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

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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.

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

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

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:

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

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.

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

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 20th 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.

CHAPTER 3 REFERENCES

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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

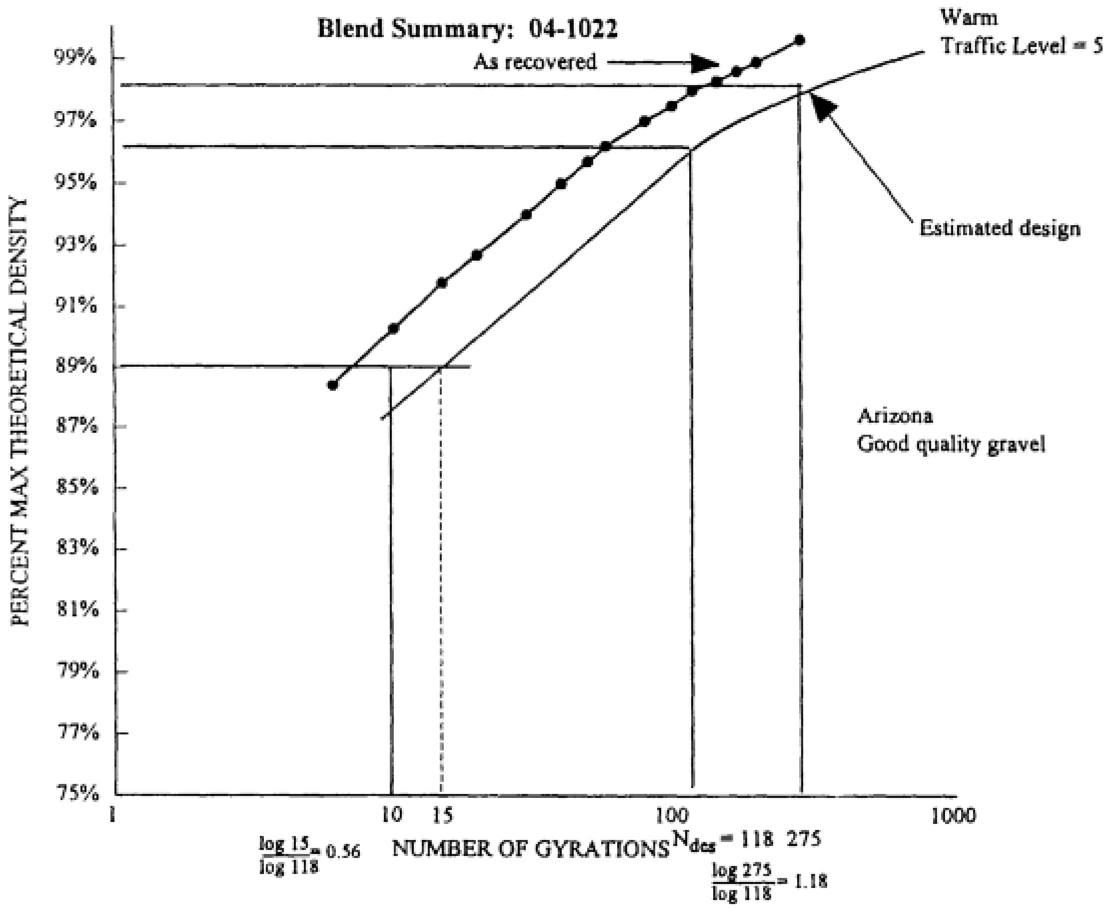

FIGURE 1 $N_{initial}$ and $N_{maximum}$ Relationship from N_{design} (1)

