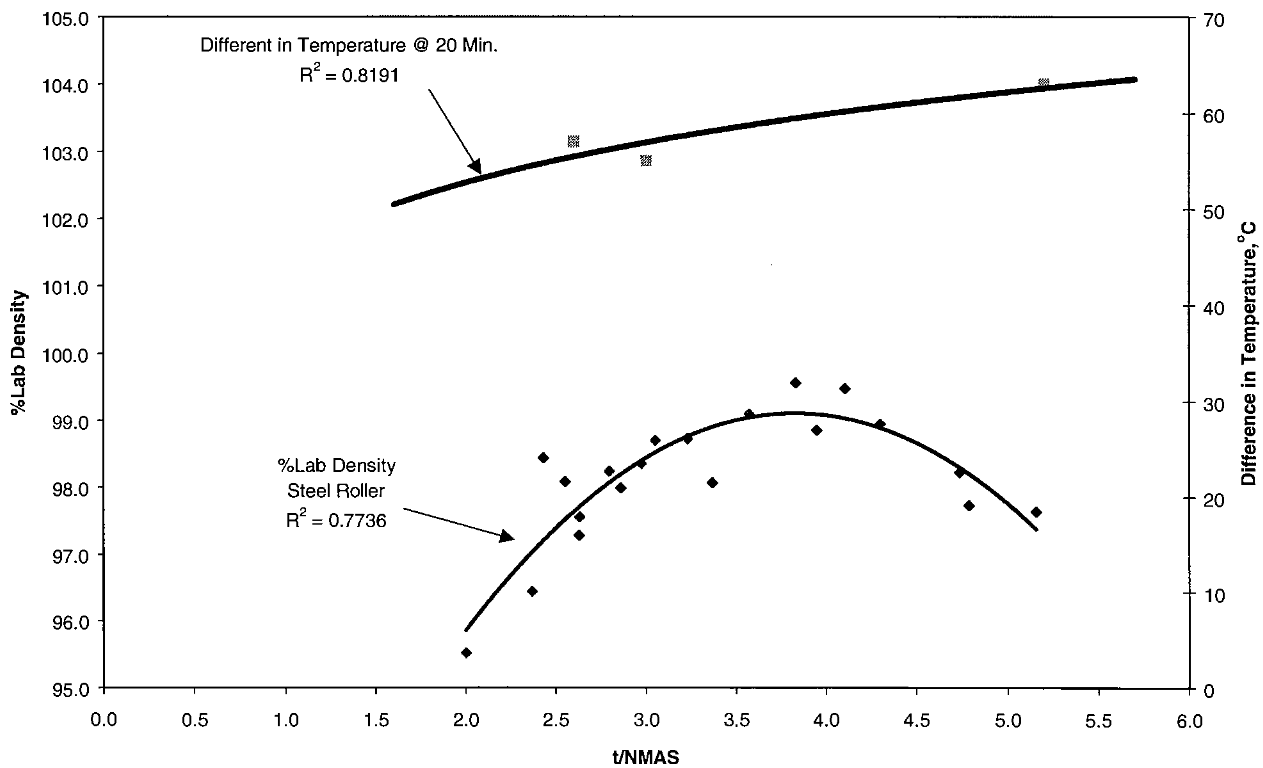

Figure 29. Relationships between density, t/NMAS, and temperature for Section 5.

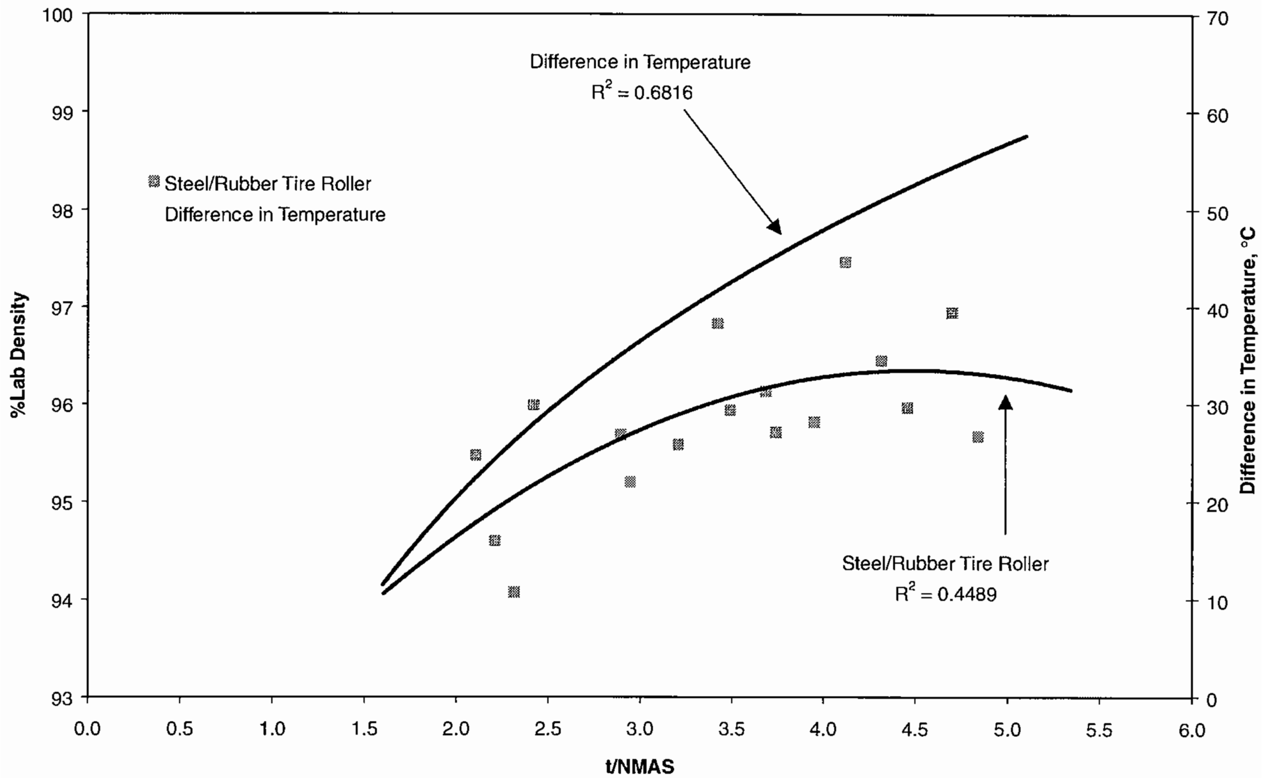

Figure 30. Relationships between density, t/NMAS, and temperature for Section 6.

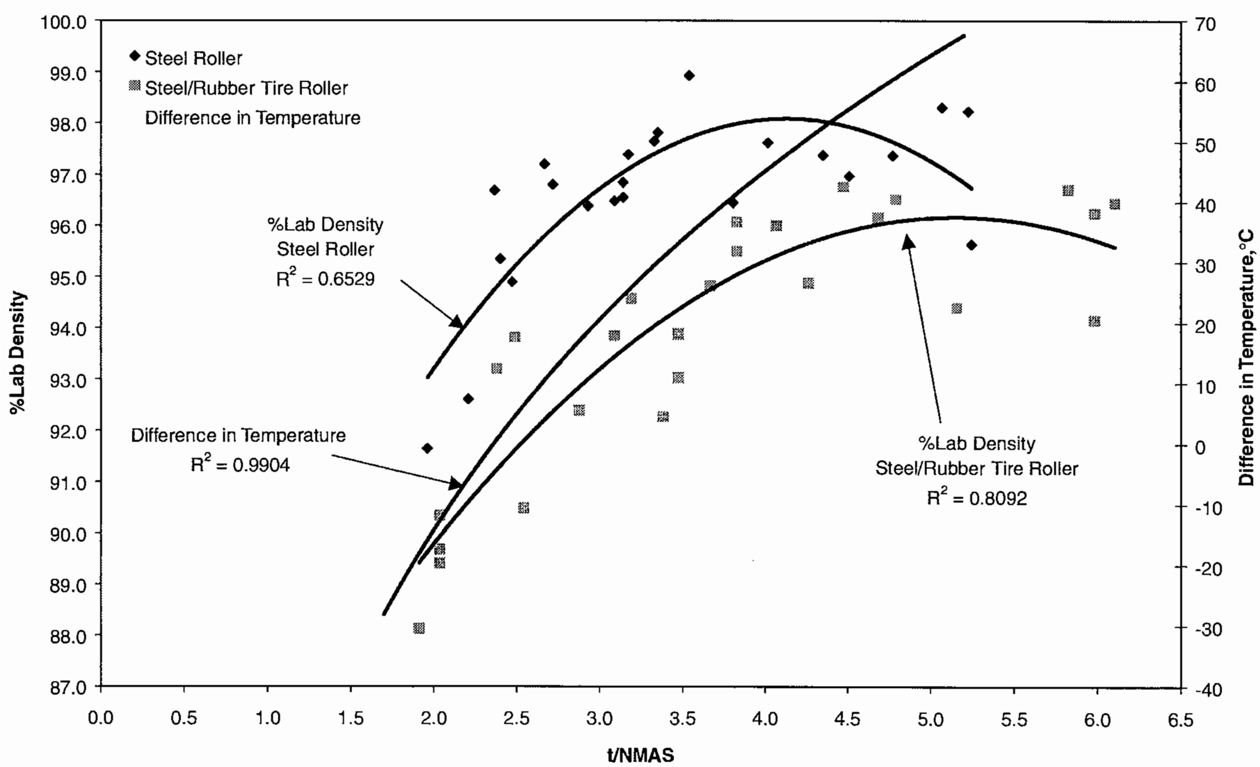

Figure 31. Relationships between density, t/NMAS, and temperature for Section 7.

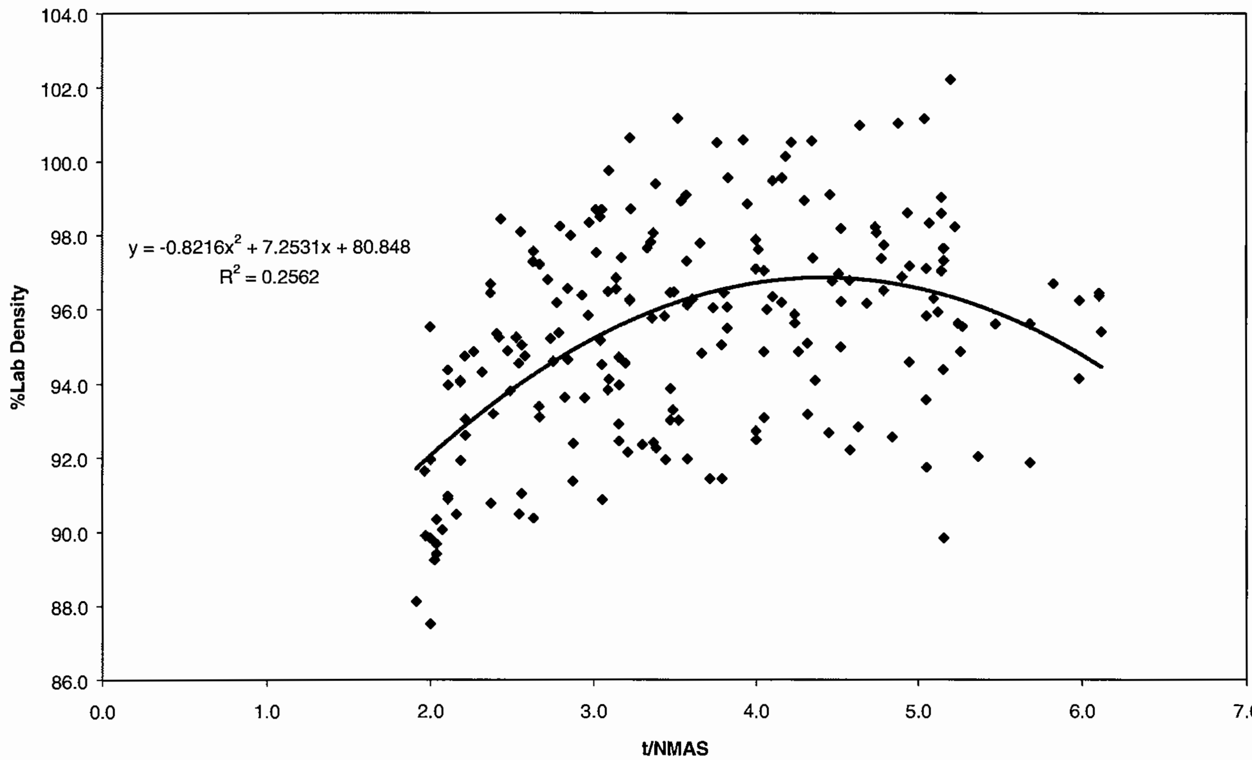

Figure 32. Relationships between density and t/NMAS for all sections.

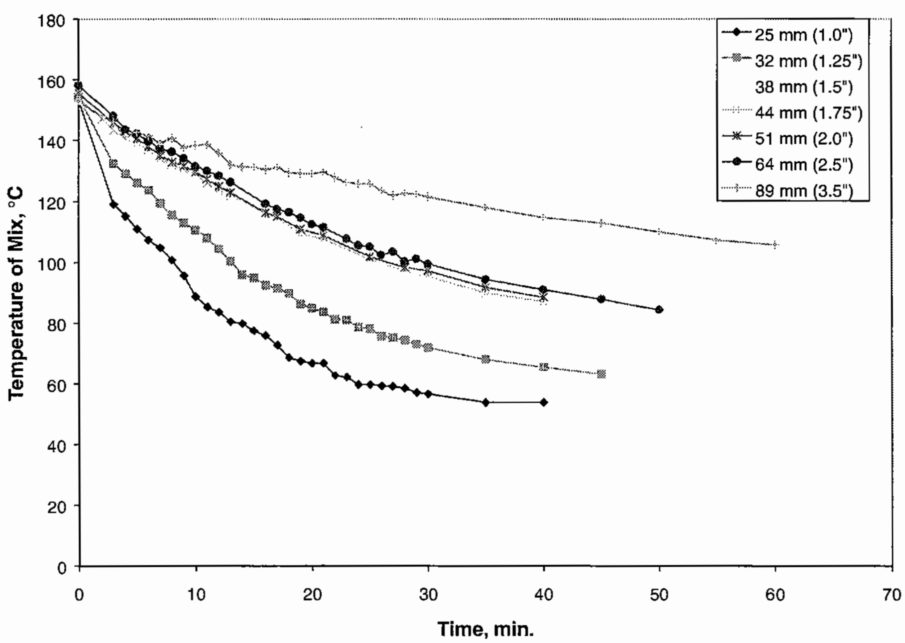

Figure 33. The effect of layer t/NMAS and cooling time on mix temperature.

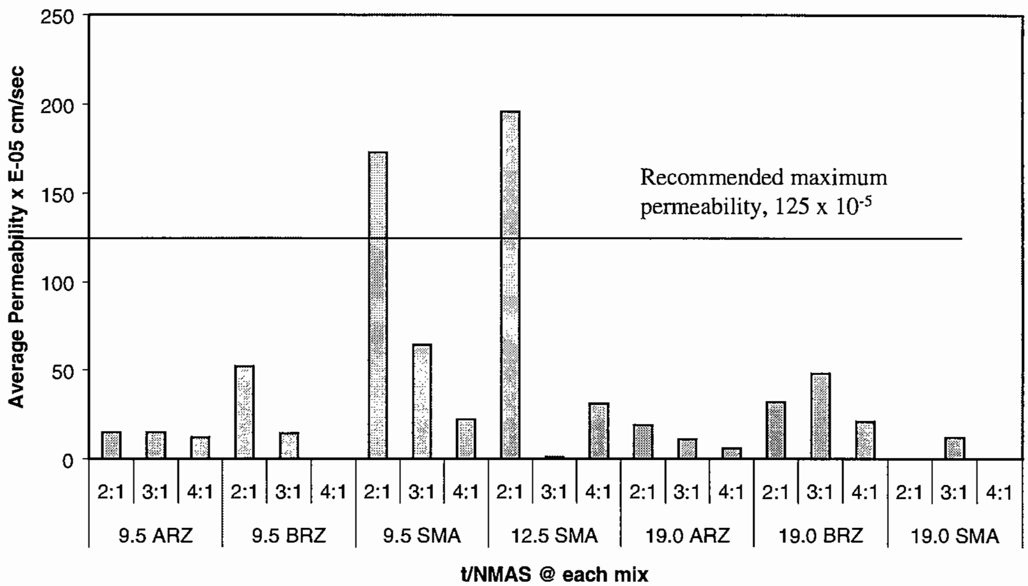

Figure 34. Relationships between permeability and t/NMAS.

less than a 3:1 t/NMAS, the air voids have to be lower to ensure that the mixes are impervious.

4.8 EVALUATION OF EFFECT OF t/NMAS ON PERMEABILITY FROM FIELD STUDY

Permeability tests were conducted on the seven HMA sections that were evaluated in the field. These tests were conducted in-place with the field permeameter and in the laboratory with the lab permeability test. Cores were taken from the in-place pavement for measurement of density and for measurement of lab permeability. The field permeability values were determined adjacent to the location where the cores were taken for density and for lab permeability. The results of these tests for the 7 sections are provided in Table 14.

In summary, the coarse-graded mixes had permeability values that exceeded the recommended value when the air voids exceeded about 8 percent. The fine graded mixes never exceeded the recommended value even up 9 to 10 percent air voids.

4.9 PART 2—EVALUATION OF RELATIONSHIP OF LABORATORY PERMEABILITY, DENSITY AND LIFT THICKNESS OF FIELD COMPACTED CORES

The average thickness, the average air void content by the vacuum seal device method, and the average laboratory permeability values were determined for each of the cores obtained from the work under NCHRP Project 9-9 (1). Figures 35 through 37 present the plots of in-place air voids versus permeability for each NMAS mix. The relationship between in-place air voids and permeability for 9.5-mm NMAS is illustrated in Figure 35. The R^2 values for both coarse-graded and fine-graded mixes were relatively high (0.70 and 0.86, respectively) and both relationships are significant (p-value = 0.000). At 8 percent air voids, the pavement is expected to have a permeability of 60×10^{-5} cm/sec for coarse-graded mix and 10×10^{-5} cm/sec for fine-graded mix. Because there are only a couple of data points for fine-graded mix above approximately 10 percent air voids, this model should not be used to predict permeability at these higher void levels. At

TABLE 14 Comparison of laboratory and field permeabilities

Section Number	Mix Type	In-Place Air Voids (percent)	Field Permeability (cm/s x 10^{-5})	Lab Permeability (cm/s x 10^{-5})
1	9.5mm FG	6.6 to 8.8	1 to 28	1 to 35
2	9.5mm CG	9.0 to 12.6	14 to 632	107 to 1070
3	9.5mm SMA	7.7 to 12.6	110 to 651	29 to 168
4	12.5mm SMA	4.1 to 17.9	3 to 1778	0.1 to 5850
5	19.0mm FG	5.7 to 9.5	38 to 161	1 to 77
6	19.0mm CG	5.3 to 9.8	10 to 1760	1 to 141
7	19.0mm CG	4.8 to 15.2	72 to 3030	0 to 1203

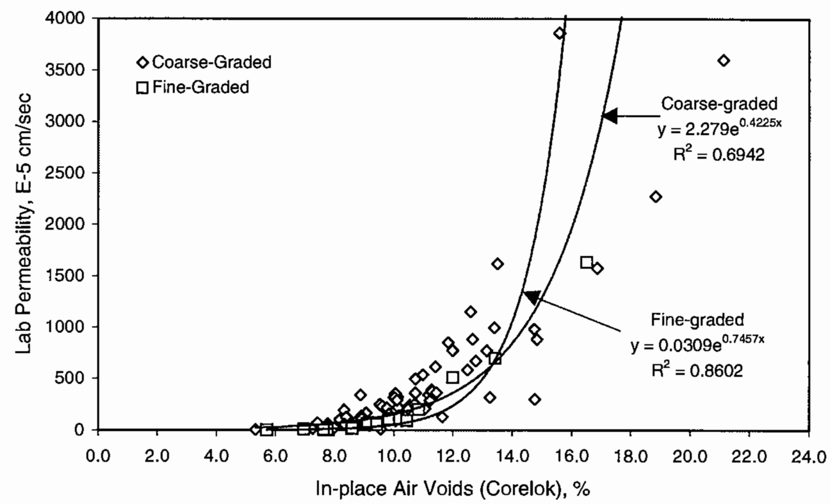

Figure 35. Plot of permeability versus in-place air voids for 9.5-mm NMAS mixes.

lower void levels the coarse-graded mixes are more permeable than fine-graded mixes.

The relationships for the coarse-graded and fine-graded 12.5-mm NMAS mixes are shown in Figure 36. For these projects there was no significant difference between fine and coarse graded mixes. The relationships between in-place air voids and permeability for both gradation types were reasonable and significant with an R^2 of 0.61 for coarse-graded mixes (p-value = 0.000) and 0.58 for fine-graded mixes (p-value = 0.000). As shown by the best-fitted lines, the permeability values for both gradation types were basically the same at a given air void content. The permeability at 8.0 percent air voids for coarse-graded and fine-graded mixes was approximately 30×10^{-5} cm/sec.

Figure 37 illustrates the relationship between in-place air voids and permeability for fine-graded 19.0-mm NMAS mixes. The R^2 value for this figure is 0.59 and the relationship is significant (p-value = 0.000). Based on the trend line, permeability is very low at air void contents less than 8 percent. At air void contents above 8 percent, the permeability begins to increase rapidly with a small increase in in-place air void content. At 8 percent air voids, the fine-graded 19.0-mm NMAS mix has a permeability value of 16×10^{-5} cm/sec.

4.10 CONTROLLED LABORATORY EXPERIMENT TO EVALUATE METHODS OF MEASURING THE BULK SPECIFIC GRAVITY OF COMPACTED HMA

4.10.1 Introduction and Problem Statement

A major concern of the HMA industry is the proper measurement of bulk specific gravity (G_{mb}) for compacted samples. This issue has become a bigger problem with the increased

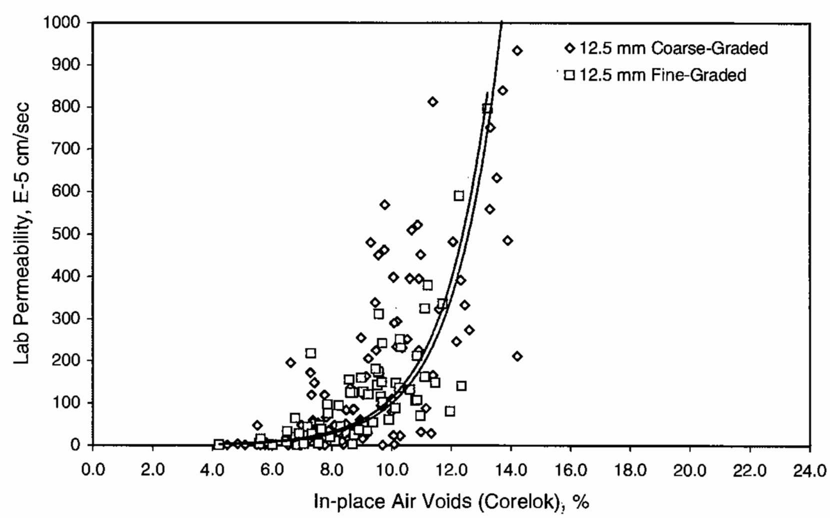

Figure 36. Plot of permeability versus in-place air voids for 12.5-mm NMAS Mixes.

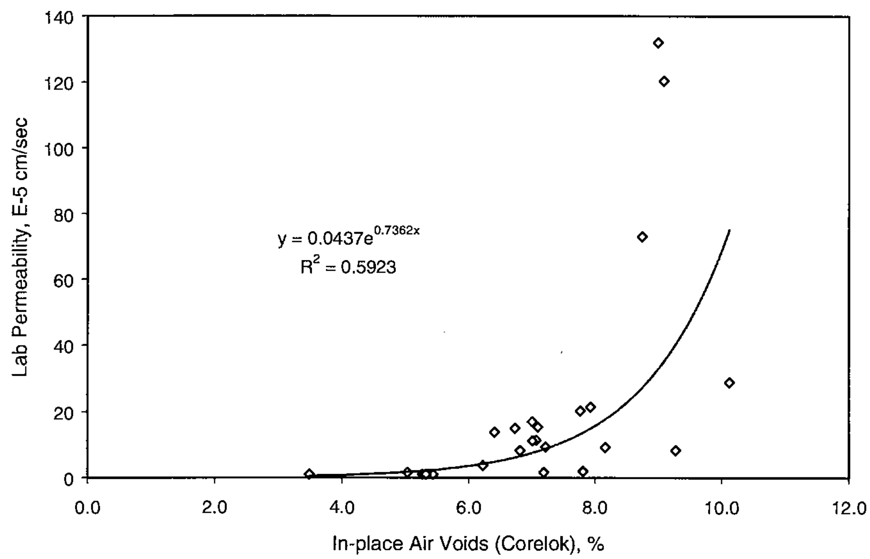

Figure 37. Plot of permeability versus in-place air voids for 9.5-mm NMAS mixes.

use of coarse gradations. Bulk specific gravity measurements are the basis for volumetric calculations used during HMA mix design, field control, and construction acceptance. During mix design, volumetric properties such as air voids, voids in mineral aggregates, voids filled with asphalt, and percent theoretical maximum density at a certain number of gyrations are used to evaluate the acceptability of mixes. All of these properties are based upon G_{mb}.

In most states, acceptance of HMA construction by the owner is typically based upon percent compaction (density based upon G_{mb} and theoretical maximum density). Whether nondestructive (e.g., nuclear gauges) or destructive (e.g., cores) tests are used as the basis of acceptance, G_{mb} measurements are equally important. When nondestructive devices are utilized, each device first has to be calibrated to the G_{mb} of cores. If the G_{mb} measurements of the cores are inaccurate in this calibration step, then the nondestructive device will provide inaccurate data. Additionally, pay factors for construction, whether reductions or bonuses, are generally based upon percent compaction. Thus, errors in G_{mb} measurements can potentially affect both the agency and producer.

For many years, the measurement of G_{mb} for compacted HMA has been accomplished by the water displacement method using saturated-surface dry (SSD) samples. This method consists of first weighing a dry sample in air, then obtaining a submerged mass after the sample has been placed in a water bath for a specified time interval. Upon removal from the water bath, the SSD mass is determined after patting the sample dry using a damp towel. Procedures for this test method are outlined in AASHTO T166 (ASTM D2726).

The SSD method has proven to be adequate for conventionally designed mixes, such as those designed according to the Marshall and Hveem methods, that generally utilized fine-graded aggregates. Historically, mixes were designed to have gradations passing close to or above the Superpave defined maximum density line (i.e., fine-graded). However, since the adoption of the Superpave mix design system and the increased use of SMA, mixes are being designed with coarser gradations than in the past.

The potential problem in measuring the G_{mb} of mixes like coarse-graded Superpave and SMA using the SSD method comes from the internal air void structure within these mix types. These types of mixes tend to have larger internal air voids than the finer conventional mixes, at similar overall air void contents. Mixes with coarser gradations have a much higher percentage of large aggregate particles. At a certain overall air void volume, which is mix specific, the large internal air voids of the coarse mixes can become interconnected. During G_{mb} testing with the SSD method, water can quickly infiltrate into the sample through these interconnected voids. However, after removing the sample from the water bath to obtain the saturated-surface dry condition the water can also drain from the sample quickly. This draining of the water from the sample is what causes errors when using the SSD method.

Because of the potential errors noted with the saturated surface-dry test method of determining the bulk specific gravity of compacted HMA, the primary objective of this task was to compare AASHTO T166 with other methods of measuring bulk specific gravity to determine under what conditions AASHTO T166 is accurate.

The plan for this part of the study was to evaluate two separate sample types: laboratory compacted and field compacted. Laboratory compacted mixtures having various aggregate types, nominal maximum aggregate sizes, gradation shapes, and air void levels were prepared. Each of the prepared samples was tested to determine bulk specific gravity by four different test methods: water displacement, vacuum-sealing, gamma ray, and dimensional.

For the field compacted samples, cores obtained during the field validation portion of this study were subjected to the same four bulk specific gravity test methods. Because cores have a different surface texture than laboratory compacted samples, it was necessary to evaluate them also. Testing also conducted on core samples included laboratory permeability tests and effective air void content using the vacuum-sealing device.

4.10.2 Field Compacted Samples

Each of the cores obtained during the Task 5 field validation were tested to determine bulk specific gravity using the same four tests as the laboratory experiment: water displacement, vacuum sealing, gamma ray, and dimensional analysis. Because of the differences in surface texture between laboratory compacted samples (surface texture around entire sample) and field compacted samples (surface texture only on top of sample because of core bit and sawing), the experiment was also extended to core samples.

Because of the differences in resulting air voids for the four methods of measuring bulk specific gravity, a Duncan's multiple range test (DMRT) was conducted to determine which methods, if any, provided similar results. This analysis method provides a ranking comparison between the different methods. The range of sample means for a given set of data (method) can be compared to a critical valued based on the percentiles of the sampling distribution. The critical value is based on the number of means being compared (four, representing the different methods) and number of degrees of freedom at a given level of significance (0.05 for this analysis). Results of the DMRT analysis for the Superpave mixes are illustrated in Figure 38.

Statistically, results of the DMRT comparisons show that all methods produced statistically different air void contents. However, vacuum-sealing and gamma ray bulk specific gravity methods provided similar results given a difference of 0.24 percent air voids. On average, the dimensional method resulted in the highest air void contents, followed by the vacuum-sealing and gamma ray methods, respectively. Air void contents determined from AASHTO T166 resulted in the lowest air void contents. None of the alternative methods provided similar results to AASHTO T166.

The results for SMA mixtures are provided in Figure 39. As with the Superpave mixes, the vacuum-sealing and gamma ray methods resulted in similar air void contents. The dimensional method again resulted in the highest air voids and the AASHTO T166 method resulted in the lowest air voids. Analysis of both the Superpave and SMA data indicated that the four methods of measuring bulk specific gravity significantly affected resulting air voids. For both mix types, the vacuum-sealing and gamma ray methods provided similar air voids; however, the dimensional method provided significantly higher air voids and AASHTO T166 provided significantly lower air void contents.

Theoretically, the dimensional method should provide the highest measured air void content, as this method includes both the internal air voids and the surface texture of the sample. Therefore, the results in Figures 38 and 39 pass the test of reasonableness for the vacuum-sealing, gamma ray, and AASHTO T166 methods as all three provided air void content lower than the dimensional method.

Because it was assumed that the T-166 method would be accurate at low water absorption levels, it was decided to test the mixes with low absorption, less than 0.5 percent, to see which mixes provided results similar to the T-166 method. The results are provided in Figure 40. This figure shows that the vacuum-sealing and AASHTO T166 methods provided

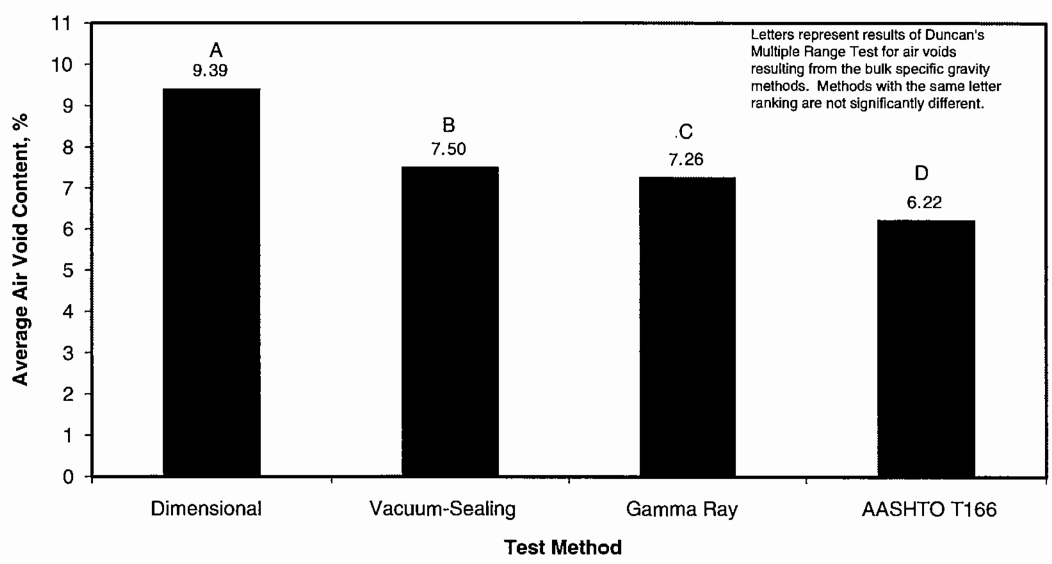

Figure 38. Average air voids and DMRT results for Superpave mixes.

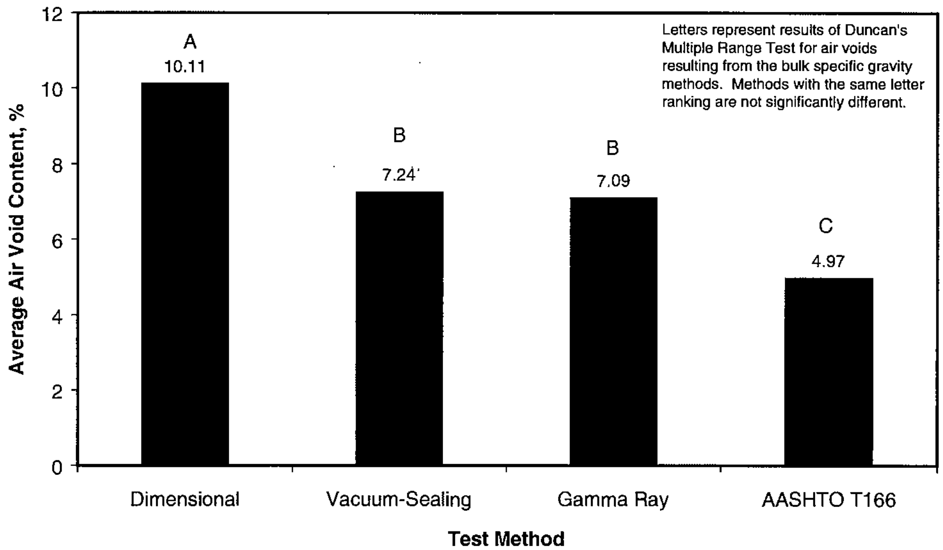

Figure 39. Average air voids and DMRT results for SMA mixes.

similar results and that both were significantly different than the dimensional and gamma ray methods. The dimensional method provided the highest air void content, as expected. The AASHTO T166 method is accurate for low water absorption mixes and at these low void levels provide similar density values to that of the vacuum seal method. These results suggest that the vacuum-sealing method provides an accurate density for low voids, which indicates that it also provides an accurate density at higher void levels because the plastic seal will clearly prevent water from being absorbed into the mixture. Figures 38 and 39 suggest that the gamma ray method does an overall adequate job of estimating bulk specific gravity; however, Figure 40 suggests that it is not as accurate as AASHTO T166 or the vacuum-sealing methods. Refinements to the gamma ray method may make this method a viable option in the future.

4.10.3 Analysis of Field Compacted Samples

Included within this portion of the study were the cores obtained during the Task 5 field validation experiment. Only the vacuum-sealing and AASHTO T166 test methods were analyzed, as they were shown most accurate during the laboratory phase of this experiment. Figure 41 illustrates the relationship between air voids determined from the two methods for all field cores obtained from the 20 field projects during Task 5. This figure illustrates that when air void content is less

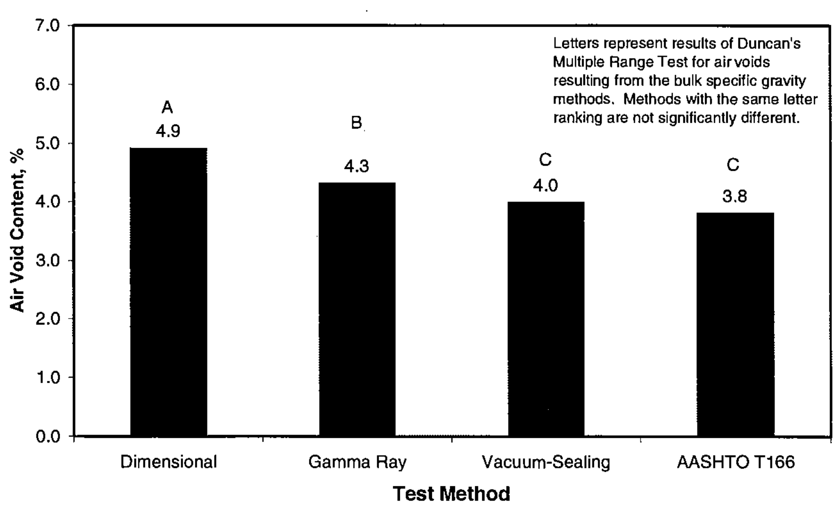

Figure 40. Comparison of test methods, mixes with low water absorption level.

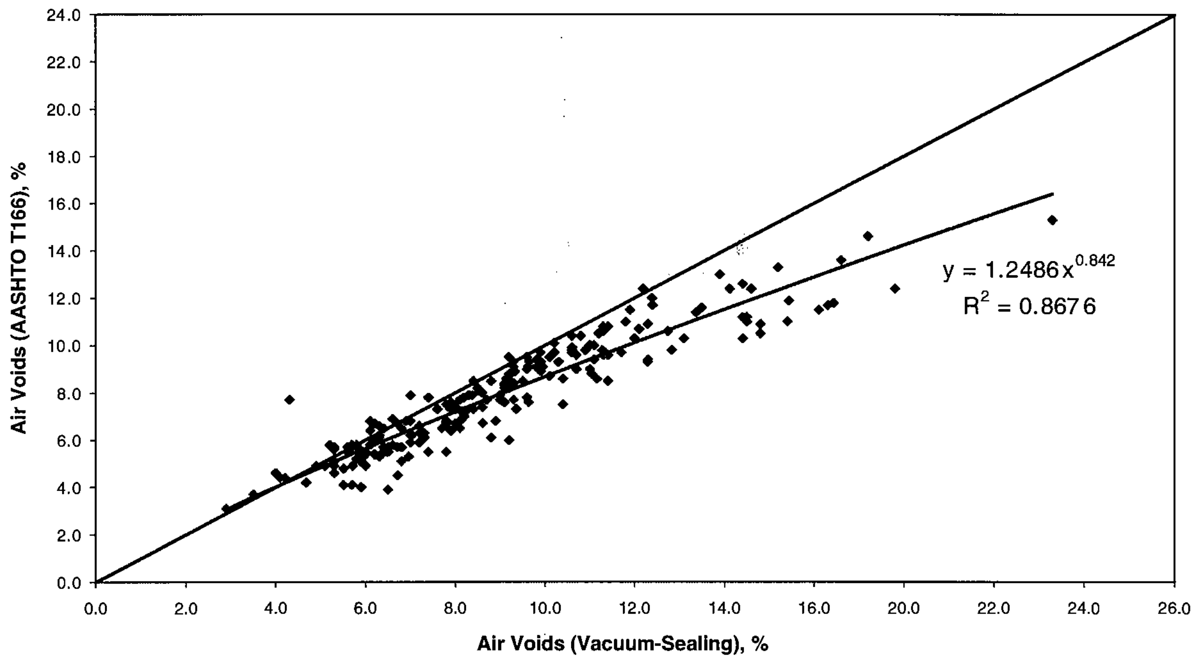

Figure 41. Comparisons between AASHTO T166 and vacuum-sealing methods, field projects.

than about 5 percent, the two methods provided approximately similar results. Above 5 percent air voids, the vacuum-sealing method resulted in higher air void contents. As air voids increased, the two methods diverged and it is believed that the reason for this divergence is the loss of water during the SSD method. Hence, at low air voids, both methods should be close to correct; however, at higher air voids the vacuum-sealing method should be more correct.

4.11 FIELD VALIDATION OF RELATIONSHIPS BETWEEN PERMEABILITY, LIFT THICKNESS, AND IN-PLACE DENSITY

The main objective of the field portion of NCHRP 9-27 (Task 5) was to provide a field validation of the relationships between permeability, lift thickness, and in-place density so the overall objectives of the study could be accomplished. In order to field verify the relationships between air voids, lift thickness, and permeability, 20 HMA construction projects were visited. Testing at these projects included tests on plant-produced mix and on the compacted pavement. Testing of the plant produced mix included compacting samples to both the design compactive effort and to a specified height. Testing on the compacted pavement included performing field permeability tests with the NCAT Field Permeameter. Selection of the 20 projects was based upon the following factors: NMAS, gradation type (fine-graded, coarse-graded, and SMA), and the lift thickness to NMAS ratio (t/NMAS). Table 15 presents the 20 projects evaluated.

Table 15 shows that both fine- and coarse-graded Superpave designed mixes were investigated for each of four NMAS, ranging from 9.5 to 25.0 mm NMAS. SMA mixes were investigated for 12.5 and 19.0 mm NMASs. The effect of lift thickness was evaluated within the 9.5, 12.5, and 19.0 mm NMASs. To determine if a general trend occurred between in-place air voids and t/NMAS, a regression was performed on the combined data. Figure 42 illustrates this general relationship. From this regression, a low R^2 of 0.09 was found. The trendline suggested that as the ratio of lift thickness to NMAS increased, in-place air voids decreased.

To determine if the relationship between in-place air voids and the t/NMAS ratio was significant, an ANOVA was conducted on the regression. For the combined data, the p-value was 0.014, which indicated that the overall relationship was significant. Then the data were separated into the three mix types. When an ANOVA was conducted on the regressions for the mix types, it was found that the relationship was not significant for any of the mix types (p-values of 0.956, 0.994, and 0.107 for fine-graded, coarse-graded, and SMA, respectively). There is a lot of scatter in the data, but, as can be seen in Figure 42, every increase of 1 in the t/NMAS results in a decrease in voids of approximately 0.6 percent. This finding involves average numbers, and it must be realized that many other factors affect the density of these field projects.