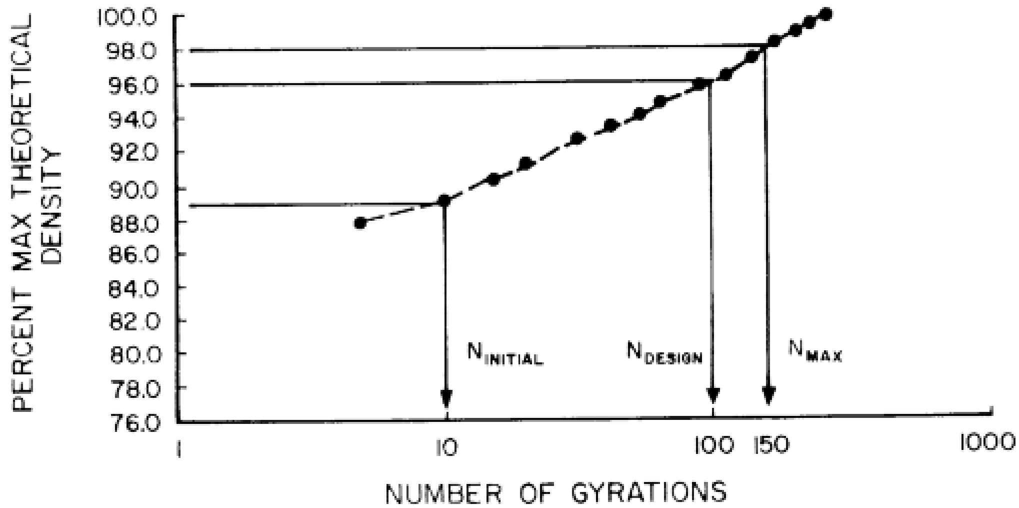

FIGURE 2 Typical Superpave Gyratory Compactor Densification Curve (2)