Another factor to consider for these projects is the specification requirements were approximately the same for all of these mixes. Hence, the contractor was trying to compact all mixes to a low void content. Even with the same target density the t/NMAS affected the results.

For Figure 43, a best-fit line was produced on the combined data for the 12.5-mm NMAS mixes. A low correlation was also found for this regression (0.19), but the general trend suggested that in-place air voids decreased as the lift

TABLE 15 Field project summary information

Project ID	NMAS	Fine or Coarse Gradation	Average Lift Thickness (mm)	Actual Lift Thickness/ NMAS Ratio	AC Performance Grade	Ndesign
1	9.5	Fine	48.7	5.1:1	70-22	65
2	19.0	Coarse	65.7	3.5:1	64-22	65
3	9.5	Coarse	32.3	3.4:1	64-22	65
4	12.5	Fine	68.6	5.5:1	*	75
5	9.5	Fine	41.0	4.3:1	70-22	100
6	12.5	Coarse	50.3	4.0:1	58-28	75
7	9.5	Fine	40.6	4.3:1	64-28	75
8	19.0	Coarse	58.9	3.1:1	64-22	100
9	19.0	Coarse	96.4	5.1:1	64-22	100
10	19.0	Coarse	70.9	3.7:1	64-34	100
11	19.0	Coarse	38.0	2.0:1	64-34	125
12	25.0	SMA	42.6	1.7:1	76-22	50
13	25.0	Fine	70.0	2.8:1	67-22	100
14	9.5	SMA	26.8	2.8:1	76-22	75
15	19.0	Coarse	50.4	2.7:1	76-22	100
16	12.5	Coarse	43.8	3.5:1	67-22	86
17	12.5	Fine	43.3	3.5:1	64-22	75
18	12.5	Coarse	44.5	3.6:1	67-22	75
19	9.5	Fine	41.5	4.4:1	67-22	75
20	12.5	Fine	34.5	2.8:1	67-22	80

* Designated RA295 by the agency

thickness increased. An ANOVA conducted for the combined regression indicated that the relationship was significant (p-value = 0.001). The data were then separated into the different mix types to see if the relationship was significant for each mix type. For the fine-graded mixes, the relationship was significant (p-value = 0.000). The coarse-graded mixes did not have a significant relationship between in-place air voids and t/NMAS (p-value = 0.932). These data indicate that an increase of 1 for the t/NMAS resulted in an average decrease in air voids of 0.5 percent.

Figure 44 shows the relationship between lift thickness and in-place air voids for the combined data set for the 19.0-mm NMAS mixes, as well as for the individual mix types. For the combined data, the regression produced a low R^2 value (0.09). An ANOVA performed on the regression determined that the relationship between t/NMAS and in-place air voids for the 19.0-mm NMAS mixes was significant (p-value of 0.000). The data indicate that an increase of 1 for the t/NMAS results in an average decrease of 1.0 in the air voids.

In summary, even though there is a large amount of scatter in the data for the three NMAS mixes, the results suggest that the air voids dropped 0.5 to 1.0 percent for each increase of 1 in the t/NMAS. This shows the importance of making sure that the t/NMAS is sufficiently high.

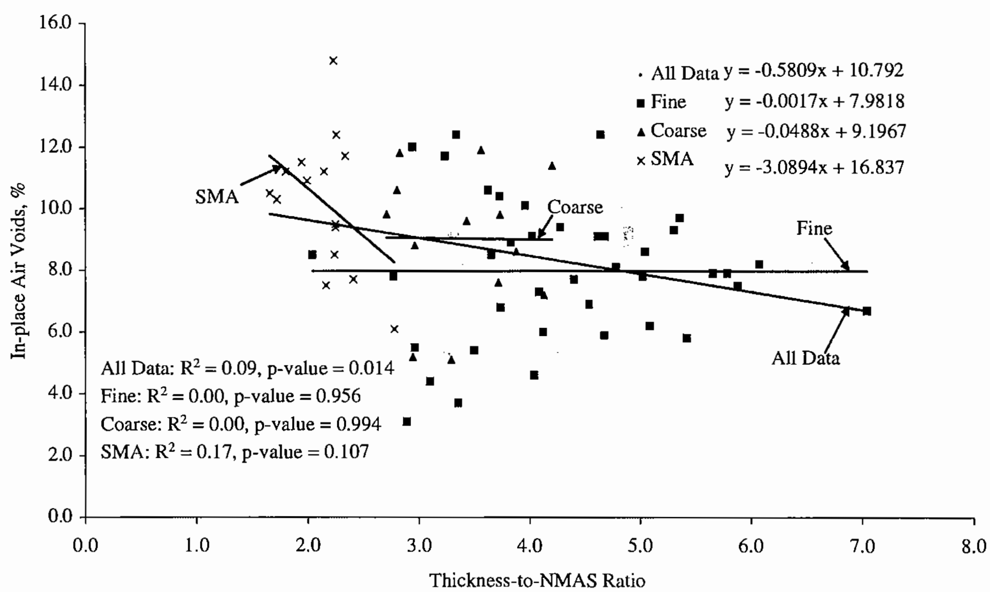

Figure 42. Relationship between t/NMAS and in-place air voids—9.5 mm, all data.

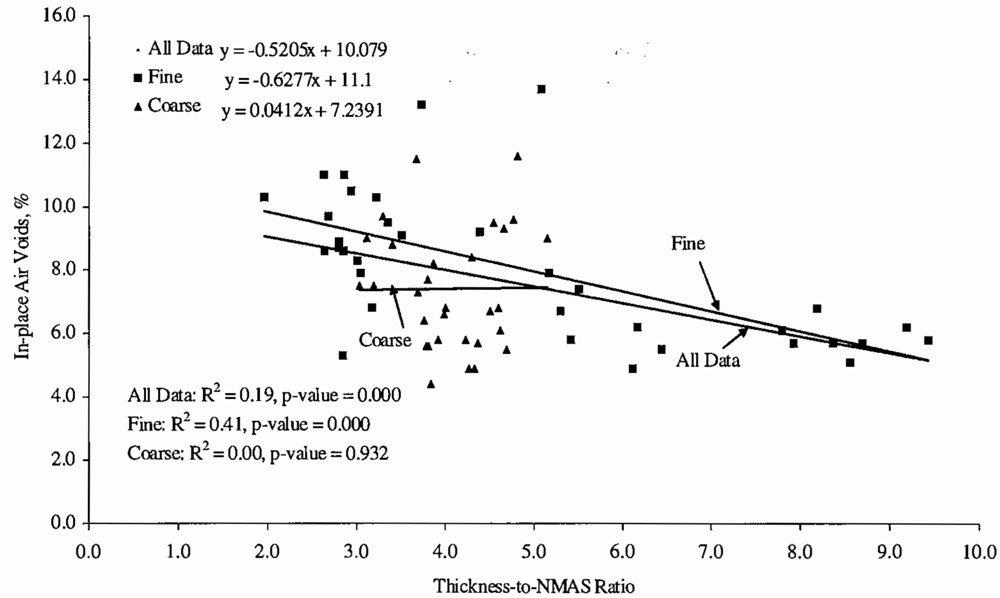

Figure 43. Relationship between t/NMAS and in-place Air Voids—12.5 mm NMAS.

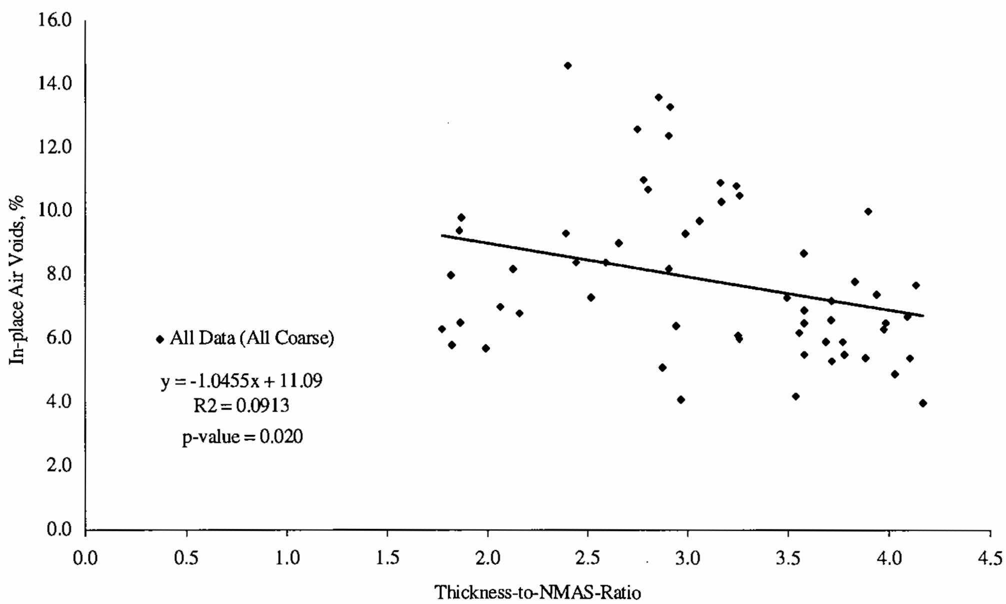

Figure 44. Relationship between t/NMAS and in-place air voids—19.0 mm NMAS.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

The density that can be obtained under normal rolling conditions is clearly related to the t/NMAS. For improved compactibility, it is recommended that the t/NMAS be at least 3 for fine-graded mixes and at least 4 for coarse-graded mixes. The data for SMA indicate that the ratio should also be at least 4. Ratios less than these suggested numbers could be used, but more compactive effort would generally be required to obtain the desired density. In most cases, a t/NMAS of 5 does not result in the need for more compactive effort to obtain maximum density. However, care must be exercised when the thickness gets too large to ensure that adequate density is obtained.

The results of the evaluation of the effect of mix temperature on the relationship between density and t/NMAS indicate that one of the reasons for low density at thinner sections (lower t/NMAS) is the more rapid cooling of the mixture. Hence, for thinner layers it is even more important that rollers stay very close to the paver so that rolling can be accomplished prior to excessive cooling. For the conditions of this study, the mixes placed at the NCAT test track at 25-mm thickness cooled twice as fast as mixes placed at 37.5-mm thickness. For thicker sections (larger t/NMAS), the rate of cooling is typically not a problem.

The in-place void content is the most significant factor impacting permeability of HMA mixtures. This is followed by coarse aggregate ratio and VMA. As the values of coarse aggregate ratio increases, permeability increases. Permeability decreases as VMA increases for constant air voids.

The variability of permeability between various mixtures is very high. Some mixtures are permeable at the 8 to 10 percent void range and others do not seem to be permeable at these higher voids. However, to ensure that permeability is not a problem, the in-place air voids should be between 6 and 7 percent or lower. This appears to be true for a wide range of mixtures regardless of NMAS and grading.

When laboratory prepared samples having low levels of water absorption were evaluated, the dimensional method resulted in the highest air void contents followed by the gamma ray method. The vacuum-sealing and water displacement (AASHTO T166) methods resulted in similar air void contents when the water absorption level was low. The vacuum seal method is an acceptable method to use for low and high void levels.

At low levels of water absorption, the water displacement method is an accurate measure for bulk specific gravity. The error develops when removing the sample from water to determine the SSD weight. When water flows out of the sample, an error occurs. The allowable absorption level to use the displacement test method is specified as 2 percent in AASHTO T166, but this level of absorption can create accuracy problems, as shown in this report. It is recommended that the absorption limit for the displacement test method be reduced to 1 percent. If the vacuum-seal method is adopted on a project, the measured voids may now be somewhat higher than with the water displacement method.

The water displacement method was accurate for all water absorption levels encountered for mixes that were fine-graded (ARZ gradations). For mixes having gradations near the maximum density line (TRZ) or coarser (BRZ and SMA), the level of water absorption at which AASHTO T166 began to lose accuracy was between 0.2 and 0.4 percent.

For mix design samples and other laboratory samples that are compacted to relatively low voids, the displacement method will provide reasonably accurate answers. However, for field samples where the void levels will typically be 6 percent or higher, it is important to evaluate absorption to determine if the vacuum-seal method needs to be used.

Care must be used when using the vacuum sealing method to measure density. Many times the plastic bag develops a leak during the test, leading to an error in the result. Weighing the sample in air after measuring the submerged weight will indicate if a leak has developed. If a leak is identified, the test must be repeated until an acceptable test is achieved.

There appears to be a need for a correction factor for the vacuum-sealing and water displacement methods to provide equal measured air void contents even when the air void level is low. The correction factor for the mixtures evaluated in this report was approximately 0.2 percent air voids. A better determination of the correction factor can be made for specific dense graded mixes by compacting samples in the Superpave gyratory compactor to approximately 4 percent air voids (design air void content) and testing using the two test methods. The difference between these two tests will be the correction factor for the mix.

The in-place air voids of the 20 field projects were high. Fourteen of the 20 mixes tested had average in-place air voids above 8 percent and seven of the mixes had average air voids over 10 percent (based on test results with the vacuum-seal method). This low density on a high percentage of random projects is disturbing because this lower density will most certainly lead to significant loss in pavement life.

More emphasis must be placed on obtaining adequate density. Regardless of the method of density measurement used, some cores have to be taken and tested for calibration. The most reliable way to measure density is to take cores for density testing. If the amount of absorption during density measurement exceeds 1 percent, the T166 method will likely provide a higher measured density than the true density. The vacuum seal method is one approach to measure a density more accurately when the water absorption exceeds 1 percent.

Even though there is a lot of scatter within and between projects, most field results support the finding that higher t/NMAS ratios generally provide lower void levels. Coarse-graded mixtures generally have higher permeability values than the fine-graded mixtures for a given air void level. Air voids were clearly shown to be a key determinant of permeability. However, many times the air voids were reasonably low (5 to 7 percent) and the permeability was still high.

CHAPTER 6

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APPENDICES A THROUGH E UNPUBLISHED MATERIAL

Appendices A, B, C, D, and E as submitted by the research agency are not published herein. For a limited time, they are available for loan on request to NCHRP. Their titles are as follows:

Appendix A: Mix Design Summary Information for Part 1
Appendix B: Lift Thickness Versus Density Data Using Gyratory Compactor
Appendix C: Lift Thickness Versus Density Data Using Vibratory Compactor
Appendix D: Lift Thickness Versus Permeability Data
Appendix E: Factors Affecting Permeability Data Using Field Core Samples

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

NCHRP Web Document 34 (Project D9-9[1]): Contractor's Final Report

Literature Review: Verification of Gyration Levels in the Superpave N_{design} Table

Prepared for:

National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Submitted by:

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Auburn University, Alabama

January 2001

ACKNOWLEDGMENT

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CHAPTER 1 INTRODUCTION

1.0 BACKGROUND

The Superpave gyratory compactor (SGC) is utilized as the compaction device in the mix design and field production control of hot mix asphalt. The design number of gyrations, N_{design}, used in the Superpave system was originally established based on a limited set of field data. The N_{design} level, for dense-graded mixes, was based on the design high air temperature (average seven day high air temperature) of the paving location and the traffic level in terms of equivalent single axle loads (ESALs). Ideally, the N_{design} used in the laboratory mix design for a given mix and design ESAL level should result in that mix ultimately achieving a stable density equal to the laboratory mix design density.

The original Superpave N_{design} table of 28 levels has been reduced through the National Cooperative Highway Research Program (NCHRP) 9-9 and the Federal Highway Administration (FHWA) mixture Expert Task Group (ETG), and other efforts, to four levels (50, 75, 100, and 125 gyrations). These four levels were selected to represent a range of traffic from low to high volume roads. However, when the N_{design} table was consolidated, no effort was made to verify that the number of gyrations at each compaction level was correct; the number of levels were simply consolidated from the original 28 to 4 levels, as shown below in Table 1.

These four levels were selected so that the mix volumetrics at each compactive effort would be significantly different from adjacent levels. For the purpose of the NCHRP 9-9 study it was assumed that a change in VMA of 1 percent was significant. Hence, the compaction levels were established to provide a difference in VMA between adjacent compaction levels of 1 percent. Initially, the numbers were set at 50, 70, 100, and 130; but after several meetings, external to the project, the numbers in Table 1 were adopted for consideration to be added to AASHTO standards. When the N_{design} table was reduced, no effort was made to verify that the number of gyrations at each compaction level was correct; the levels were simply consolidated. There is a need to verify that the number of gyrations for each traffic level is correct. This is the primary concern of most state departments of transportation (DOTs).

1.2 OBJECTIVES AND SCOPE

The primary objective of the NCHRP 9-9(1) study is to verify that the gyration levels in the N_{design} table are correct and to modify these levels if appropriate. The $N_{initial}$ and $N_{maximum}$ requirements will also be evaluated for the various mixtures selected. This type of project will take several years to get a "final" answer, but this work must begin now and, within approximately two years, approximate answers can be obtained.

Task 1 of the study involved conducting an extensive literature review pertaining to the development and evaluation of the SGC, the use of the SGC for mix analysis, and the in-place densification of HMA pavements over time with respect to traffic. The results of the literature review are presented in this document.

CHAPTER 2 LITERATURE REVIEW

2.1 DEVELOPMENT AND EVALUATION OF THE SUPERPAVE GYRATORY COMPACTION PROCEDURE

The following information is provided pertaining to the development and subsequent evaluation of the Superpave gyratory compaction procedure.

Cominsky, R., Leahy, R. B., 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.

Cominsky et al (*1*) provide the 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 operating protocols very similar to the French (LCPC) gyratory compactor. Summaries of the development of Superpave compaction parameters are provided below.

Gyrations Per Minute (Rotational Speed)

The French gyratory compactor operates at a rotational speed of six revolutions per minute (rpm). SHRP researchers wanted a rotational speed as fast as possible, provided the volumetric properties of mixes were not adversely affected. An experiment was conducted using the RB aggregate and the AAK-1 asphalt binder from the SHRP's Materials Reference Laboratory to determine if the mixture volumetrics (optimum asphalt content, air voids, VMA, and VFA) were affected by rotational speeds of 6, 15, and 30 rpm. Statistical analysis showed that the volumetric properties for the three rotational speeds were not statistically different, and a speed of 30 rpm was selected for Superpave gyratory compactor operation.

Gyratory Compactor Comparison

Next, an experiment was conducted to determine if the specification parameters of gyration angle, 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 GTM. The SHRP gyratory compactor was manufactured by the Rainhart Company. The variables in the experiment consisted of the following:

Aggregate Blends:	Four blends were selected with nominal maximum sizes ranging from 9.5 to 25 mm. Mixes comprised of these blends had been previously designed using the modified Texas gyratory compactor at an angle of gyration of 1.27 degrees. The designed mixes were used for SPS projects in Indiana and Wisconsin.
Specimen Sizes:	Two specimen sizes were evaluated: 150 mm and 100 mm. 100-mm specimens were not possible with the modified Texas gyratory compactor.
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, with the exception of the gyration angle for the GTM. The GTM operated 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 GTM versus the other two gyratory compactors was accomplished with a single mix at three asphalt contents. All specimens were short term aged at 135°C for four hours. Conclusions reached from the experiment were as follows:

1. The modified Texas gyratory compactor and the SHRP gyratory compactor did not compact the specimens the same. This difference was attributable to the difference in the gyration angle of the two compactors. A check of the gyration angle showed that the modified Texas gyratory compactor had an angle of 0.97 degrees, while the SHRP gyratory compactor had angles of 1.14 and 1.30 degrees for 150 mm and 100 mm specimens, respectively.
2. An angle of gyration variation for all compactors of 0.02 degrees resulted in an average air

void variation of 0.22 percent at 100 gyrations. This resulted in an average 0.15 percent change in the determined optimum asphalt content for the 19.0-mm mixture.

3. Specifying the angle of gyration, rotational speed, and vertical pressure alone is not sufficient to produce similar compactors.
4. Based on the limited evaluation, the USACOE gyratory compactor does not produce similar results as the SHRP gyratory compactor. This is attributed to the variable angle of the USACOE gyratory compactor.

Cominsky et al also (1) document a separate study in which the SHRP gyratory compactor was used to design nine SPS-9 projects in the states of Arizona, Indiana, Maryland, and Wisconsin. A total of seven different mixes was designed using the Superpave gyratory compactor. It was determined in the designs that the specified 1.0-degree angle of gyration was not sufficient to achieve the design air void level of 4 percent using the specified N_{design} of 113 gyrations. Therefore, the angle was increased to 1.27 degrees and mix designs performed again. The researchers determined that the asphalt content at a design air void level of 4 percent was suitable (resulted in a lowering of the asphalt content)and that the angle of 1.27 degrees was more appropriate than the 1.0 degree.

The report (1) additionally documents how present gyratory compaction levels of $N_{initial}$ and $N_{maximum}$ were established. Initially, in the Superpave procedure, $N_{initial}$ and $N_{maximum}$ were referred to as N_{89} and N_{98}, respectively. As mentioned previously, the Superpave gyratory compaction procedure was modeled, in part, after the French gyratory compaction protocol. Wherein, N_{89} is set at 10 gyrations; at which the compacted sample density must be less than 89 percent of the maximum theoretical specific gravity. The value of N_{89} does not change based upon the selected level of N_{design}. The SHRP researchers felt that the level of N_{89} or $N_{initial}$ should be a function of the N_{design} level and should increase as the N_{design} level increased to yield a more stable mixture for higher temperatures and traffic levels.