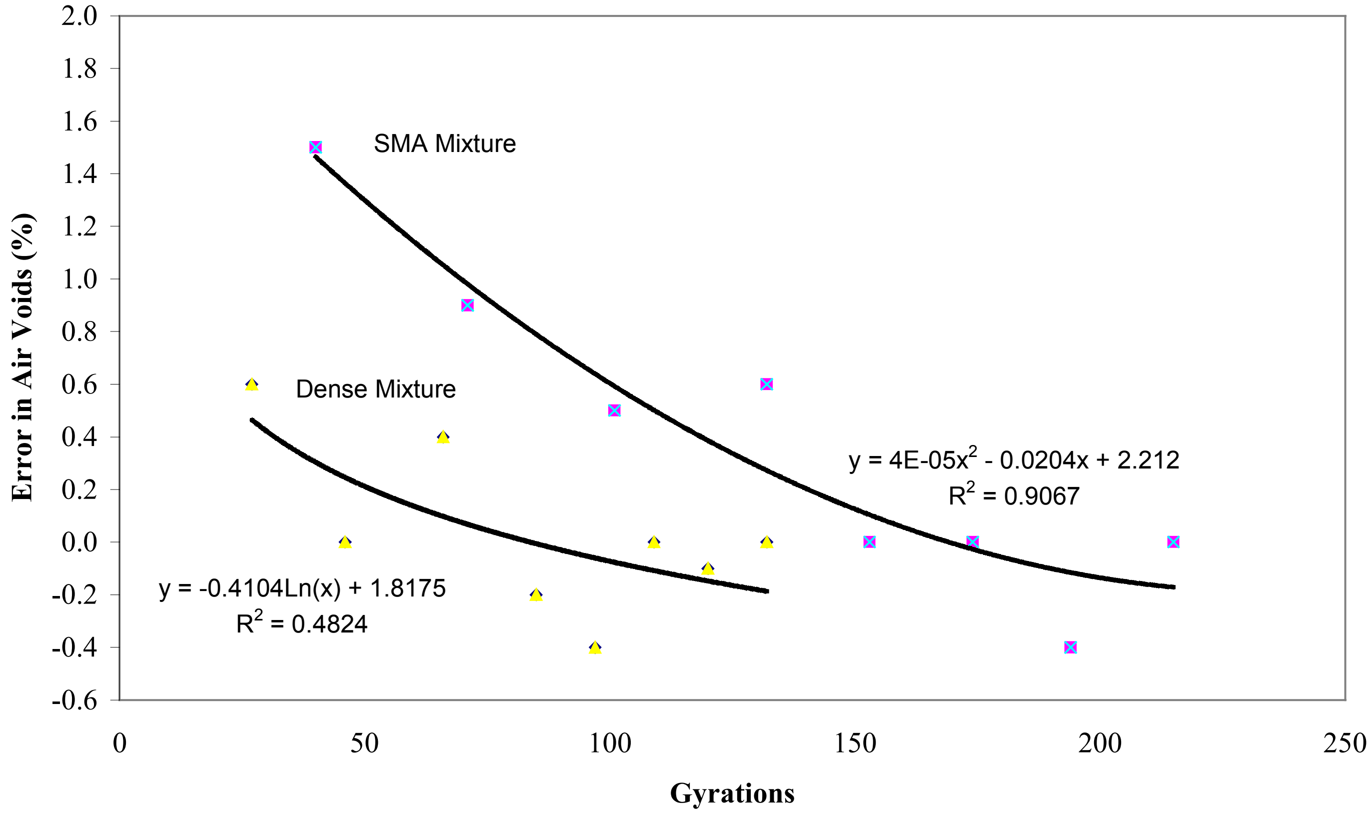

FIGURE 3 Error in Air Voids versus Gyrations (8)

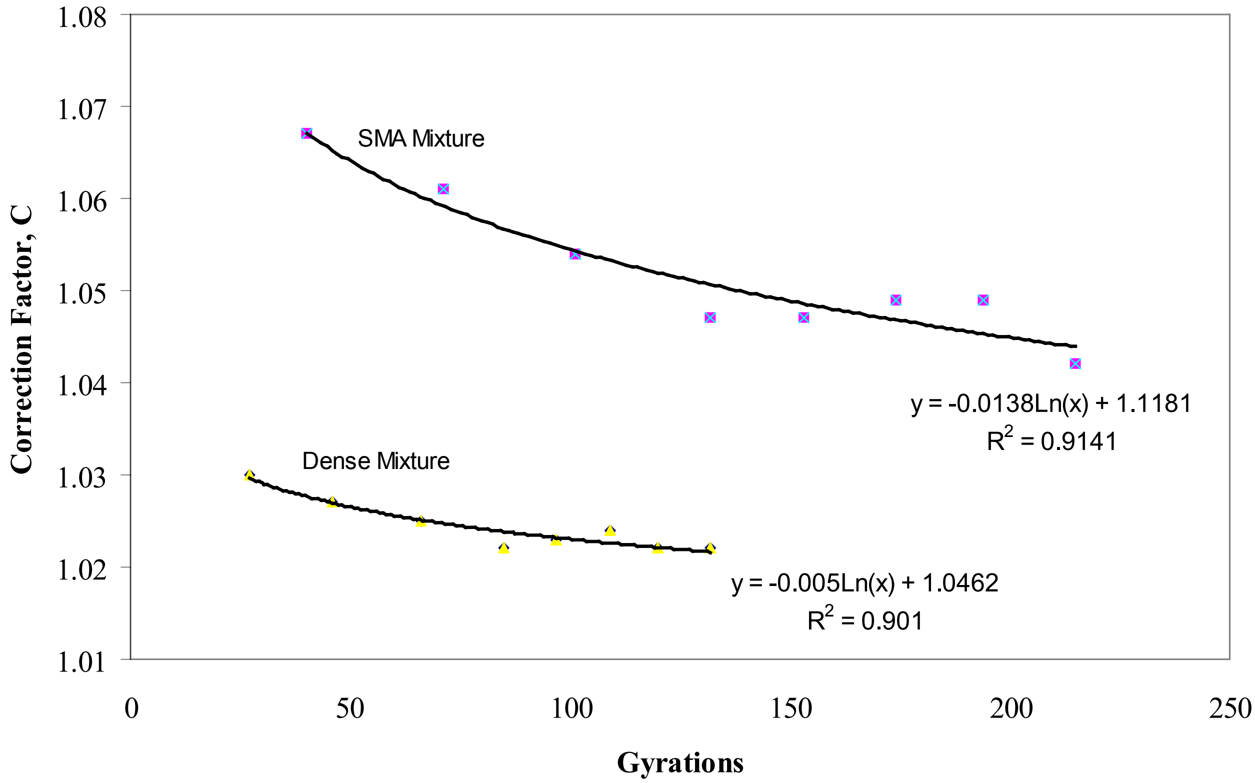

FIGURE 4 Relationship of the Correction Factor versus Gyration Level (8)