Additionally, a value of the maximum allowable achieved density in the Superpave gyratory compactor was established and is referred to as N_{98} or $N_{maximum}$. The researchers felt that any mix that compacted to greater than 98 percent of the maximum theoretical specific gravity in the laboratory would be prone to excessive densification or rutting in the field.

From the results of the initial N_{design} experiment (SHRP-A001, Task F), the relationship between $N_{initial}$ and $N_{maximum}$ was established. Figure 1 illustrates the procedure used by the researchers for one mix from Arizona. Aggregate recovered from cores was re-mixed with an equivalent asphalt cement to the original asphalt cement and compacted to approximately 275 gyrations in the Superpave gyratory compactor. The densification curve of this mix is referred to as the "as-recovered" curve. The next step was to determine the intersection point of 96 percent G_{mm} and the established N_{design} value for the mix. The "as-recovered" compaction curve was then translated horizontally until it passed though the intersection point. In Figure 1, this shifted curve is referred to as the "estimated design" curve. Finally, lines were drawn vertically from levels of 89 and 98 percent G_{mm} on the "estimated design" curve to the x-axis, which is the number of gyrations. The number of gyrations corresponding to 89 and 98 percent G_{mm} were then referred to as $N_{initial}$ and $N_{maximum}$, respectively. The ratio of the log of $N_{initial}$ and $N_{maximum}$ to the log of N_{design} was then used to determine the relationship between a given N_{design} and the corresponding $N_{initial}$ and $N_{maximum}$ values.

This process was repeated for each of the mixes used in the N_{design} experiment. The researchers found that the average $N_{initial}$ level for the mixes evaluated in the N_{design} experiment was approximately equal to 0.47 * log N_{design}, which then evolved to the currently used Superpave criteria of $N_{initial}$ = 0.45 * log N_{design}. Likewise, the average $N_{maximum}$ level was determined to be approximately 1.15 * log N_{design}, which was later specified as 1.10 * log N_{design}.

With the operational characteristics of the Superpave gyratory compactor established, the next task in the SHRP study, as documented by Cominsky et al (*1*), was to determine if the gyratory compactor could be used to verify or control mix production. More specifically, the study was designed to evaluate the effect on the compaction characteristics in the gyratory compactor resulting from changes in the asphalt content, percent passing the 0.075 mm sieve, percent passing the 2.36 mm sieve, aggregate nominal maximum size, and the percentage of natural and crushed sand. The mix used for the baseline evaluation was a previously designed SPS-9 mix for Interstate 43 in Milwaukee, Wisconsin. This mix was a coarse-graded (below the restricted zone) 12.5-mm nominal maximum size mix. In the procedure the above-mentioned parameters were evaluated at each of three levels, with the design mix parameters being the mid range or medium level, as shown in the testing plan in Table 2. A total of 243 samples would comprise the total factorial experiment. However, only 33 samples were prepared and evaluated in the study. Compaction of all samples in the study was completed using a gyration angle of 1.14 degrees, a vertical pressure of 600 kPa, and a rotational speed of 30 rpm. The angle of 1.14 degrees was the angle measured during the previous study comparing the modified Texas gyratory compactor and the SHRP gyratory compactor manufactured by the Rainhart Company.

After compaction, response variables of C_{10} (%G_{mm} at $N_{initial}$), C_{230} (%G_{mm} at $N_{maximum}$), K (gyratory compaction slope), air voids, VMA, and VFA were calculated and evaluated. The results indicate that all volumetric properties (air voids, VMA, VFA) were significantly influenced by changes in asphalt content, percent passing the 0.075 mm sieve, and the percent natural sand. Less significant changes were shown in the percent passing the 2.36-mm sieve. Further, the nominal maximum aggregate size did not significantly change volumetric properties (air voids, VMA, and VFA) of the mixes. The effect of the input variables on the C_{10} (%G_{mm} at $N_{initial}$), C_{230} (%G_{mm} at $N_{maximum}$), K (compaction slope) are shown in Table 3. It is seen that asphalt content and the percent passing the 0.075 mm sieve have the greatest effect (causing all three parameters to increase) on the gyratory compaction response variables, with the percent passing the 2.36 mm sieve and the percent natural sand having a lesser effect. As was the case with the volumetric properties, the nominal maximum aggregate size did not have a significant effect on the compaction response variables.

Cominsky, R., Huber, G. A., Kennedy, T. W. and Anderson, R. M. "The Superpave Mix Design Manual for New Construction and Overlays." Strategic Highway Research Program, Report SHRP A-407, 1994.

In another report by Cominsky et al (2), the detailed operational parameters of the Superpave Gyratory Compactor are provided. In the Superpave gyratory compaction procedure, the density at three specific points, $N_{initial}$, N_{design}, and $N_{maximum}$, is determined as the sample is being compacted. The N_{design} level is dependent upon the design traffic level (ESALs) and the design seven day maximum air temperature for the project. The values of $N_{initial}$ and $N_{maximum}$ are then determined depending upon the chosen N_{design} level through the following equations 1 and 2.

$$\log N_{initial} = 0.45 \log N_{design} \quad \textit{Equation 1}$$

$$\log N_{maximum} = 1.10 \log N_{design} \quad \textit{Equation 2}$$

Values of $N_{initial}$, N_{design}, and $N_{maximum}$ for each traffic level and temperature are provided in Table 4. Superpave specifies that the design or optimum asphalt content be selected to provide 96 percent G_{mm} (4 percent air voids) at the given N_{design} level. Furthermore, the designed mix must have densities which are less than 98 percent G_{mm} (2 percent air voids) and 89 percent G_{mm} (11 percent air voids) at $N_{maximum}$ and $N_{initial}$, respectively. A typical densification slope that is obtained from the Superpave gyratory compaction procedure is shown in Figure 2. From Figure 2, it can be seen that the densification slope of a gyratory compacted sample is approximately linear when plotted on a semi-log scale.

In the Superpave procedure, all specimens are compacted to $N_{maximum}$ and their densities at N_{design} and $N_{initial}$ determined through a back-calculation procedure. The procedure consists of first determining the uncorrected density of the sample at a given gyration level as follows:

$$C_{ux} = \left[(M_{mix} / V_{mix}) / G_{mm}\right] * 100 \quad \textit{Equation 3}$$

where,

C_{ux} = the uncorrected density of the sample at a given gyration level (x), (g/cm^3),

M_{mix} = the mass of the mix being compacted (g),

V_{mix} = the volume of the mix being compacted at (x) gyrations (cm^3).

This calculated uncorrected density can then be used to calculate the corrected specimen density as follows in Equation 4. The sample density must be corrected because the calculated volume at "x" gyrations based upon the mold diameter and sample height is not the true volume of the sample. This is due to errors resulting from surface irregularities along the sides and ends of the sample. The true volume is usually slightly less than the calculated volume.

$$C_x = \frac{C_{ux} G_{mb} V_{mm}}{M_{mix}}$$ *Equation 4*

where,

C_x = the corrected density of the sample at a given gyration level (x), (g/cm^3),

G_{mb} = the measured bulk specific gravity of the sample at $N_{maximum}$,

V_{mm} = the volume of the mix at $N_{maximum}$ (cm^3),

M_{mix} = the mass of the mix at (x) gyrations (g)

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.

McGennis et al (3) report the results of the Superpave gyratory compactor study to determine the effect of various compaction parameters on the mixture volumetric properties. Parameters included mold diameter, short-term aging time, and compaction temperature. Additionally, the study was performed to determine if changing any of the parameters affected the AASHTO T-283 moisture susceptibility results. In order to determine the variability of mixes 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 blends were used. The gradations, seven total, ranged from gap-graded to finer gradations, with all the gradations being below the restricted zone. The optimum asphalt content for each of these mixes was established to provide 4 percent air voids at a 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 mixes. Next, specimens were compacted, at the optimum asphalt content, in 150 mm and 100 mm gyratory molds. For the experiment the volumetric properties of the mixes were compared at gyration levels of 10, 100, 150, and 250 gyrations. Specimen bulk gravities from the two mold sizes were then compared. Two sample t-tests were performed at a level of significance of 5 percent and indicated that for 56 percent of the comparisons there was a significant difference between the 150-mm and the 100-mm diameter specimens. Also, within the 12.5-mm nominal maximum size, mold size affected the densification of coarser mixes more often than it affected that of the mixes that were slightly finer.

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 a 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 aging, the specimens were placed in an oven at the specified compaction temperatures. The compaction temperatures used were 120°C, 135°C, 150°C, 165°C, and 180°C. 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) mixes; however, the volumetric properties of the modified binder (PG 76-28) were significantly affected by the variation of compaction temperatures.

Moisture Susceptibility

An experiment 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) on moisture susceptibility. The mixture evaluated was identified by the Kentucky Department of Highways as being stripping susceptible. Results indicated that the SGC yielded higher TSR values than the Marshall hammer; however, the SGC did correctly identify the Kentucky mixture as moisture susceptible, based upon a minimum tensile strength ratio (TSR) of 0.80. TSR values 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

To evaluate the effect of varying short term aging times a mixture with the same asphalt binder content, and the same aggregate gradation was aged at 135°C for periods of 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 determined. The bulk specific gravities were then compared to the theoretical maximum specific gravities that were determined from an average of two specimens at each aging temperature. Results indicate specimen volumetric properties were affected by aging time. The general trend was as aging time increased, the compacted bulk specific gravities decreased and the theoretical maximum specific gravities increased.

Gyratory Compactor Comparison

The testing for comparing of the gyratory compactors consisted of preparing specimens at the design 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 SGC. Specimens were prepared to determine differences in the percent G_{mm} at $N_{initial}$ (10 gyrations), at N_{design} (100 gyrations), and at $N_{maximum}$ (152 gyrations). The compaction slopes of the different mixes were also analyzed for differences. Two sample t-tests, at a level of significance of 5 percent, were used to compare the bulk specific gravities from the various compactors and indicated that there were significant differences between the four gyratory compactors. The modified Texas gyratory and the Pine SGC produced mixes with lower air voids and, therefore, lower optimum asphalt contents than did the Rainhart SGC and the Troxler SGC. In addition, the modified Texas and the Pine SGC yielded flatter compaction slopes than did the Rainhart and the Troxler SGC.

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

McGennis et al (*4*) discuss 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" (*5*). The main objectives of the experiment were 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, with each factor being evaluated at a high and low level. The factors and their values are provided in Table 5.

Gyration Angle

The current AASHTO TP-4 specified gyration angle is 1.25 ± 0.02 degrees. Low and high levels for analysis were originally planned to be 1.23 to 1.25 degrees, and 1.25 to 1.27 degrees, respectively. 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 5 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 compaction 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 thrusts 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 mixes be compacted within a temperature range resulting in 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 too restrictive for the experiment and a tolerance of ±5 mm was selected. Therefore, levels of specimen height, after compaction to $N_{maximum}$, 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)" (*6*) require sample aging for four hours at 135°C. After the 4 hours are complete, each sample is 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 4-hour short-term aging period. Therefore, the two levels of aging time selected were (1) to place the mixture in the aging oven for 4 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 SGC and the Pine SGC were utilized. A total of six laboratories participated in the experiment. Three laboratories used the Troxler SGC and three used the Pine SGC. However, one of the Troxler laboratories was unable to complete the study and the experiment continued with five labs.

The mixes 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 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, the transfer bowl method 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. (The tolerance was later changed to ±5 mm).
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 4 hour short term aging period.

Vavrik, W. R., and Carpenter, S. H., "Calculating Voids at a Specified Number of Gyrations in the SUPERPAVE Gyratory Compactor." Transportation Research Record 1630, TRB, National Research Council, Washington, D.C. 117-125.

Vavrik and Carpenter (7) conducted a study to determine the cause of 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 $N_{maximum}$ were determined. The Superpave system uses a back-calculation procedure in which specimen 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. The procedure consisted of compacting one specimen to N_{design} and one to $N_{maximum}$. The densities of the specimens compacted to N_{design} and densities back-calculated from $N_{maximum}$. The results showed differences in density between 0.5 and 1.5 percent.

Due to these differences, 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 over compaction of their designed mixes. 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" is a modification of the now specified $N_{maximum}$

gyration level. 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 mixes used by the Illinois Department of Transportation on Superpave demonstration projects in 1996 were evaluated. These mixes varied in gradation, size, aggregate type, design compactive effort, and asphalt binder type (polymer and unmodified). The results of the evaluation indicated that 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.

Mallick, R. B, Buchanan, M. S., Brown E. R. and Huner, M. H., "Evaluation of Superpave Gyratory Compaction of Hot Mix Asphalt." Transportation Research Record 1630, TRB, National Research Council, Washington, D.C., (1998), 111-119.

Mallick et al (8) compared the correction factors obtained at different gyration levels during the compaction. 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 mixes. Mixes were prepared at their respective optimum asphalt contents and compacted in the Pine SGC 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 compaction, bulk specific gravities and correction factors were determined. Next, separate 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 back-calculated using correction factors from the highest gyration levels and compared with directly measured densities. The results showed that correction factors were not constant at different gyration levels for the mixes evaluated. Relationships between errors in voids and correction factors versus gyrations are shown in Figures 3 and 4, respectively. As expected, the coarser mixture (SMA) exhibited the greatest difference between the back-calculated and the actual specimen densities due to the open surface texture nature of the mix. 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. This is attributable to the increased amount of surface irregularities of the sample at the lower gyration levels relative to high gyration levels. The recommendation from the study was to compact specimens to N_{design} in the volumetric mix design procedure. This would ensure that the true specimen density is obtained at the design level of gyrations.

Forstie, D. A. and Corum, D. K., "Determination of Key Gyratory Compaction Points for Superpave Mix Design in Arizona." ASTM Special Technical Publication, Volume 1322, September 1997. ASTM, Philadelphia, PA., pp. 201-209.

Forstie and Corum (9) present a study conducted by the Arizona Department of Transportation to evaluate the level of Superpave laboratory compaction necessary to equal the in-place field density after various levels of traffic. The basic premise of the research was to determine if N_{design} levels were appropriate for Arizona interstate highways. This was important to the researchers because of the following reasons:

1. The angle of gyration used by SHRP researchers to develop the current levels of N_{design} was 1.0 degrees, while the angle currently specified in AASHTO TP-4 is 1.25 degrees. A gyration angle of 1.30 degrees was used unknowingly by SHRP researchers for a portion of the N_{design} study due to a manufacturing error.
2. The N_{design} experiment was conducted using 100 mm diameter specimens, not the currently used 150 mm specimens.
3. The mixes used in the N_{design} experiment were predominately fine-graded mixes, not the coarse-graded mixes, which are most commonly used today.
4. Only two cores per project location were obtained for testing and evaluation in the original N_{design} experiment. More specimens may have provided a greater confidence in the field

density.

Complete results from seven projects on Interstate 10 are presented. Project testing consisted on obtaining field cores from and between the wheel paths. Gradation, bulk specific gravity, asphalt content, and theoretical maximum specific gravity were determined for the cores. Extracted aggregate from each project was then recombined with an equivalent asphalt cement and compacted in the Troxler SGC to determine its volumetric properties at N_{design}. The N_{design} level of gyrations was determined from the project traffic and temperature. All of the projects evaluated were from a hot climate location and ranged in age from 5 to 8 years and had N_{design} values ranging from 113 to 135 gyrations. Statistical analysis (t-tests at a level of significance of 5 percent) indicated that average bulk specific gravities from the Superpave gyratory compactor were significantly higher (2.355 to 2.318) than the field cores.

Based on the results of the study it was concluded that the current N_{design} compaction levels should be revised in magnitude to account for the 1.0 to 1.25 gyration angle change that occurred during the original SHRP research. Mixes designed at the original N_{design} levels and a gyration angle of 1.25 degrees will likely have higher laboratory densities (lower optimum asphalt content) than mixes designed using a gyration angle of 1.0 degrees, which was the angle used to establish the original N_{design} levels. This over compaction could lead to compaction problems during laydown and also a resistance to traffic densification down to the designed 4 percent air void level.

2.2 USE OF THE SUPERPAVE GYRATORY COMPACTOR FOR MIX ANALYSIS

The following portion of the literature review is a summary of research conducted using the Superpave gyratory compactor for the mix analysis and control of various HMA mix types.

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.

Anderson et al (*10*) provide results from a case study in which the SGC 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 asphalt content.

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

The 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.

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.

A report by Harman et al (*11*) summarized the FHWA's effort to determine the effectiveness of the SGC for field management of the construction of HMA. In a side study of the project, the Marshall hammer was compared to the SGC for possible use as a supplement for field control. The results indicated that the SGC can be used as an effective tool for the field verification of laboratory designed HMA mixes. 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 Marshall hammers should not be used for field quality control of HMA designed using the Superpave system.

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.

A research project by Hafez and Witczak (*12*) consisted of performing designs for 20 different mixes using both the Marshall procedure and the Superpave gyratory compactor Level I (Volumetric) procedure. The mixes were classified into five groups as follows: conventional mixes, wet process asphalt rubber (manufacturer preblended), dry process asphalt rubber, polymer modified, and wet process asphalt rubber (plant blended). All mixes had the same aggregate type, source and gradation (Maryland State Highway Administration-dense aggregate gradation with nominal maximum size of 12.5 mm). These two mixes were Plus Ride No. 12 and No. 16 open-graded mixes with nominal maximum sizes of 12.5 mm and 19.0 mm, respectively.

Optimum asphalt contents for all mixes in the study were determined by the Marshall 75 blow and Superpave Level I (Volumetric) procedures. The Marshall procedure consisted of preparing three replicates at 1.0 percent asphalt content increments in order to cover an air voids 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 ≤34°C, 37-39°C, and 43-44°C. The N_{design} values corresponding to these parameters are 67, 96, and 119 gyrations, respectively. In addition to determining the optimum asphalt content at 4.0 percent air voids, the asphalt content was selected to provide both 3.0 and 5.0 percent air voids for comparison to the Marshall procedure.

Conclusions drawn from the study were as follows:

- The Superpave gyratory Level I (Volumetric) mix design procedure cannot be used to design dry-process asphalt rubber mixes. Specimens in this category experienced 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 mixes evaluated can be accurately designed and evaluated using the Superpave gyratory Level I (Volumetric) procedure.
- As the compactive effort, N_{design}, for the SGC is decreased from 119 to 67 gyrations, an increase of approximately 1.0 percent asphalt content is experienced for all mixes evaluated.
- For a given level of compaction with the Superpave gyratory compactor there were no consistent trends between the density obtained using the Superpave procedure and the Marshall procedure.

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.

Sousa et al (*13*) describe a study conducted by the Arizona Department of Transportation to evaluate mixes designed using the Marshall, Superpave Level I, and a performance based procedure developed under SHRP-A003A. The mixture was placed in two 1-mile test sections 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 the fine crushed gravel. All mixes were designed with 1 percent Portland cement to reduce moisture susceptibility. The mix design gradation conformed to a fine 19.0 nominal maximum size Superpave gradation, however, during production the aggregate source (same material type) was changed. This resulted in the field gradation being coarser and passing through the Superpave restricted zone.

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 3,000 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 mix 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 mix deficiencies, 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} at $N_{initial}$ requirements.

Field cores from this project were also evaluated in the Hamburg wheel tracking device at 55°C. Prediction of performance indicated a "good" pavement that would last approximately 10 to 15 years.

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 usually occur during 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 UC-Berkeley rolling wheel compactor, the California kneading compactor, the Texas gyratory compactor, the Marshall hammer, the SHRP Rainhart gyratory compactor (Asphalt Institute), and the SHRP gyratory compactor (FHWA field trailer). The results indicated that the rolling wheel compactor produced specimens that best correlated against field cores based upon their permanent deformation resistance in the repeated simple shear at constant height test (RSST-CH).

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

D'Angelo et al (14) provide the results of a study in which five different asphalt mixes, produced at five different asphalt plants, were compared using the Superpave Level I and the Marshall compaction procedures. Two of the mixes were designed using the SGC at N_{design} levels of 86 and 100 gyrations. These two mixes were evaluated with the Marshall hammer using 112 blows (6 inch sample) and 50 blows, respectively. Three of the mixes were designed using the Marshall hammer with 112 (6 inch sample), 50, and 75 blows. The SGC was used to evaluate these mixes at N_{design} levels of 100, 126, and 109 gyrations, respectively. Samples of the five mixes were obtained and compacted in both the SGC and the Marshall hammer to determine the quality control ability of the SGC and Marshall hammer. The results of the analysis indicate that samples compacted with the SGC had slightly less variability in air voids than did the Marshall samples. Based on air voids alone, the SGC 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 SGC samples had lower VMA than Marshall samples. For three of the five mixes, the VMA of the gyratory and Marshall compacted samples tended to decrease with an increase in asphalt content. The other two mixes showed that as the asphalt content increased, the VMA decreased for the SGC 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 SGC and the Marshall hammer, respectively. The general trend of lower VMA with the SGC indicates that the compaction effort obtained with the SGC is greater than with the Marshall hammer. The overall conclusion of the study was that the SGC was better able to track plant production variability than the Marshall hammer.

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." Journal of the Association of Asphalt Paving Technologists, Volume 67, 1998, pp. 189-232.

Bahia et al (15) conducted a study to evaluate a method to utilize the gyratory compaction data to predict the densification characteristics under construction and traffic. More specifically, the objective was to evaluate the effect of aggregate gradation and fine aggregate angularity on the densification characteristics of HMA. The following variables were controlled in the study:

1. Aggregate: All aggregates 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 with each aggregate blend at 5 percent asphalt content was compacted to determine the densification variability.
5. Compactive Effort: The HV mixes were compacted to $N_{maximum}$ = 150 gyrations and the MV compacted to $N_{maximum}$ = 129 gyrations. Two samples were compacted for each blend; one to N_{design} and the other to $N_{maximum}$.
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 mixes were compacted and their compaction data 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 mixes showed the following:

1. Mixes with higher $\%G_{mm}$ at $N_{initial}$ do not necessarily show higher $\%G_{mm}$ at $N_{maximum}$. In fact, the opposite seems to hold true.
2. Values of $\%G_{mm}$ at $N_{initial}$ were very close to greater than the maximum limit of 89 percent of G_{mm} for blends above and through the restricted zone for both the HV and the MV mixes. Percent G_{mm} at $N_{initial}$ for aggregate blends below the restricted zone are well below the 89 percent maximum limit.
3. The $\%G_{mm}$ at $N_{maximum}$ was close to the limit of 98 percent for all aggregate blends. The % G_{mm} for coarser mixes are closer to the limit than the % G_{mm} for finer mixes. This indicates that coarser mixes would be more susceptible to densification beyond the 2 percent air void limit.
4. Densification slopes were between 6.2 and 6.7 for the HV mixes above the restricted zone and between 8.1 and 9.8 for HV mixes below the restricted zone.

Anderson, M. R., Cominsky, R. J. and Killingsworth, B. M., "Sensitivity of Superpave Mixture Tests to Changes in Mixture Components." Journal of the Association of Asphalt Paving Technologists, Volume 67, 1998, pp. 153-188.

Anderson et al (*16*) evaluated the effects of component proportions and properties on mixture properties. To complete the study, the SGC was used to evaluate volumetric changes and the Superpave shear tester (SST) for the mechanical properties. The volumetric properties determined from the SGC 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 percentage 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 mixes 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 caused significant changes in 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.

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

Kandhal and Mallick (*17*) evaluated the Asphalt Pavement Analyzer (APA) wheel tracking device predicting the rutting potential of laboratory designed Superpave HMA. The sensitivity of the APA, as indicated by rut depths and rut slopes, to changes in the aggregate type and gradation, and the performance grade (PG) of the asphalt binder was obtained in the study. Two mix types (wearing and binder course), three aggregates (granite, limestone, and gravel), three gradations (above, through, and below the restricted zone), and two asphalt binders (PG 64-

22 and PG 58-22) were evaluated. The limestone and the granite aggregate blends were comprised of 100 percent crushed material, with fine aggregate angularity (FAA) values of 49.3 and 45.8 percent, respectively. The crushed gravel had approximately 90 percent two crushed faces and a FAA of 46.0 percent. Among the items addressed in the study was whether a correlation existed between the density at $N_{initial}$ and $N_{maximum}$ and the APA rut depths, and also whether a correlation existed between the gyratory compaction slope and the APA rut depths.

None of the mixes evaluated had densities at $N_{maximum}$ greater than 98 percent G_{mm}, but 44 percent of the mixes had densities greater than 89 percent G_{mm} at $N_{initial}$. Mixes that failed the $N_{initial}$ requirement of 89 percent G_{mm} did not have greater rut depths than mixes which met the 89 percent G_{mm} at $N_{initial}$ requirement. Although none of the mixes failed that $N_{maximum}$ density requirement, the data indicated mixes which were within 0.1 to 0.2 percent of 98 percent G_{mm} slightly higher rut depth. Additionally, the results indicated that there was no correlation between APA rut depths and the gyratory compaction slope calculated between $N_{initial}$ and N_{design}.

2.3 IN-PLACE DENSIFICATION WITH RESPECT TO TRAFFIC

The following is a review of literature pertaining to the relationship between applied traffic and in-place densification of HMA pavements.

Dillard, J. H. "Comparison of Density of Marshall Specimens and Pavement Cores" Proceedings of the Association of Asphalt Paving Technologists, Volume 24, Minneapolis, MN, 1955, pp. 178 - 232.

Dillard (*18*) presents the findings of research undertaken to determine if the Marshall 50 blow design method was capable of providing the ultimate density of pavements in Virginia. Samples were taken from 26 construction projects in which the traffic varied from 1,166 to 13,808 vehicles per day. Sand asphalts and conventional dense-graded mixes were the two mix types used on the projects. Samples of produced mix were molded in the Marshall procedure to a range of blow counts to determine the count that matched the ultimate density of the pavement.

Cores were obtained from the outside wheel path of each of the sections and compared to the Marshall densities. For the majority of the pavements, it appeared that the Marshall 50 blow procedure yielded significantly higher densities than the in-place densities after 16 months. For the sand mixes, a good relationship between the in-place densities after 16 months and the 30 blow Marshall densities was achieved.

The data indicated that the amount of traffic did not have a significant effect on the ultimate density achieved. The author states that two pavements with different traffic levels may reach the same ultimate density, but will require different amounts of time.

Field, F. "Correlation of Laboratory Compaction with Field Compaction of Asphaltic Concrete Pavements." Proceedings of the Third Annual Conference of the Canadian Technical Asphalt Association. Volume III, 1958. pp. 9 - 46.

Field (*19*) explains the 1958 research efforts of the Materials and Research Section of the Department of Highways of Ontario conducted to answer the following questions pertaining to the field densification of Marshall 75 blow mixes.

1. Does a pavement, in particular high strength mixes, densify to the design laboratory density?
2. Do pavements densify beyond the design laboratory density?
3. How does traffic affect the pavement density over a few years?

To answer the questions, 31 pavements in southern Ontario were evaluated in the study. These pavements were broken down into three groups as provided below: (All pavements were typically dense-graded).

Group I. 11 pavements of medium to high traffic. Fine aggregate for the natural sand.

Group II. 10 pavements of heavy traffic. Fine aggregate a blend of screenings and natural sand. Most pavements in Groups I and II were evaluated after five months, 17 months, and 29 months.

Group III (a). Four pavements of heavy traffic. Minimum Marshall stability of 1500 lbs. Constructed before September. Pavements were evaluated after 3 months.

Group III (b). Six pavements of heavy traffic. Minimum Marshall stability of 1500 lbs. Two were constructed in mid-Summer, one in September, and three in October. Pavements were evaluated after two months.

The results of the evaluation for the mixes are summarized below:

Group I. Seven of the 11 mixes had density greater than 97 percent of laboratory after five months. (Four were placed in mid summer and three in the fall). After one year, these seven mixes were at or slightly greater than the laboratory density. Three of the 11 projects had less than 95 percent of lab density and one was between 96 and 97 percent. Only one of these four mixes had a density close to laboratory density after two years. (These last four were all constructed in late fall)

Group II. After five months, six of the 10 pavements had densities that were approximately 98 percent of lab density. (These six pavements were constructed in mid-Summer). The other four pavements, constructed in October and November, had average densities that were approximately 95 and 97 percent of the lab density after 5 and 17 months, respectively.

Group III(a). After three months, the density of three of the four pavements constructed during the mid-Summer was approximately 98 percent of lab density, while the density of the remaining pavement was 95 percent of lab.

Group III(b). After two months, the density of the six pavements was as follows: Two pavements constructed in mid-Summer had densities of 96.8 and 97.6 percent of lab. One pavement constructed in September had a density of 94.8 of lab density. Three pavements constructed in October had densities of 95.1, 94.6 and 94.8 of lab density.

The results emphasize the importance of obtaining adequate density at construction, especially when paving late in the season. The majority of mixes constructed during mid-Summer were close to the lab density at the times of evaluation. Additionally, it seems that further compaction from traffic can be slow resulting in the pavement experiencing durability problems before the design lab density is achieved.

Campen, W. H., Smith, J. R., Erickson, L. G., and Mertz, L. R. The Effect of Traffic on the Density of Bituminous Paving Mixtures. Proceedings of the Association of Asphalt Paving Technologists. Minneapolis, MN. 1960. pp. 378-397.

Campen et al (*20*) report on the densification over time of 18 mixes placed between 1955 and 1959 in the city of Omaha, Nebraska. The mixes were all surface mixes and varied in layer thickness from 19 mm to 50 mm. Traffic on the various streets ranged from an average daily traffic of 6,000 to 35,000 vehicles. Traffic consisted of both passenger cars and trucks, but no breakdown of either was reported. Aggregates used in the mixes throughout the period consisted of crushed limestone, crusher run gravel, and coarse and fine natural sand. All gradations were dense to fine-graded with between 56 and 76 percent passing the 4.75 mm sieve. The 50 blow Marshall design procedure was used for each mix and resulted in optimum asphalt contents from 4.5 to 5.25 percent. The asphalt binder ranged from a 60/70 to an 85/100 pen grade.

In July of 1960, samples were cut from the various pavements to determine the densification over time, with some having been in service for 5 years and some for only 1 year. Field inspections indicated that only mixes placed in 1955 showed any evidence of rutting or shoving; however, mixes placed in 1956 through 1959 showed more evidence of durability problems. Bulk specific gravity of the obtained samples were compared to the lab bulk specific gravity to determine a relative density. The following relative density results were found for the 18 projects.

- Three between 100.1 to 100.5 percent.
- Ten between 99 and 100.0 percent.
- Three between 98 and 99 percent.
- Two between 96.6 and 98 percent.

The relative densities indicated that the applied traffic generally did not densify the pavement past the density achieved during the 50 blow Marshall design procedure. Other conclusions were that the ultimate field density is usually attained in a few months during hot weather and the initial field density does not control the ultimate density in the pavement.

The author suggested that the laboratory design compactive effort (from 50 blows/side) should possibly be reduced for light/medium trafficked pavements to allow for more asphalt in the mixes which should provide increased durability.

Graham, M. D., W. C. Burnett, J. T. Thomas, and W. C. Dixon. "Pavement Density - What Influences It." Proceedings of the Association of Asphalt Paving Technologists, Volume 34, Minneapolis, MN, 1965, pp. 286 - 308.

Graham et al (21) report on research conducted by the New York Department of Public Works to determine the influence of mix composition, thickness, temperature, roller passes, and applied traffic on the in-place density of 47 test sections, located on 12 construction projects.

All mixes were conventional dense-graded mixes and were designed using 50 blow Marshall procedures. Immediately after construction a series of cores was obtained from each of the sections to determine the in-place density and the possible variation of density in the transverse and longitudinal directions. The density of each core was then related to the average Marshall 50 blow density that was achieved during construction. Approximately 68 percent of the sections had densities, which exceeded the Marshall density after construction, with the average density of the cores from the test sections being 95.6 percent.

The data indicated a statistical difference (range of 1.6 percent) in the in-place density across the travel lane (inner wheel path, between the wheel paths, and outer wheel path), with the between the wheel path having the highest density and the outer wheel path having the lowest. There was no statistical difference in the density in the longitudinal direction. The core data after one and two years of service, shown in Figure 5, indicates that the pavements densified significantly during the first year, but to a lesser degree in the second year. After one and two years of traffic, approximately 92 and 96 percent, respectively, of the sections had densities greater than the 50 blow Marshall density.

Woodward, E. J., and J. L. Vicelja. "Aviation Boulevard - Evaluation of Materials,

Equipment, and Construction Procedures." Proceedings of the Association of Asphalt Paving Technologists, Volume 35, Minneapolis, MN, 1965, pp. 215 - 233.

Woodward and Vicelja (22) discuss the construction and testing of Aviation Boulevard in Los Angeles. The boulevard was paved using three mix types, with the dense-graded surface mix being a 0.5 inch maximum size aggregate mix placed 2 inches thick. A variety of testing, including field coring, was conducted on the project. Approximately 169 cores were obtained from the time of construction to a period of 180 days after applied traffic and showed the asphalt mix was increasing in density with age, as expected. The largest increase in density occurred during the first 30 days (3 lbs/ft^3), approximately 1 to 1.5 lbs/ft^3 during the next 60 days, and 1 to 1.5 lbs/f^{t3} from 90 to 180 days. The increase in density appeared to be consistent across the travel lanes, without any appreciable increase in density in the wheel paths compared to other locations.

Serafin, P. J., L. L. Kole, and A. P. Chritz. "Michigan Bituminous Experimental Road: Final Report." Proceedings of the Association of Asphalt Paving Technologists, Volume 36, Minneapolis, MN, 1967, pp. 582 -614.

Serafin et al (23) discuss research work conducted by the Michigan Department of State Highways to determine the performance of various HMA test sections comprised of differing asphalt cements. Twenty-four sections were evaluated, with each test section being approximately 1200 feet in length.

The aggregate type and blend gradation for the test sections were held constant with the asphalt cement type and content being varied. All the test sections were constructed in the summer months of 1954. The mixes were fine-graded with a maximum aggregate size of 5/8 inches and were placed at a rate of 130 lb./sq. yd.

In November of 1954, a coring program was started and continued for approximately 12 years with the purpose of determining the in-place density and other mix properties. Good relationships between the core bulk specific gravity (in-place density) and time (traffic) over the

12 year period were recorded for the vast majority of the twenty- four test sections. An example of the relationship is shown in Figure 6. From Figure 6, it is evident that the increase in bulk specific gravity seems to level off after approximately 3 to 4 years of service. Traffic levels and percent commercial vehicles on the test sections remained fairly constant over the initial 7 years, but dropped approximately 30 percent during the last 5 years of evaluation.

Galloway, B. M., "Laboratory and Field Densities of Hot-Mix Asphaltic Concrete in Texas." Highway Research Board Bullentin 251: Asphaltic Concrete Construction-Field and Laboratory Studies. National Academy of Sciences, Washington, D. C., pp. 12 - 17.

Galloway (*24*) conducted research on 12 field test sections for the purpose of comparing laboratory and field densities. The test sections were comprised of a variety of aggregates (gravel, limestone, and basalt) and were compacted using many different roller types and weights. Lift thicknesses ranged from 7/8 to 2 inches. Cores obtained from each of the sections nine months after construction showed that the density of five of the sections exceeded the laboratory density by 1 to 3 percent. The average in-place density of the sections was determined to be 94.6 with a maximum density of 97.2 being observed. Based on the data, the author concluded that the Texas Highway Department procedure for the laboratory design of HMA mixes does not, in all cases, produce the ultimate density for mixes.

Bright, R., B. Steed, Steele, J., and A. Justice. "The Effect of Viscosity of Asphalt on Properties of Bituminous Wearing Surface Mixtures." Proceedings of the Association of Asphalt Paving Technologists, Volume 36, Minneapolis, MN, 1967, pp. 582 -614.

Bright et al (*25*) report the results of an experiment in which 24 field test sections were placed near Raleigh, North Carolina, on Highway 64 to determine the effect of varying asphalt cement viscosities on the performance of the compacted mixes. All of the sections were 1 inch thick, with half being comprised of a crushed gravel and half with a crushed granite aggregate. An 85/100 pen grade asphalt cement was used for all the mixes. The temperature of the mixes was varied (225, 250, 287, 345°F) to provide a spread of mix viscosities from 40 to 900 Saybolt Furol Seconds for placement. All sections were produced using the same plant and constructed using the same equipment and procedures.

Cores were obtained from the test sections periodically to determine the in-place density and other mix properties. The change in the mixture bulk specific gravity in relation to the test section age is shown in Figure 7. It appears that generally, the mixes, with the exception of the 225°F, seemed to converge to the same bulk specific gravity after 20 months, regardless of the initial compaction level.

Palmer, R. K., and J. T. Thomas. "Pavement Density - How It Changes." Proceedings of the Association of Asphalt Paving Technologists, Volume 37, Minneapolis, MN, 1968, pp. 542 - 571.

Palmer and Thomas (26) provide the results of the continuation of research conducted by the New York State Department of Transportation, reported by Graham et al (21), in 1965 is described by the authors. The research involved the evaluation of the in-place density of 47 test sections over the first 5 years of service. The original project work had been conducted after two years of service.

It was observed from the data that the first year density increase averaged about half the total 5 year increase in density. The average gain in the density was 3.5 percent for the wheel paths and 2.5 percent between the wheel paths. High volume pavements were seen to have a density increase approximately twice that of the low/medium volume pavements.

Rutting was not a problem on any of the sections after 5 years of service. One of the interesting conclusions was that there did not appear to be a good correlation between the applied traffic and the increase in density.

Epps, J. A., B. M. Gallaway, and W. W. Scott, Jr., "Long Term Compaction of Asphalt Concrete Pavements." Highway Research Record 313, National Research Council, Washington, D. C., 1970, pp. 79 - 91.

Epps et al (27) evaluated 15 field test sections constructed in Texas to determine, in part, the relationship between traffic and the in-place air voids over a period of two years. The mixes were comprised of gravels, slag, and limestone aggregates with AC-10, AC-20, and 85-100 pen asphalt cements. Eleven of the 15 sections used the AC-20 asphalt. Each section was further divided into three sub-sections in which the compactive effort was varied as the normal number of roller passes, half the number of roller passes, and twice the number of roller passes.

After construction, four inch cores were taken from each of the sections at periods of 1 day, 1 week, 1 month, 4 months, 1 year, and 2 years to determine the mix properties and in-place density. The effect of traffic on the in-place pavement air voids over the two year period is illustrated in Figure 8. The amount of initial compaction did not seem to significantly affect the amount of pavement densification, as illustrated in Figures 9 and 10. The majority of the field pavements compacted to densities that were within 1 to 2 percent of each other after the two-year period, with a decrease of 4 to 6 percent occurring in the pavements. It was concluded from the project that approximately 80 percent of the average total 2-year densification was obtained during the first year.

Paterson, W. D. O., A. Williman, and J. S. Pollard, "Traffic Compaction of Asphalt Surfacings.", National Institute for Road Research, South Africa, 1974.

Paterson et al (28) evaluated 20 test sections, comprised of varying combinations of asphalt type, asphalt content, maximum stone size, lift thickness, tire pressure, and constructed density, on an accelerated test track facility in New Zealand. The purpose was to determine the effect of the factors on the stable state density achieved in the resulting mixes. The mix used was a continuously graded crushed aggregate material blend designed using the 75 blow Marshall procedure.

After all the sections had been constructed, a testing vehicle with a 20 kN wheel load made 700 vehicle passes per hour for up to approximately 30,000 total passes. The temperatures at the mid-point in the lift were held constant at 25°C and also at 40°C, to determine the effect of temperature.

Each of the sections was loaded by four combinations of tire pressure and temperature and their stable state density determined through core testing.

The results of the study indicated that the following:

1. The temperature greatly influenced the increase in density under traffic, while tire pressure influenced the density to a lesser degree.
2. Compaction under traffic could increase the density by approximately six percent.
3. The influence of the construction density was dependent upon the test temperature. At 25°C, the construction density influenced the stable state density, but not at 40°C.
4. Over compaction tended to result in thick layers while under compaction typically happened in thin layers.
5. The majority of the mixes had densities after testing which ranged from 0.5 to 1 percent greater than the 75 blow Marshall design densities.

Gichaga, F. J., "Behaviour of Flexible Road Pavements Under Tropical Climates." Proceedings of the 5th International Conference on the Structural Design of Asphalt Pavements, Delft University of Technology, Volume 1, 1982, pp. 221 - 239.

A report by Gichaga et al (29) discusses the performance of six test mixes placed in Kenya in 1979. The sections were place to evaluate a new structural design procedure developed by the Roads Department of Kenya. Each of the sections was evaluated periodically throughout a period of two years to determine the degree and magnitude of distress present. The relationship between traffic and pavement densification for two of the six sections is shown in Figure 11. Both sections carried approximately 1,200 commercial trucks per day. The asphalt mixes for both the sections were designed using Marshall 50 blow procedures at a design air void level of 5.4 percent. Figure 11 illustrates that the densification was substantial during the first five months after construction but then leveled out at approximately 5.5 percent air voids for the remaining 19 months in the evaluation.

Wright, D. F. H., and A. Burgers, "Traffic Compaction of Bituminous Concrete Surfacings." Proceedings of the Fourth Conference on Asphalt Pavements for Southern Africa. Cape Town, South Africa. 1984.

Wright et al (30) documented a study in which six dense-graded pavements in South Africa were evaluated to determine what densities are achieved in the pavement under traffic. The evaluated pavements had been in service for 5 to 6 years and carried an average daily traffic between 350 and 1,000 heavy vehicles. Field cores from the pavements indicated a linear relationship between the relative construction compaction and the amount of traffic densification. Average in-place densities of 99 to 103.5 percent of 75 blow Marshall density were recorded. The authors concluded that a range of design air voids of 3 to 5 percent seemed appropriate for low to medium volume roadways, but heavy volume pavements may need to be designed at higher air voids (6 percent or above) to reduce the chance or rutting and bleeding.

Hughes, C. S., and G. W. Maupin, Jr., "Experimental Bituminous Mixes to Minimize Pavement Rutting." Proceedings of the Association of Asphalt Paving Technologists, Volume 56, Minneapolis, MN, 1987, pp. 1-32.