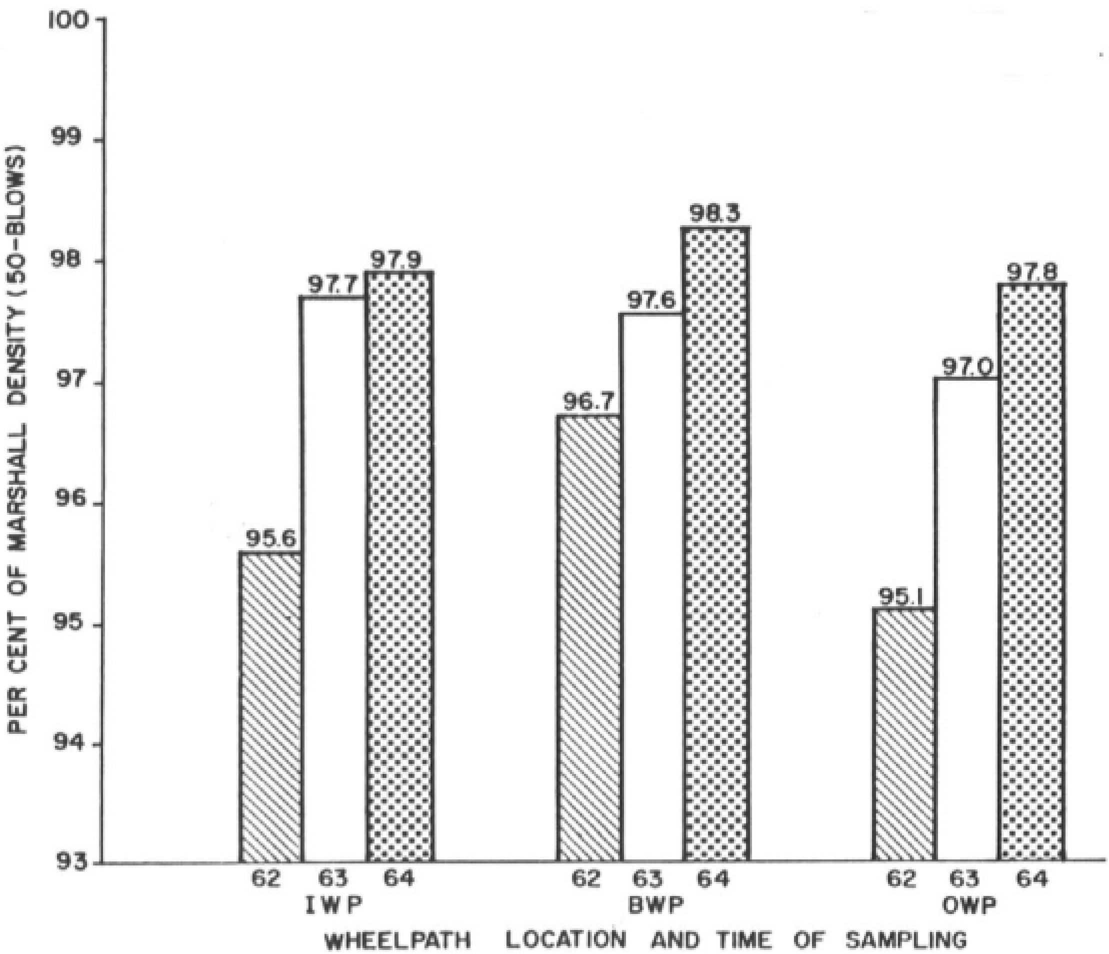

FIGURE 5 In-Place Density with Time for Different Wheelpaths (21)