Hughes and Maupin (31) discuss research that was conducted by the Virginia Transportation Research Council to determine what mix variables enhance the performance of HMA mixes. Four experimental mixes were placed on the Richmond-Petersburg Turnpike. The gradation of the four mixes was the same, with the asphalt cement (AC-20 and AC-30) and the type of anti-stripping agent (liquid and hydrated lime) being the variables. All mixes were placed on a milled surface to an approximate 2 inch lift thickness. The aggregate blend gradation passed below the maximum density line and would be described as a coarse-graded mix. Optimum asphalt contents, determined by Marshall 75 blow procedures, ranged from 4.5 to

4.6 percent for the mixes.

Average traffic levels for the travel lane of the test sections were 6,400 ESALs per day. In-place density was determined by obtaining cores at the time of construction and at 6 and 12 months after construction. As expected the density increased during the first 12 months, with an average of 0.8 percent (approximately 1.1 Million ESALs) during the first 6 months and 1.3 percent over the first 12 months (approximately 2.2 Million ESALs). Rut depths were also determined and showed an average of 0.08 inches of rutting over the first 12 months.

Brown, E. R., and S. A. Cross. "Comparison of Laboratory and Field Density of Asphalt Mixes." Transportation Research Record 1300, National Research Council, Washington, D. C., 1991, pp. 1 - 12.

Brown and Cross (32) conducted research to determine the relationship between the mix density during mix design and quality control testing to the density obtained after traffic. Eighteen pavements in six states were sampled and evaluated. Thirteen were prematurely rutted and five were satisfactory.

Cores were obtained from each pavement and were used to develop the relationship between the in-place air voids (expressed as the 20^{th} percentile air voids across the pavement) and applied traffic in 18 kip wheel loads as shown in Figure 12. A poor correlation existed between voids and traffic; however, the authors state that if a good correlation had existed, traffic alone and not other mix properties would have controlled mix densification.

Foster, C. R. "Densification of Asphalt Pavements By Traffic". Proceedings of the Conference on Airport Pavement Innovation. American Society of Civil Engineers, 1993, pp. 164 - 180.

Foster (33) documents a number of research studies that relate pavement densification to the amount of traffic. The author concludes that the densification of pavements occurs very quickly immediately following initial placement and loading (often during the first several thousand repetitions), but eventually slows to a very low densification rate with time. For the studies researched, the initial in-place air voids were determined to be the main factor that affects the pavement densification over time. Other factors such as climate and rate of loading were also found to have an effect on the densification, but not to the extent as the initial air void level. A summary of the effect of initial in-place voids on pavement densification is shown in Figure 13, where VTMd is the developed air voids and VTMc is the construction air voids. Shown in Figure 13 are the results of studies in Texas (15 pavements), Maryland (6 pavements), New York (10 pavements), Pennsylvania (24 pavements), and at the Army Corps of Engineers Waterways Experiment Station (WES) (18 pavements).

An in-place air void level of 8 percent was determined to be the void level that generally resulted in approximately 4 percent (lab) voids for the final air void level in the pavement.

Hossain, M. and L. Schofield, "Performance of Recycled Asphalt Concrete Materials in an Arid Climate." .....

Hossain et al (34) describe a study in which eight experimental asphalt mix sections were evaluated in 1981 by the Arizona Department of Transportation on Interstate 8 in Southwestern Arizona. The objective was to compare the long-term performance of recycled and virgin asphalt mixes in a very arid climate. Six of the sections were two inch overlays with the remaining two being four inch overlays. At the time of evaluation the cumulative traffic on the project was 7 million ESALs. The average maximum and minimum air temperatures over the past 30 years was 89°F and 56°F for project location.

Performance evaluations of the mixes after 10 years of service indicated that all the sections exhibited distresses to varying degrees. The results showed that the four inch overlay sections had rut depths that were approximately twice that of the two inch overlay sections (0.45 inches compared to 0.21 inches). Field cores were obtained, in and between the wheel paths of the sections, to determine the amount of densification over the 10 years of service. The results indicated that the densification of the asphalt layers was mostly responsible for the rutting for both the virgin and recycled mixes. A possible relationship between the nominal maximum aggregate size and the lift thickness and the amount of densification of the pavement over time was also determined.

Hanson, D. I., R. B. Mallick and E. R. Brown. "Five Year Evaluation of HMA Properties at the AAMAS Test Sections". Transportation Research Record 1454, National Research Council, Washington, D. C., 1994, pp. 134 - 143.

Hanson et al (35) document the results of a study designed to evaluate the change in mix properties of five Asphalt-Aggregate Mixture Analysis System (AAMAS) mixes. Project information for each of the projects is shown in Table 6.

Cores were obtained from each of the five projects and their properties determined. Among the properties was the in-place air void content. The in-place air voids after five years were found to be statistically different from the two year voids in approximately 67 percent of the cases. As expected, the vast majority of time, the five year voids were less than the two year voids.

The change in voids was related to the traffic volume to determine the magnitude and rate of mix densification. The relationship between the change in air voids and the two and five year traffic is shown in Figure 14. The results indicate that there is a clear trend for densification with traffic, but the relationship holds a large amount of scatter.

Conclusions reached from the study are that the densification of pavements continue beyond two years of service, mixes with higher initial in-place voids have higher rates of void changes, the five year in-place voids were generally less than the design air voids, and that further densification studies should be carried out on surface course and heavy duty pavements for three to four years to more accurately determine the relationship between traffic and densification.

Blankenship, 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.

Blankenship et al (*36*) discuss the experimental approach, results, and conclusions from the initial N_{design} experiment. 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 provided 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 that 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}.

Blankenship, P. B., Gyratory Compaction Characteristics: Relation to Service Densities of Asphalt Mixtures. Master's Thesis, University of Kentucky, 1994.

Blankenship, in his Master's thesis (37) entitled *Gyratory Compaction Characteristics: Relation to Service Densities of Asphalt Mixes*, presents the experimental approach, results, and conclusions from the initial N_{design} experiment. The N_{design} experiment, previously mentioned in less detail by Blankenship et al (36) 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 consisted of 27 pavement sites with 54 mixes. This provided three climates (hot, warm, and cool), three design traffic levels (low, medium, and high), three pavement ages, and two pavement layers (upper and lower). However, it was later decided to evaluate only pavements that 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. Project information for the 15 sites is provided in Table 7. Two important assumptions were made that all mixes evaluated were designed to have approximately 4 percent air voids in the laboratory and in-place air voids of 8 percent immediately after construction.

The aged asphalt was extracted from 305 mm cores taken from the wheel paths of the pavements and the recovered aggregate remixed with an unaged AC-20 asphalt cement. Only two cores were obtained from each of the projects for the evaluation. The mixed specimens were then aged for 4 hours at 135°C and compacted to 230 gyrations using the SHRP gyratory compactor using a gyration angle of 1.0 degree, a rotational speed of 30 rpm, and a vertical pressure of 600 kPa. Mixes with 19.0 mm and less nominal maximum aggregate sizes (approximately 40 percent of the mixes) were prepared using the 100-mm compaction mold while the 150-mm mold was used for mixes with nominal aggregate sizes greater than 19.0 mm. All mixes in the study were dense graded.

While the intent of the study was to compact samples at a gyration angle of 1.0 degrees, a check of the angle after the work had been completed revealed the angle to be approximately 1.3 degrees, not the 1.0 degree which had been previously selected. This was due primarily to the deflection in the frame of the gyratory compactor. Therefore, the angle was adjusted back to approximately 1.0 degree, and the process repeated. Due to the limited amount of available aggregate, the aggregate had to be extracted from the compacted samples made using the 1.3 degree angle, re-mixed with an AC-20 asphalt cement and the mix re-compacted.

Regression analysis of the 1.3 and the 1.0 degree gyration angle test results are provided in Figures 15 and 16, respectively. These figures provide a relationship between the traffic level and the number of design gyrations to achieve four percent air voids for hot, warm, and cool climate mixes. From Figure 15 and 16, two sets of N_{design} values were available from the study, one for a gyration angle of 1.3 degree and the other for a 1.0 degree gyration angle. As expected, the use of the 1.3 degree angle resulted in a N_{design} level that was lower than the N_{design} level required at the 1.0 degree angle. For a traffic level of 1 million ESALs and for hot and warm climates, a difference in the N_{design} of 30 gyrations was seen between the 1.3 and the 1.0 degree gyration angles. The average differences in N_{design} values for the various ESALs from the use of the 1.3 and the 1.0 degree gyration angle can be observed in Figure 17. From Figure 17, it is seen that the difference between the N_{design} values determined from the 1.3 and the 1.0 degree angles increases with an increase in the traffic level.

The decision was made in the study to provide N_{design} levels based upon the 1.0 degree gyration angle data, primarily because the SHRP gyratory specification called for a 1.0 degree angle. The N_{design} levels obtained from this study, using the 1.0 degree gyration angle results, were used to create the original N_{design} compaction matrix, provided previously in Table 2.3. Values of N_{design} greater than 32 million ESALs were extrapolated from the regression results obtained in the N_{design} experiment.

Newcomb, D. E., R. Olson, M. Gardiner, and J. Teig, "Traffic Densification of Asphalt Concrete Pavements." Transportation Research Record 1575, TRB, National Research Council, Washington D.C., (1997), pp. 1 - 9.

Newcomb et al (*38*) describe a five year research study conducted to determine the relationship between traffic and in-place densification on 16 projects completed in 1990 in Minnesota. The pavements were primary overlays and represented a wide range of traffic from 1,050 to 69,000 vehicles per day.

Cores were obtained from between and within the wheel path for each of the sections during construction and each year for five years after construction. The results indicated that the majority of densification occurred during the first year of service and that the densification generally occurred in the top 65 mm for pavements with ADT less than 10,000 (low traffic volume). Little densification occurred in the layers below 65 mm from the finished surface for the low traffic volume pavements. The authors suggest that the in-place voids immediately after construction for these lower layers must be close to the design voids to account for this lack of densification. The authors suggest that the lower layers may need to be designed at 2 percent lab voids to aid the field compaction. Densification for high volume pavements (greater than 50,000 ADT) occurred mostly in the top 100 mm when the initial in-place air voids were between 6 and 7 percent; however, with initial voids of 9 to 10 percent, the densification occurred throughout the full depth of the HMA. Rutting was seen to occur in the pavements that were compacted to 9 to 10 percent air voids during compaction.

Brown, E. R., and Mallick, R. B., "An Initial Evaluation of N_{design} Superpave Gyratory

Compactor." Journal of the Association of Asphalt Paving Technologists (AAPT), Minneapolis, MN, Volume 67, 1998, pp. 101 - 124..

Brown and Mallick (39) conducted research in which specimens were compacted in the Superpave gyratory compactor at different gyration levels and then 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 known traffic. The cores were taken immediately after construction and after one, two, and three years of 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 that 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. This conclusion is illustrated in Figure 18.
- The values of voids at $N_{initial}$ and $N_{maximum}$ were lower than the specified values based upon the laboratory data obtained from the project.
- The density of laboratory prepared samples was approximately one percent greater than the density of the compacted-reheated samples at similar gyration levels. The difference became less as the gyration level increased.

2.4 LITERATURE REVIEW SUMMARY

2.4.1 Development and Evaluation of the Superpave Gyratory Compactor

The Superpave gyratory compactor, developed during 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; 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 mixes with up to 37.5 nominal maximum size aggregate and reduced density variability of compacted specimens.

Generally, research (*10*) has shown that for medium/high traffic levels, i.e., higher levels of compaction or N_{design}, the gyratory compactor yields higher specimen densities and therefore lower optimum asphalt contents and lower voids in the mineral aggregate than the Marshall hammer. Other studies (*11*, *14*) indicated that the gyratory compactor better identified mixture component changes due to plant production variability than did the Marshall hammer.

Research, (*9*, *39*) in which the in-place density of pavements was monitored with time and traffic, concluded that 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 mixture volumetrics when the N_{design} levels differ by only one or two gyrations.

Initially, the Superpave mix design procedure required that specimens be compacted to $N_{maximum}$ and their volumetric properties back-calculated to N_{design} and $N_{initial}$. Literature (*8*) suggests that the procedure computes inaccurate volumetric properties. Errors vary depending upon the gradation coarseness and asphalt content. Mixes with a coarse gradation, such as a Superpave mixture below the restricted zone or a stone matrix asphalt (SMA), have higher errors than fine-graded mixes.

Research (*15*, *16*, *17*) indicates the slope of the gyratory compaction curve can provide an indication of the properties of the compacted mixture. It has been shown that the slope differentiates between different aggregate gradations. Finer gradations exhibit flatter compaction slopes. However, Superpave mixture analysis and wheel tracking testing did not support the idea that mixes with flatter slopes had weaker aggregate structures.

Literature (15, 17) suggests that many mixes with fine gradations have difficulty meeting the density requirement of less than 89 percent of G_{mm} at $N_{initial}$. Further, with few exceptions, compacted mixes meet the density requirement of less than 98 percent of G_{mm} at $N_{maximum}$ and coarse-graded mixes tend to have higher densities at $N_{maximum}$ than fine graded mixes.

2.4.2 Evaluation of N_{design} and the In-Place Densification of Mixes

Results obtained from the initial N_{design} experiment were used to establish compaction levels for Superpave mixes. A total of 28 levels (7 traffic levels and 4 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. The experiment had a number of limitations, some of which are provided below:

- The number of projects was limited and the maximum traffic level evaluated was approximately 32 million ESALs. N_{design} values for traffic levels greater than 32 million ESALs were extrapolated.
- Aggregate was extracted from field obtained cores and re-mixed with an AC-20 asphalt cement, regardless of the original asphalt cement used in the project.
- Although a 150-mm diameter compaction mold is currently specified in the Superpave system, a 100-mm mold was used for approximately 40 percent of the mixes evaluated. These mixes had nominal maximum aggregate sizes less than 19.0 mm.
- The mixes evaluated in the experiment were conventional dense-graded mixes. In many cases, the mixes used today in the Superpave system are much coarser (may not densify to the same degree or at the same rate) than those conventional mixes.
- Problems were experienced in achieving the appropriate gyration angle in the study.

Samples were originally compacted with a gyration angle of 1.3 degrees; not the 1.0 degree angle, which was the specified angle by the SHRP researchers. This was due to problems with the rigidity of the gyratory compactor used in the experiment. Therefore the gyration angle was changed to 1.0 degree and the testing performed a second time.

- The N_{design} values recommended from the study were based upon the 1.0 degree gyration angle. Currently, Superpave uses a 1.25 degree gyration angle, but recommended N_{design} levels are based upon the 1.0 degree gyration angle results. It was shown that the 1.3 degree gyration angle provided approximately a 30 gyrations difference for the warm and hot climates mixes evaluated. Therefore, it would appear that the N_{design} levels used today are too high for the gyration angle that is specified (1.25 degree).

N_{design} values obtained by Brown and Mallick (39) were approximately 30 gyrations lower than those currently specified under Superpave. The research (39) indicated that an N_{design} of 46 gyrations was appropriate for a mix with an average maximum air temperature of less than 39°C and 1 million ESALs. The Superpave specified N_{design} (at that time) value was 76 gyrations, which resulted in a difference of 30 gyrations.

Research conducted by Dillard (18), Bright et al (25), and Epps et al (27) seems to back up the assumption provided in the study test plan of mixes generally compacting to the same ultimate density, but with different traffic levels and requiring different amounts of time. Other research by Foster (33) indicates that the amount of achieved density is related to the degree of compaction during construction.

It appears from the research reviewed that the vast majority of in-place densification of a pavement occurs during the first year to two years, with some pavements achieving their ultimate density in only three to six months. Serafin et al (29) reported that the average in-place density for twenty four test pavements slowed and leveled off after three to four years of service. Palmer and Thomas (26) indicated that approximately 50 percent of the total five year in-place densification occurred during the first year of service, with high volume pavements densifying at approximately twice the rate of low to medium volume pavements. Hughes and Maupin (27) found that the increase in the in-place density during the first six months of service was approximately 62 percent of the densification observed during the first year of service. Epps et al (*28*) showed that approximately 80 percent of the total 2 year densification was obtained during the first year of service for a variety of Texas mixes. Graham et al (*32*) showed that for Marshall 50 blow designed mixes, the pavements densified significantly during the first year, but to a lesser degree in the second year. Newcomb et al (*33*) reported that the majority of densification occurred during the first year of service. In another study, Woodward and Vicelja (*23*) reported an increase of 3 lb/ft^3 within the first month of service.

Field (*19*) indicated that to some degree the rate of in-place densification was attributable to the time of placement. For example, mixes placed during the summer typically densify at a greater rate than mixes placed during the early fall, for obvious reasons. The effect of temperature was also illustrated by Paterson (*26*) in which the in-place density of mixes placed at a test facility in New Zealand increased by approximately 6 percent from the construction density. Patterson (*26*) also indicated that density was difficult to achieve in thin lifts while over-compaction typically occurred in thick lifts.

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TABLE 1 Revised N_{design} Levels

Traffic Level (Million ESALs)	Gyrations
Less than 300,000	50
300,000 to 3 million	75
3 million to 30 million	100
Greater than 30 million	125

TABLE 2 Test Plan for Superpave Gyratory Compactor Field Verification (1)

Levels	Input Variables				
	AC (%)	P0.075 (%)	P2.36 (%)	NMS (mm)	Natural [1] Sand (%)
Low	4.7	3.8	29.3	9.5	0
Medium	5.3	6.0	35.2	12.5	15
High	5.9	9.7	38.9	19.0	30

Notes: (1) Natural Sand Percentage Only Evaluated for the 19.0 mm NMS Mix

TABLE 3 Summary of the Effect of Compaction Response Variable (_1_)

Gyratory Compaction Response Variable	Input Variables Increasing				
	AC (%)	P0.075 (%)	P2.36 (%)	NMS (mm)	Natural [1] Sand (%)
C_{10}	Increases	Increases	Increases	Same	Increases
C_{230}	Increases	Increases	Same	Same	Increases
K	Increases	Increases	Same	Same	Same

TABLE 4 Superpave Gyratory Compaction Parameters (2)

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

TABLE 5 Main Factors and Levels Evaluated in the AASHTO TP-4 Ruggedness Experiment (4)

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 Pressure, 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

TABLE 6 Description of the AAMAS Test Sections (35)

State Project	Colorado CO-009	Michigan MI-0021	Texas TX-0021	Virginia VA-0621	Wyoming WY-0080
Type of Section	Lower Surface Course	Surface Course	Base Course	Base Course	Lower Surface Course
Average Thickness (mm)	34.2	45.7	71.6	96.5	55.3
Depth from Surface (mm)	57.1	0.00	76.2	>100.0	50.8
2 Year ESALs (Millions)	0.01	0.16	0.26	0.01	0.96
5 Year ESALs (Millions)	0.03	0.42	0.69	0.04	2.57

TABLE 7 N_{design} Experiment Project Information (37)

State	Age	Current Traffic (ESALs)	20 Year Design Traffic (ESALs)	20 Year Design Traffic Level	Climate	Nominal Max. Aggregate Size	HMA Depth (mm)
Washington	17	709000	834000	Low	Cool	12.5	234
Kentucky	17	430000	506000	Low	Cool	19.0	371
Delaware	25	9269000	7420000	Medium	Cool	N/A	236
Saskatchewan (Canada)	20	1930000	2270000	Medium	Cool	12.5	185
Indiana	15	24056000	32100000	High	Cool	25.0	389
Oregon	25	28713000	23000000	High	Cool	19.0	N/A
Florida	13	600000	923000	Low	Warm	9.5	267
Texas	20	937000	937000	Low	Warm	9.5	170
Oklahoma	13	2805000	4320000	Medium	Warm	25.0	241
Texas	13	2561000	3940000	Medium	Warm	37.5	267
Arizona	12	11828000	19700000	High	Warm	19.0	201
Arizona	13	11828000	18200000	High	Warm	19.0	119
Nevada	15	708000	944000	Low	Hot	12.5	173
California	19	6631000	698000	Medium	Hot	12.5	168
Arizona	15	20827000	27800000	High	Hot	19.0	320