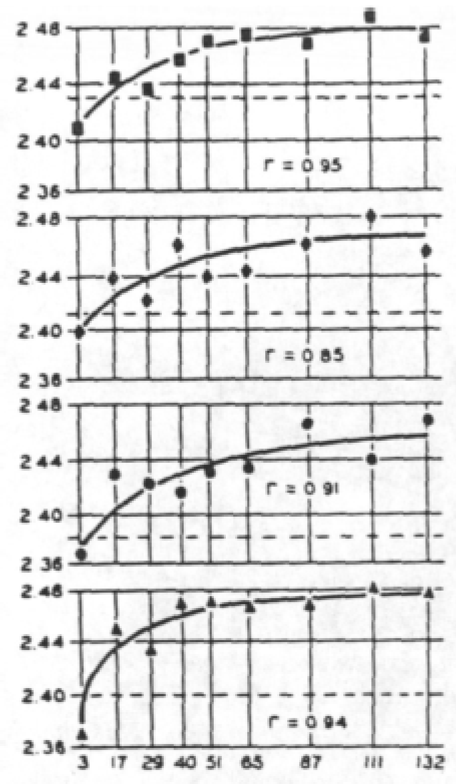

FIGURE 6 Core Bulk Specific Gravity versus Time (Traffic) (23)

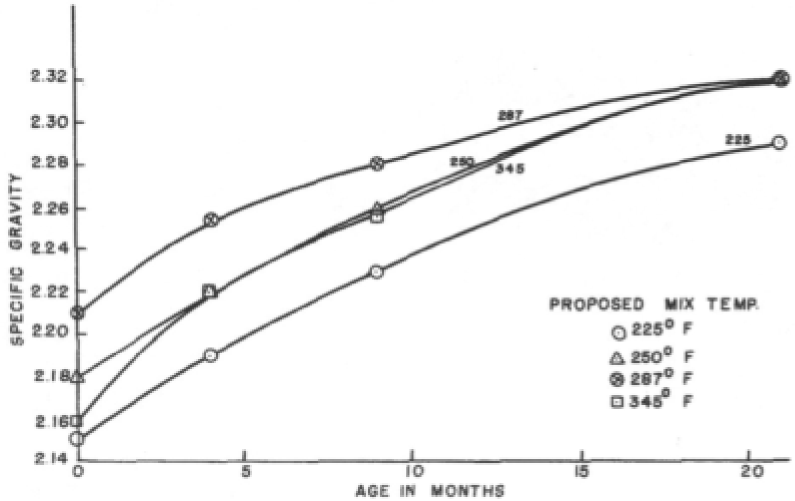

FIGURE 7 In-Place Densification with Time for Different Placement Temperatures (25)

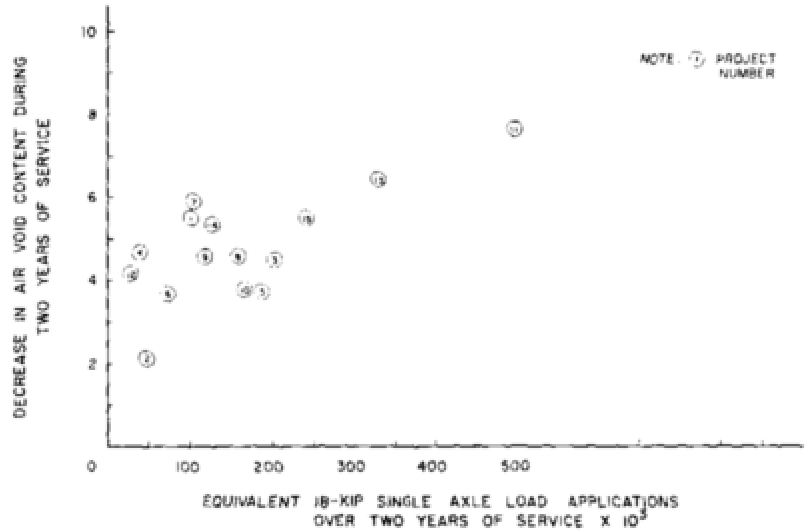

FIGURE 8 Effect of Traffic on In-Place Air Voids (27)

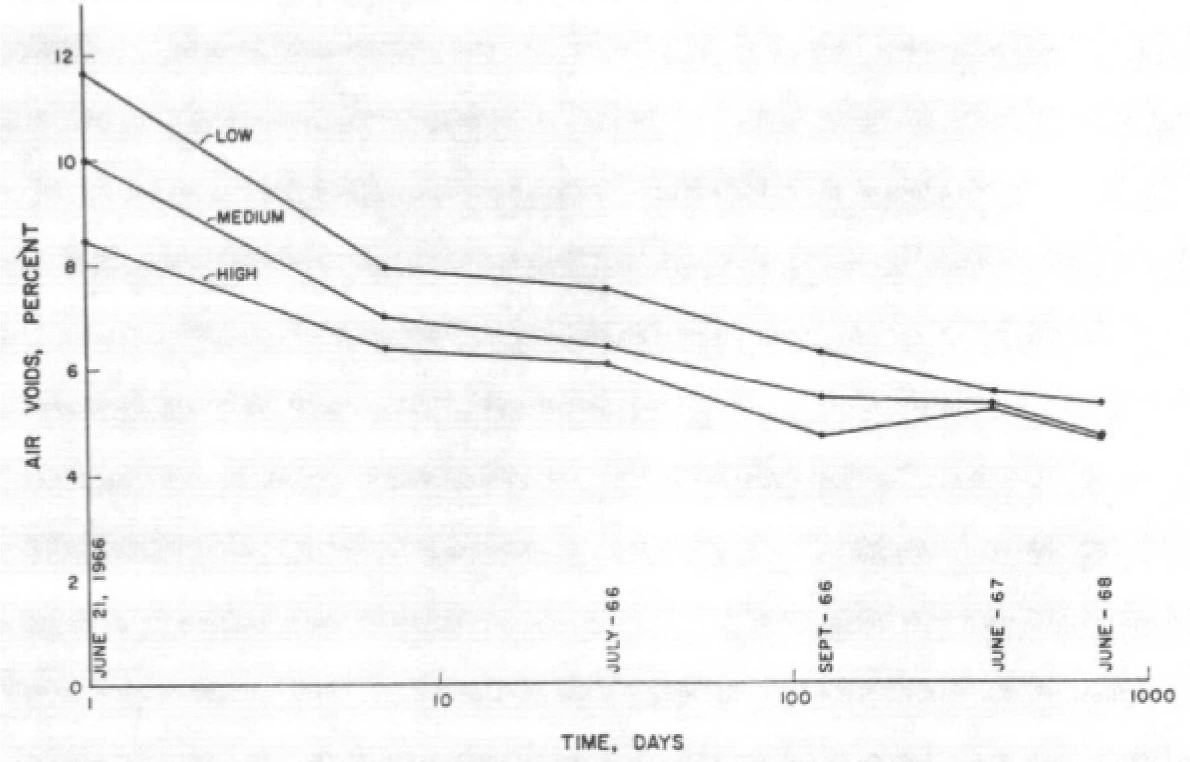

FIGURE 9 Densification for Low, Medium, and High Initial Compactive Efforts (27)