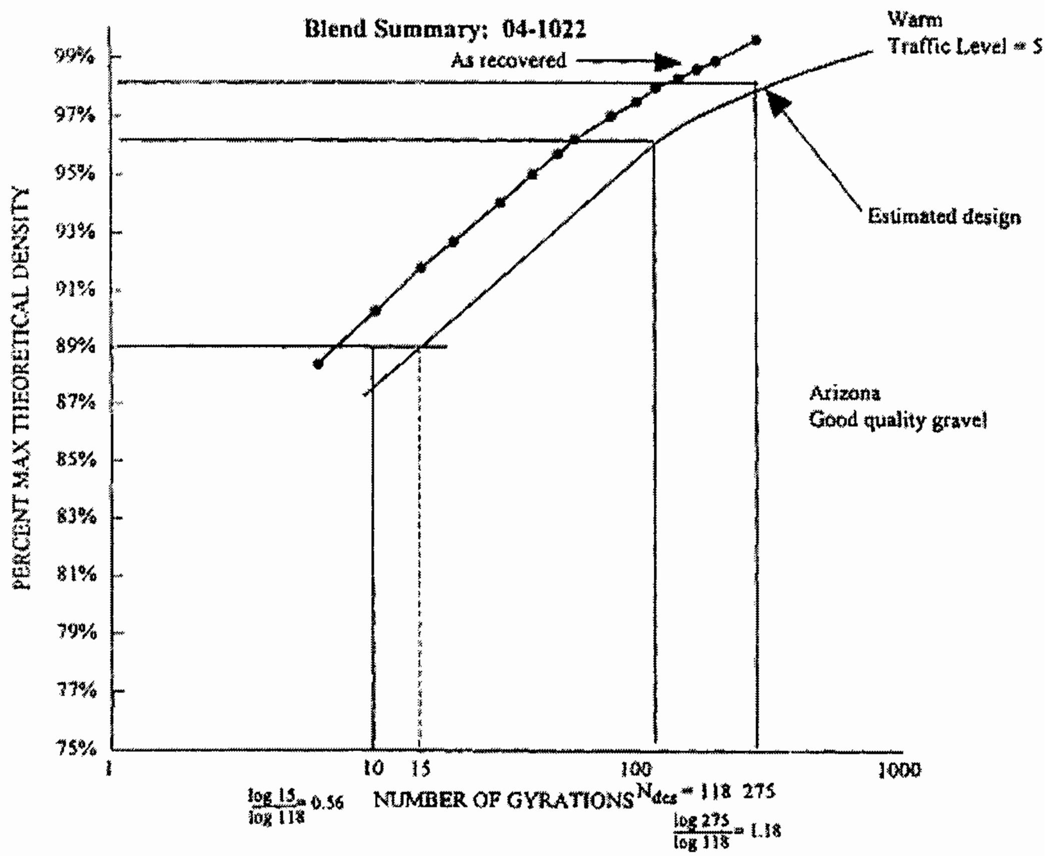

FIGURE 1 $N_{initial}$ and $N_{maximum}$ Relationship from N_{design} (L)

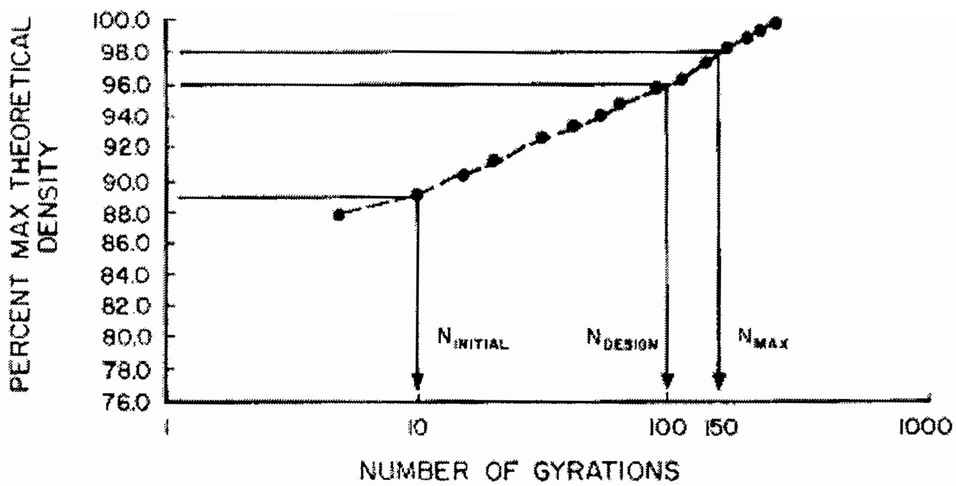

FIGURE 2 Typical Superpave Gyratory Compactor Densification Curve (2)

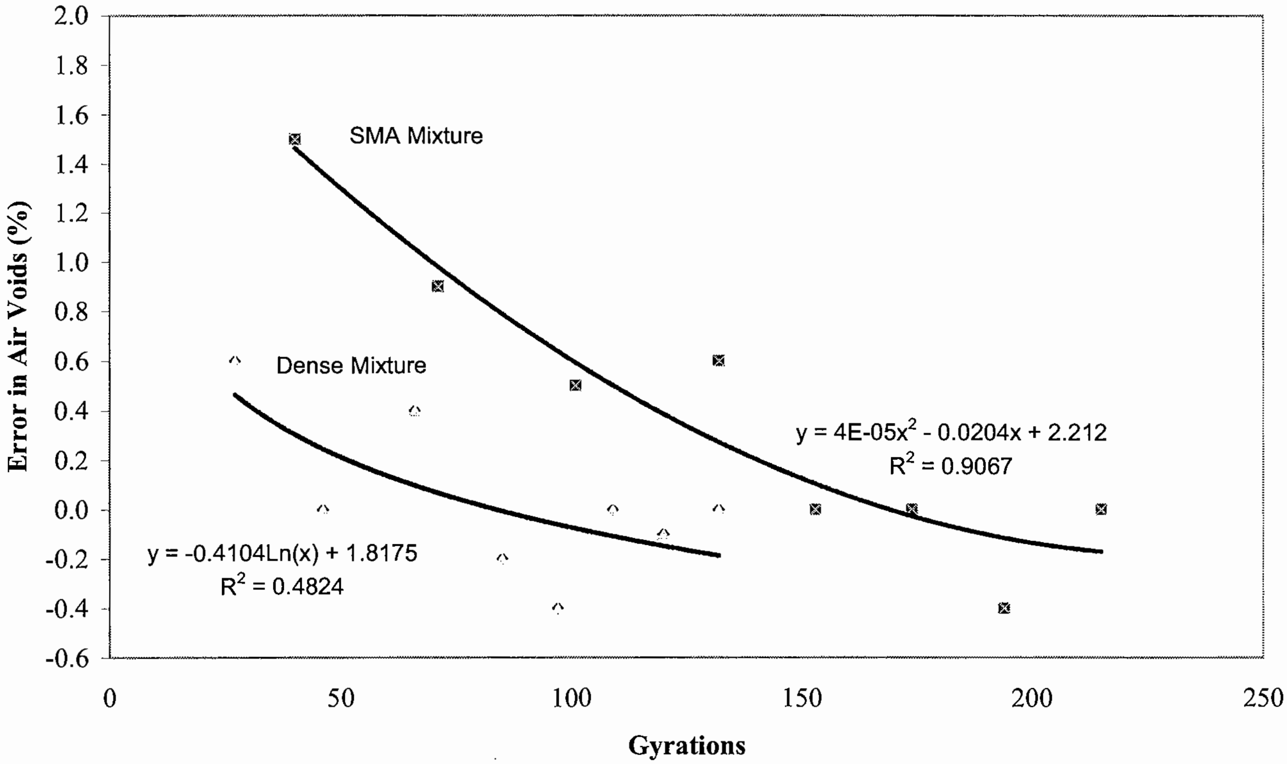

FIGURE 3 Error in Air Voids versus Gyrations (8)

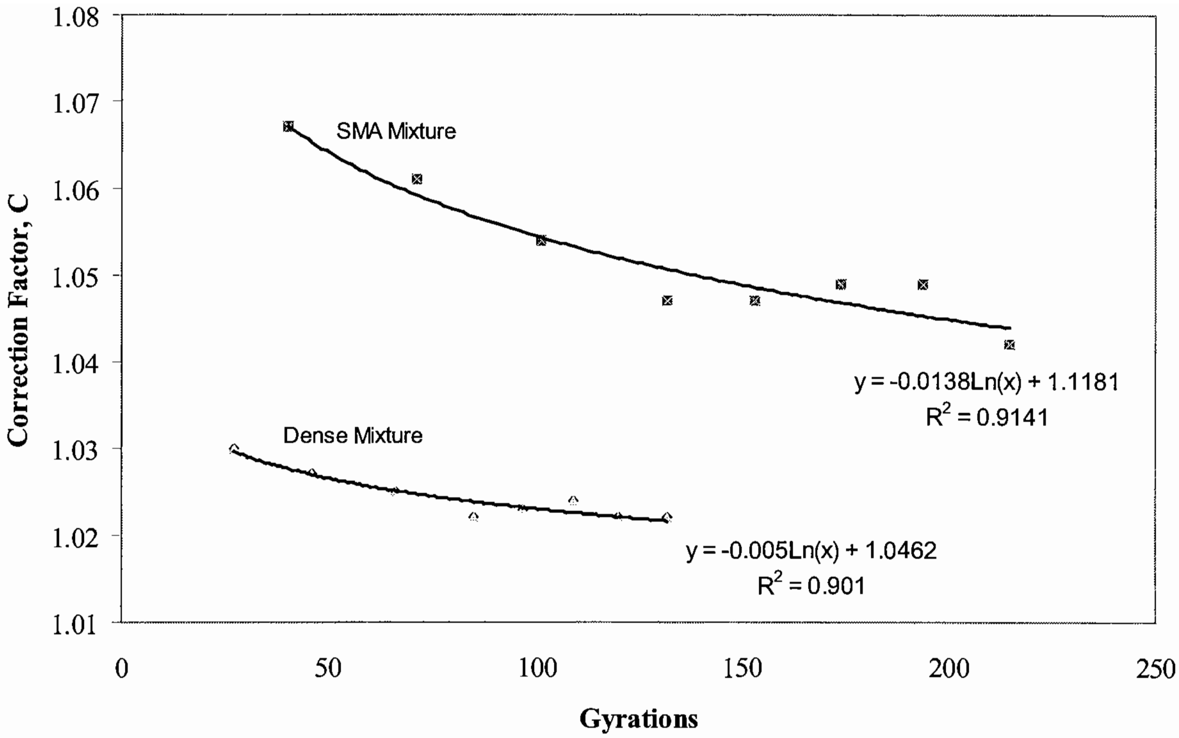

FIGURE 4 Relationship of the Correction Factor versus Gyration Level (8)

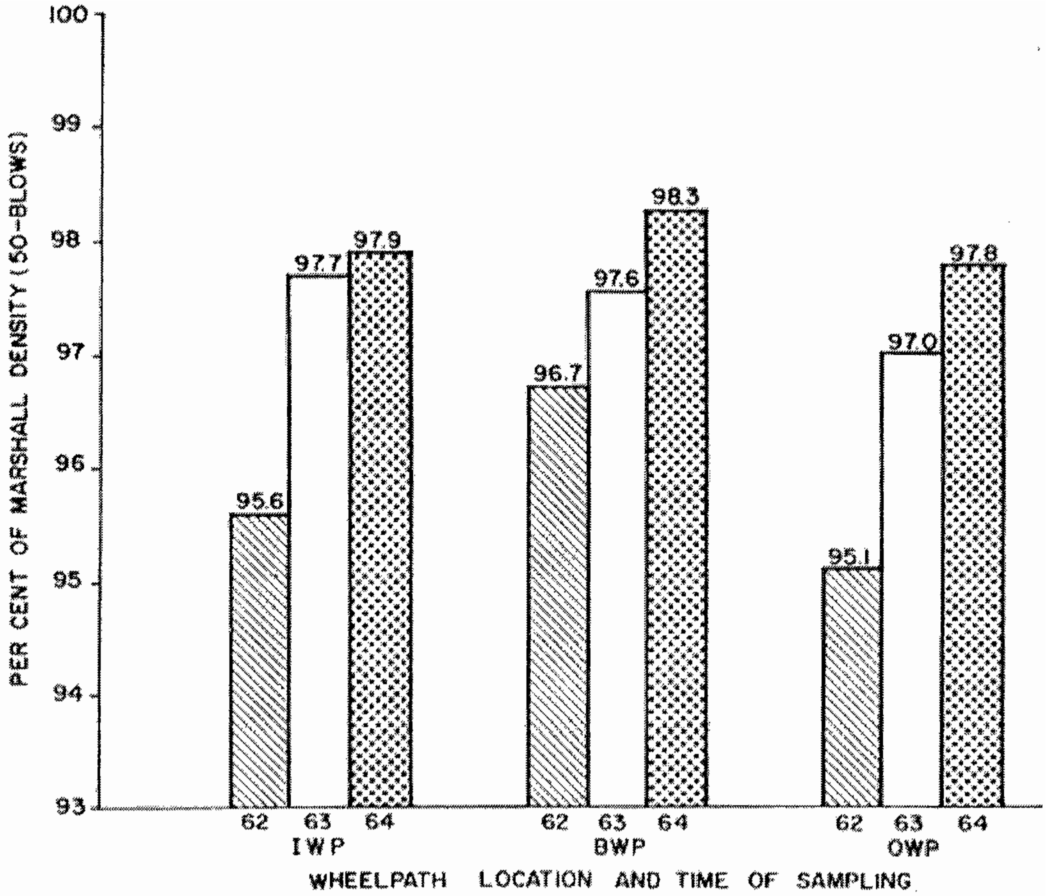

FIGURE 5 In-Place Density with Time for Different Wheelpaths (21)

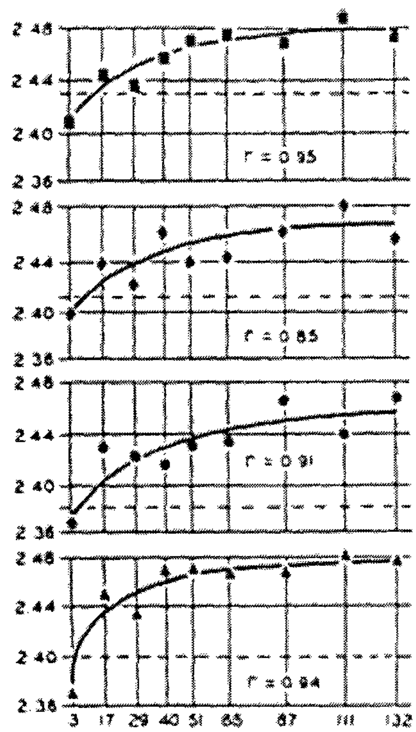

FIGURE 6 Core Bulk Specific Gravity versus Time (Traffic) (23)

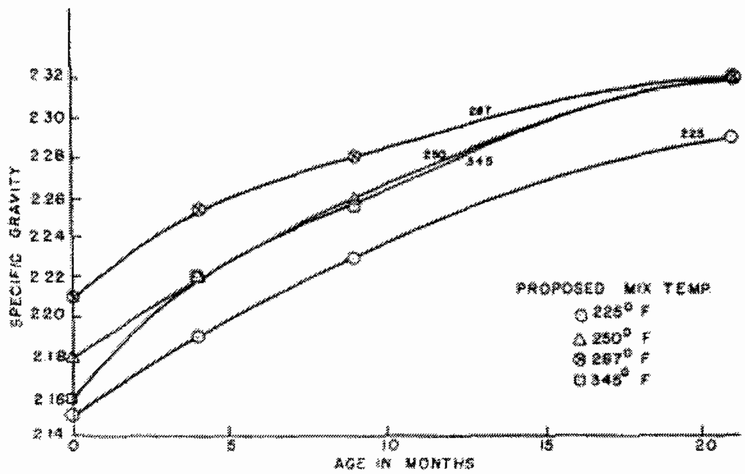

FIGURE 7 In-Place Densification with Time for Different Placement Temperatures (25)

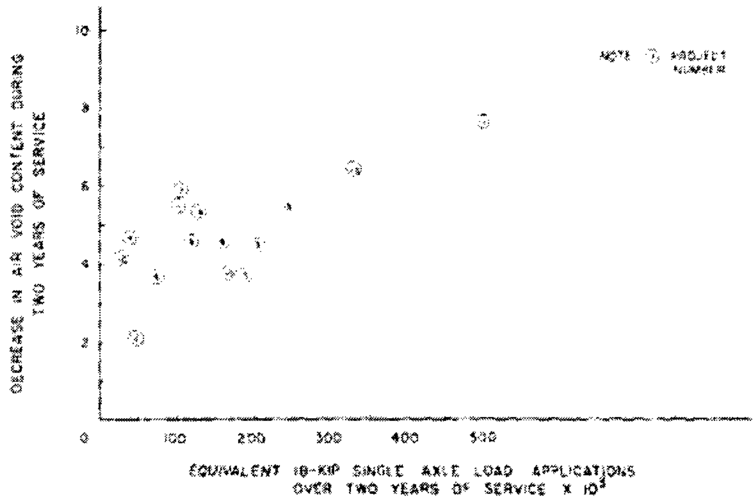

FIGURE 8 Effect of Traffic on In-Place Air Voids (27)

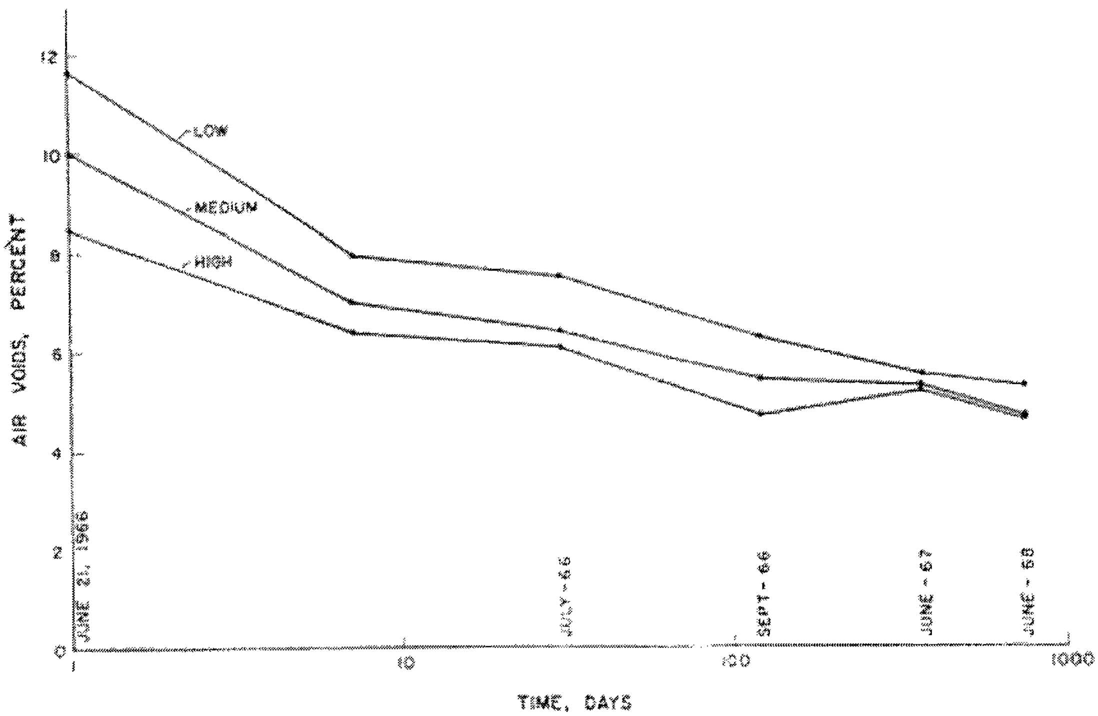

FIGURE 9 Densification for Low, Medium, and High Initial Compactive Efforts (27)

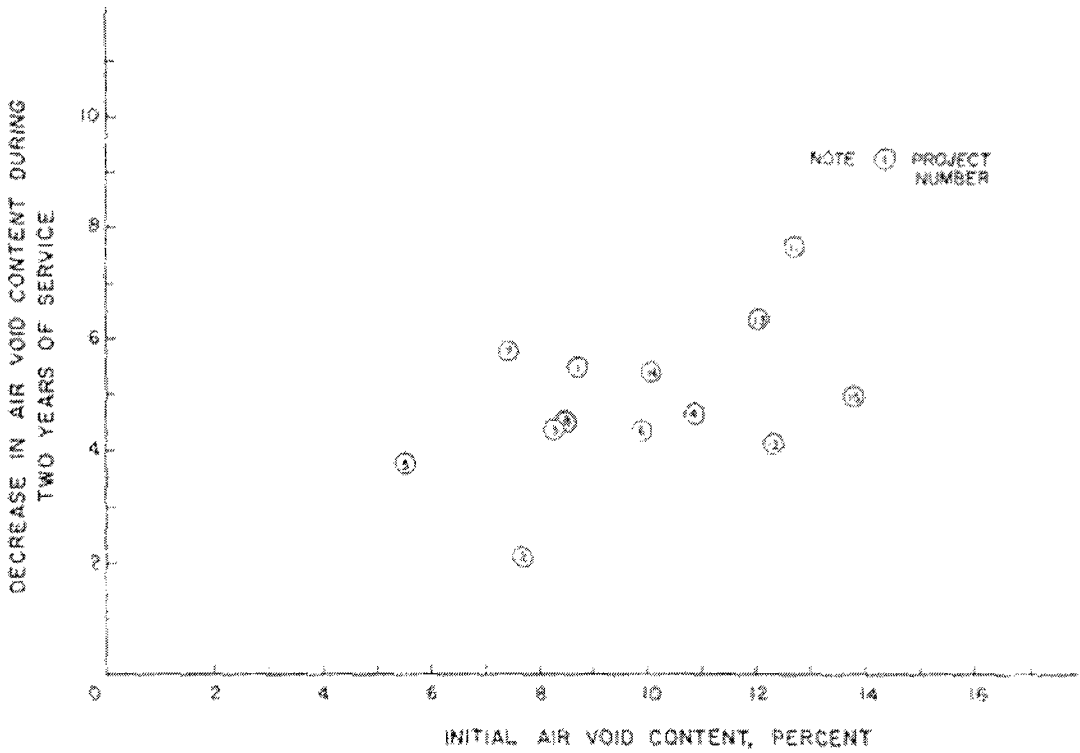

FIGURE 10 Effect of Initial Compaction Level on Air Voids for All Projects (27)