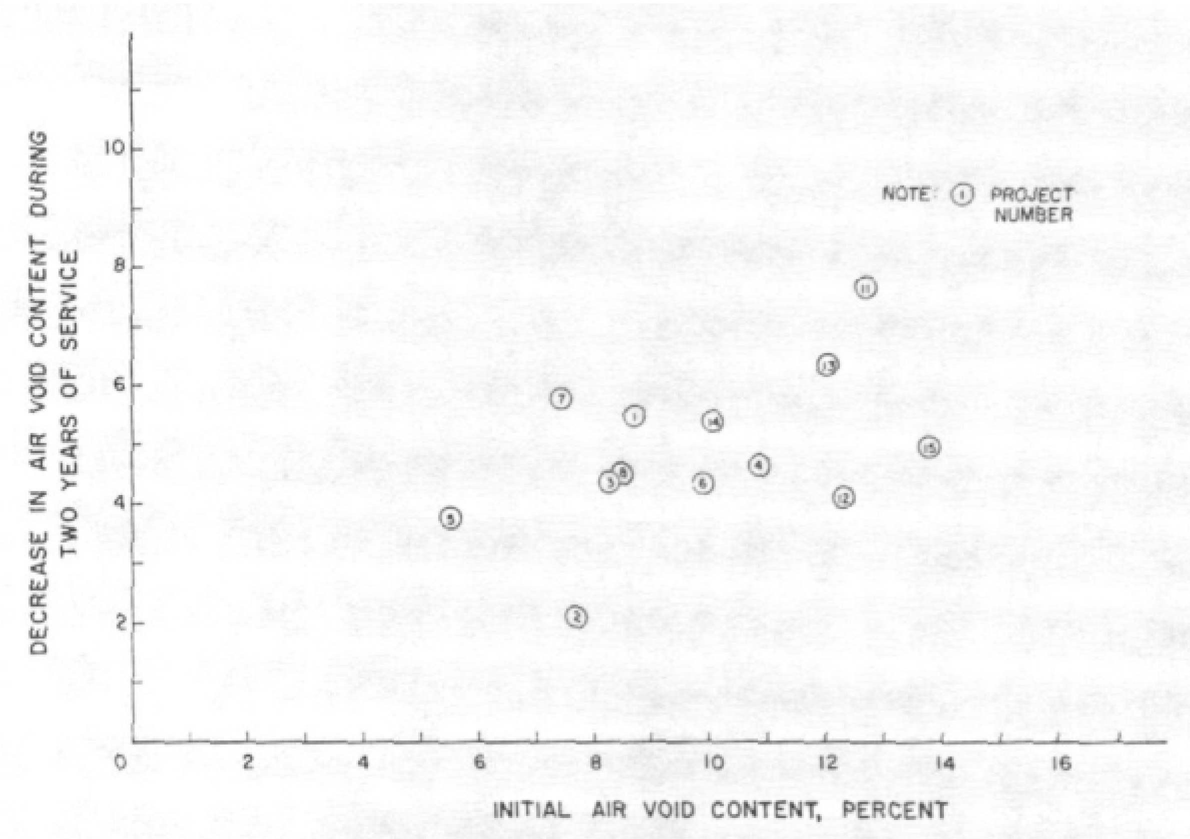

FIGURE 10 Effect of Initial Compaction Level on Air Voids for All Projects (27)

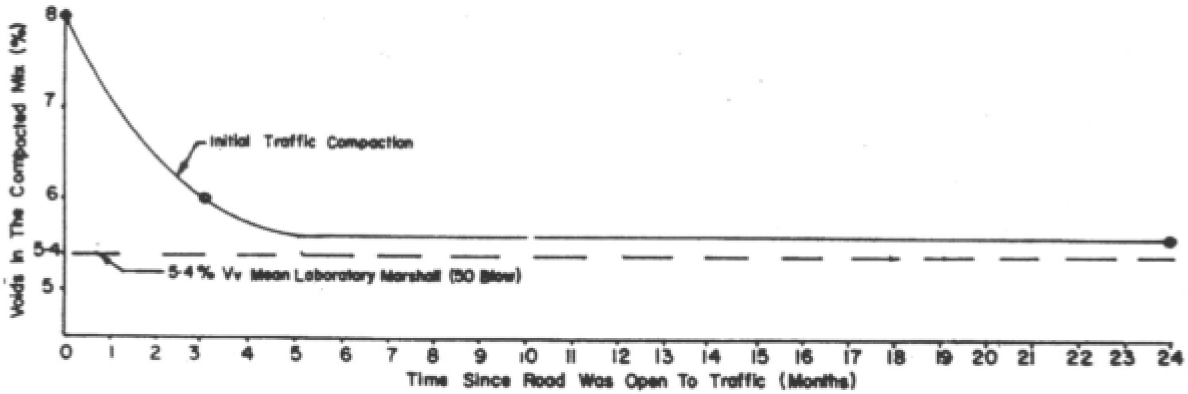

FIGURE 11 Effect of Traffic on Initial Densification (29)