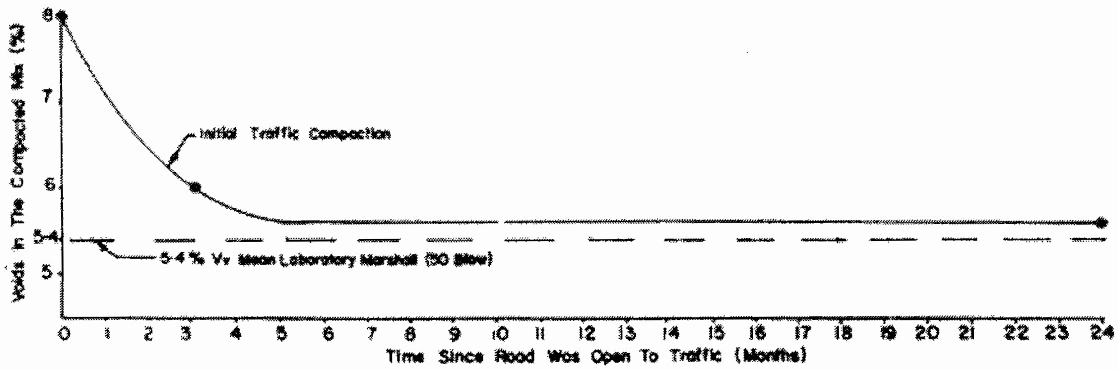

FIGURE 11 Effect of Traffic on Initial Densification (<u>29</u>)

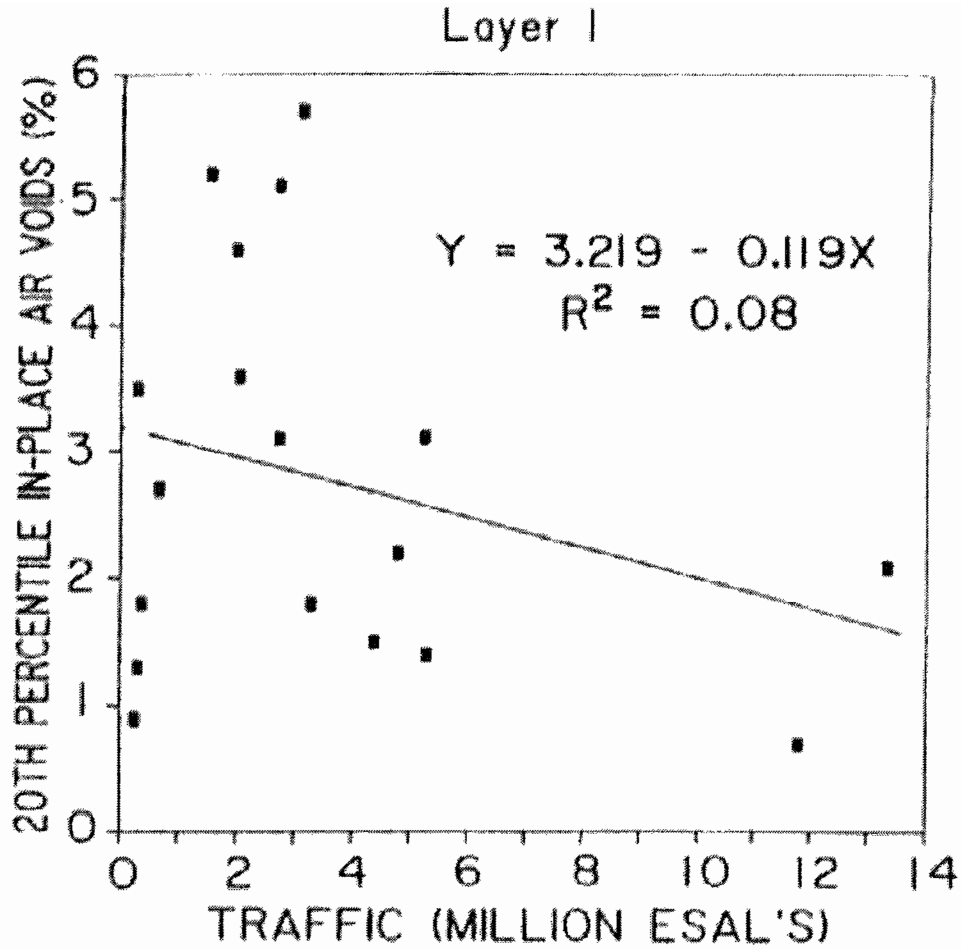

FIGURE 12 Air Voids versus Traffic (32)

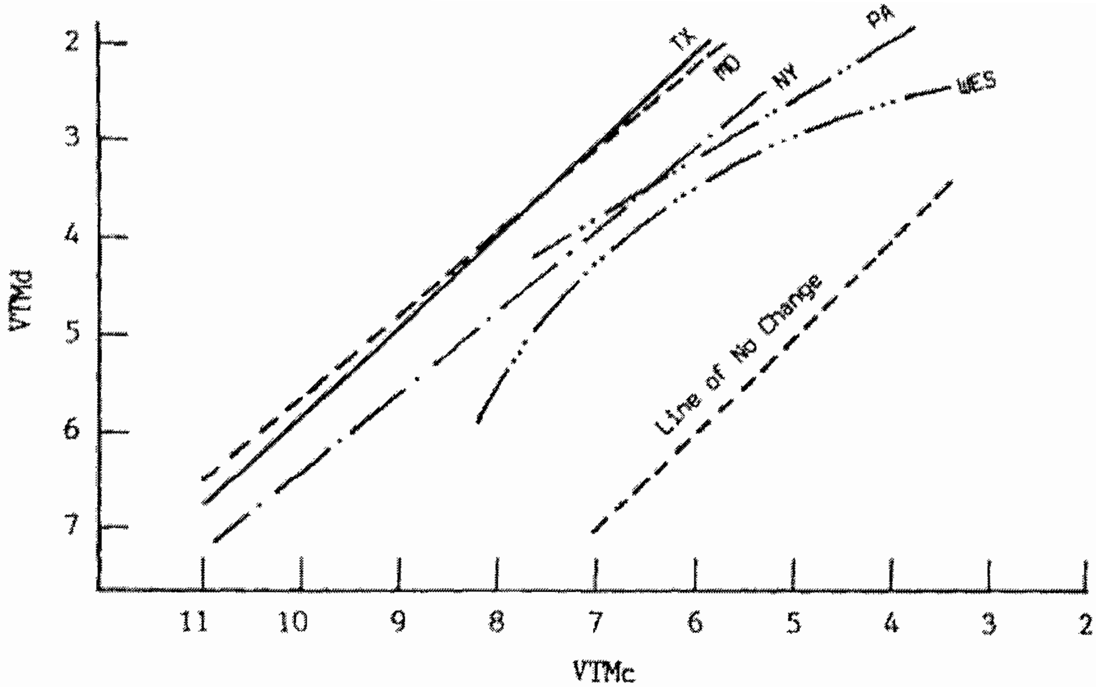

FIGURE 13 Relationship between Design and In-place Air Voids (34)

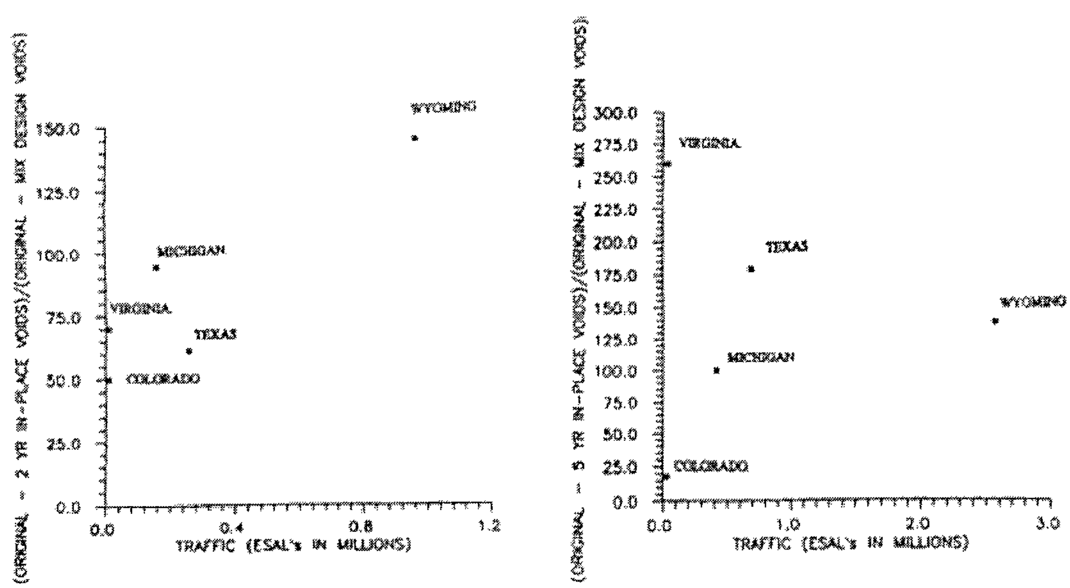

FIGURE 14 Densification for AAMAS Projects after Two and Five Years of Traffic (35)

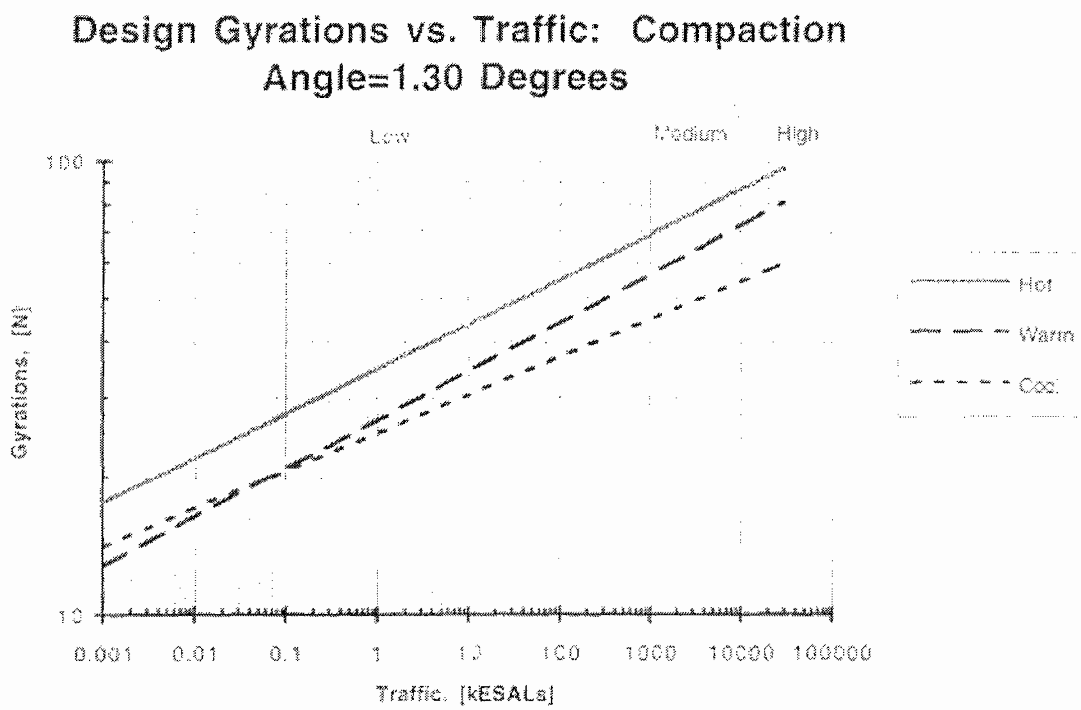

FIGURE 15 N_{design} versus Traffic for a Gyration Angle = 1.3 Degree (37)

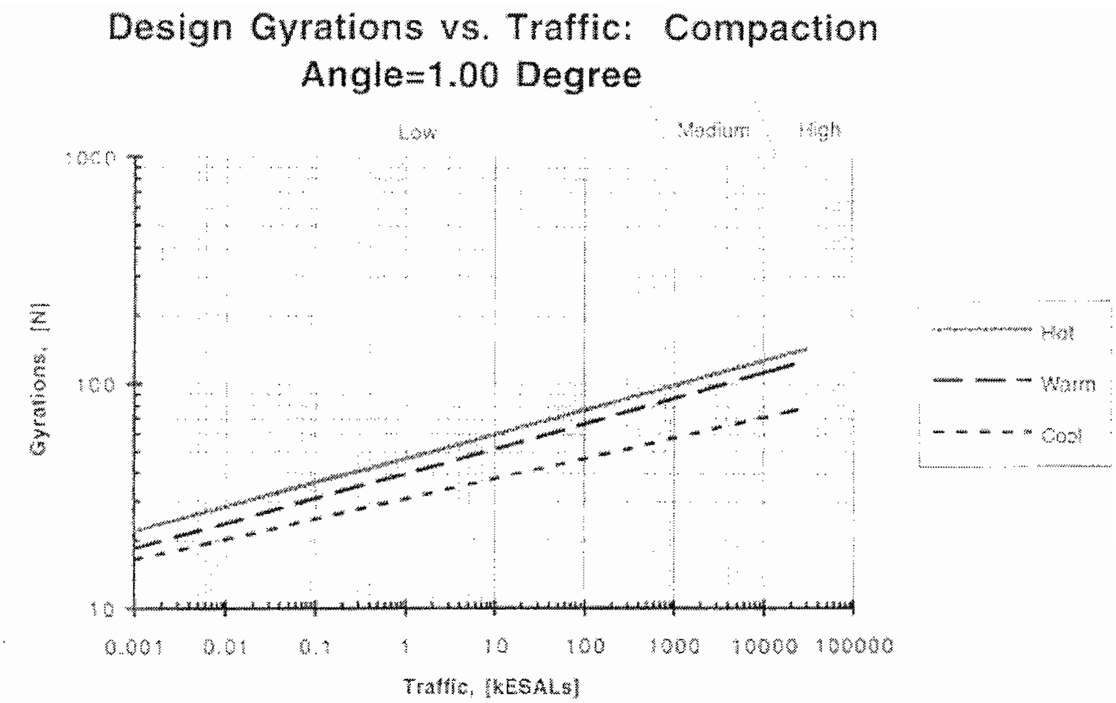

FIGURE 16 N_{design} versus Traffic for a Gyration Angle = 1.0 Degree (37)

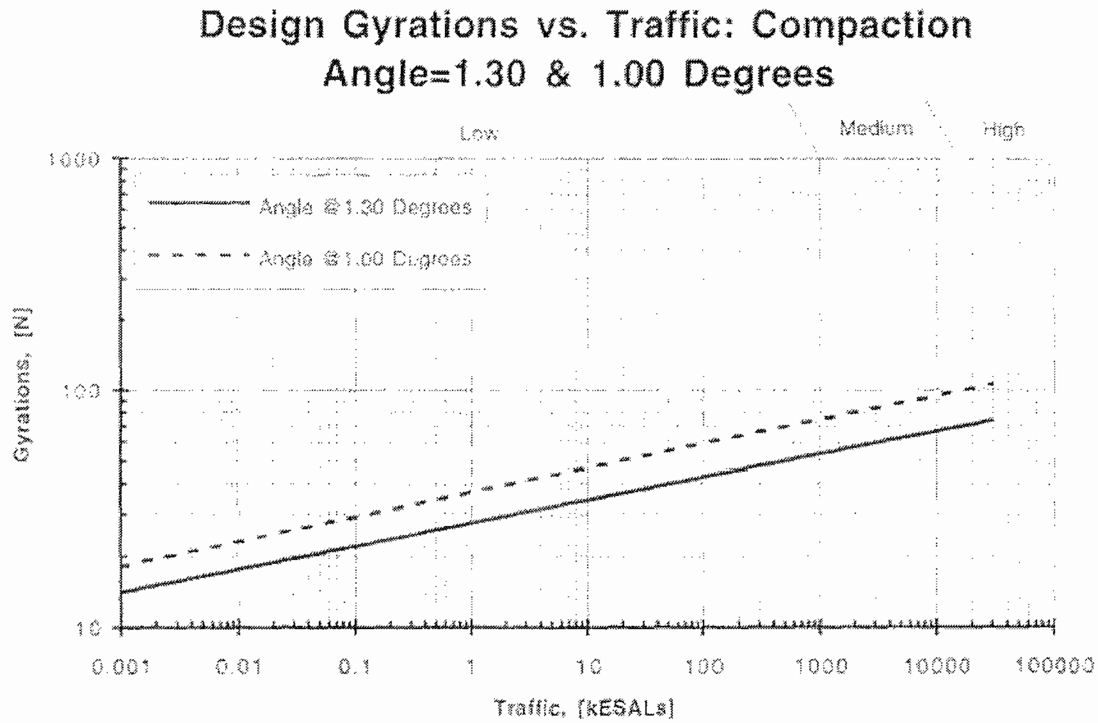

FIGURE 17 Average N_{design} versus Traffic for Gyration Angles of 1.3 and 1.0 Degrees (Hot and Warm Climates Only) (37)

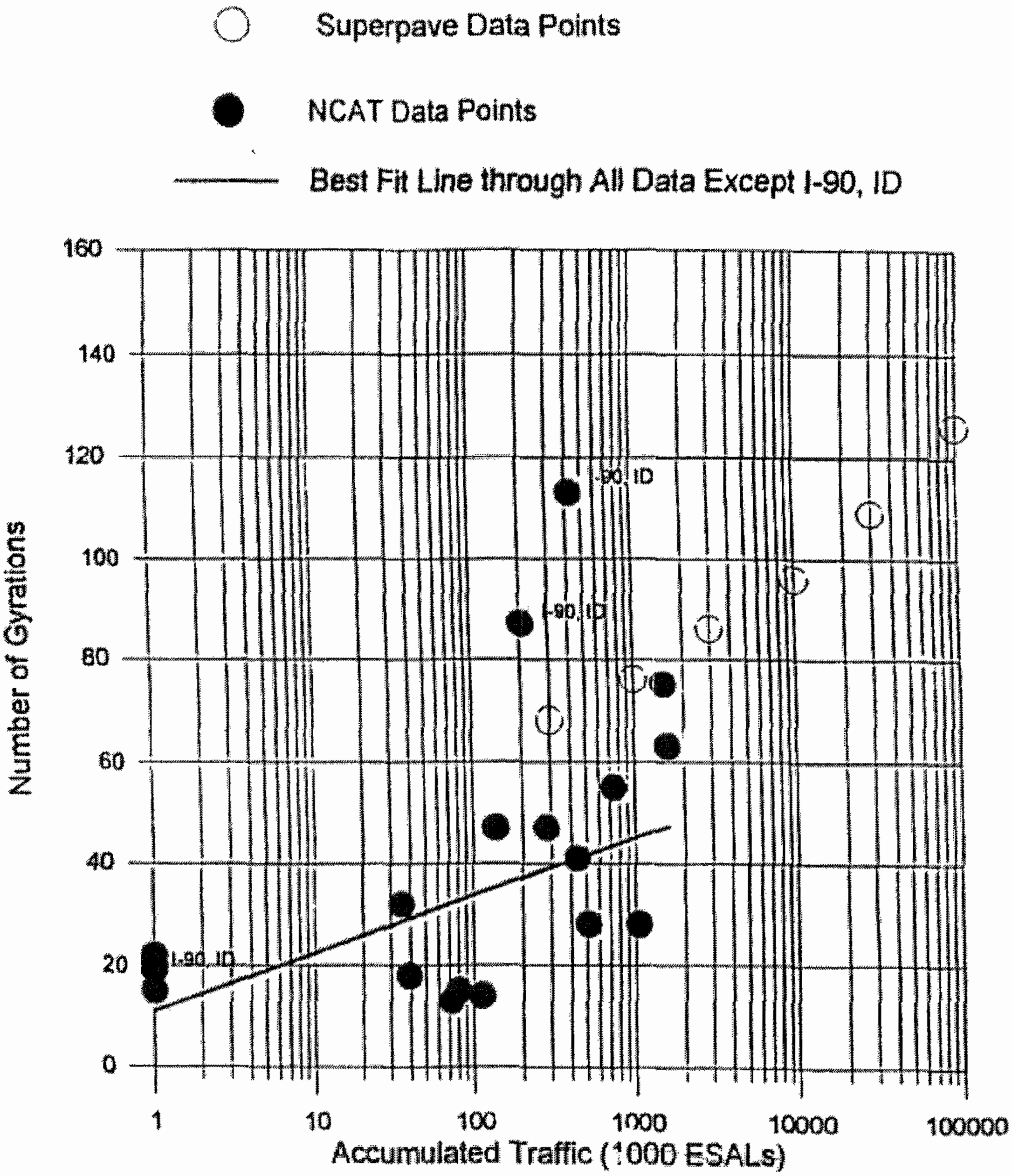

FIGURE 18 N_{design} versus Traffic (ESALs) (39)