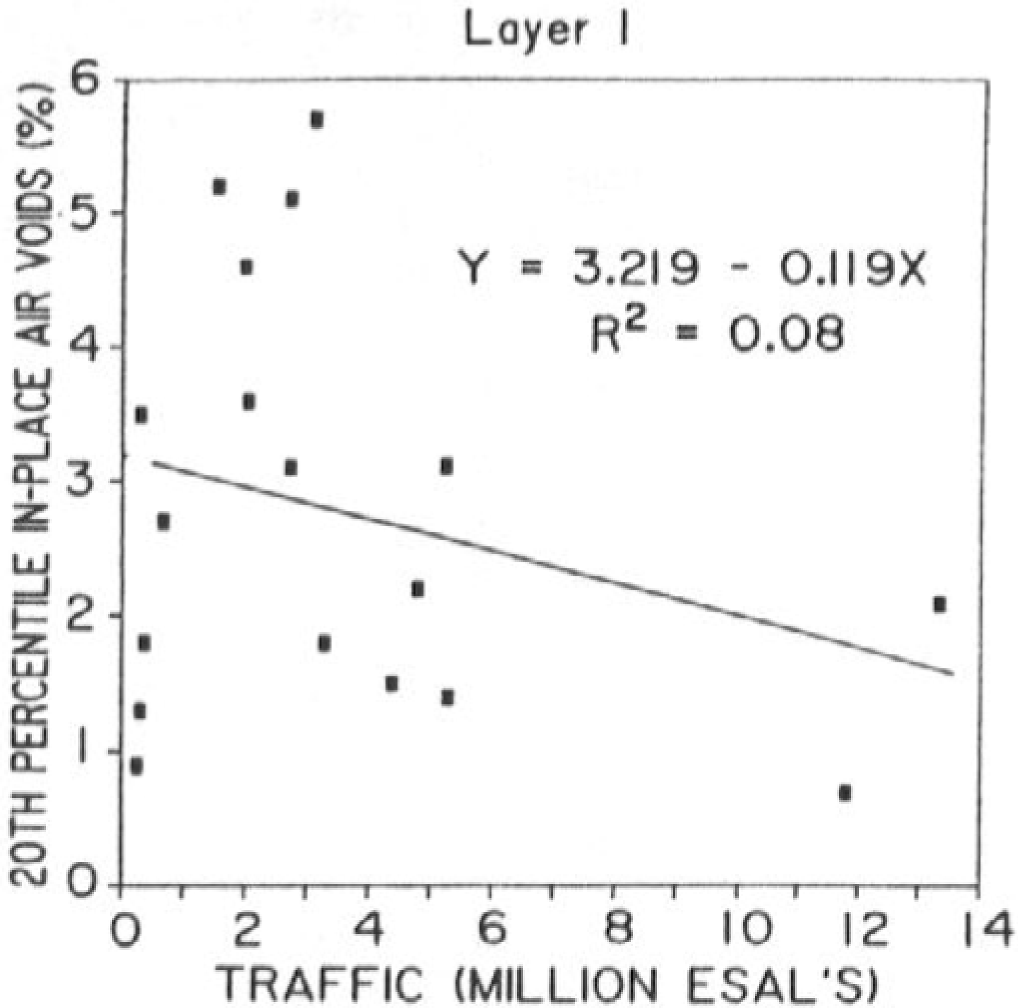

FIGURE 12 Air Voids versus Traffic (32)

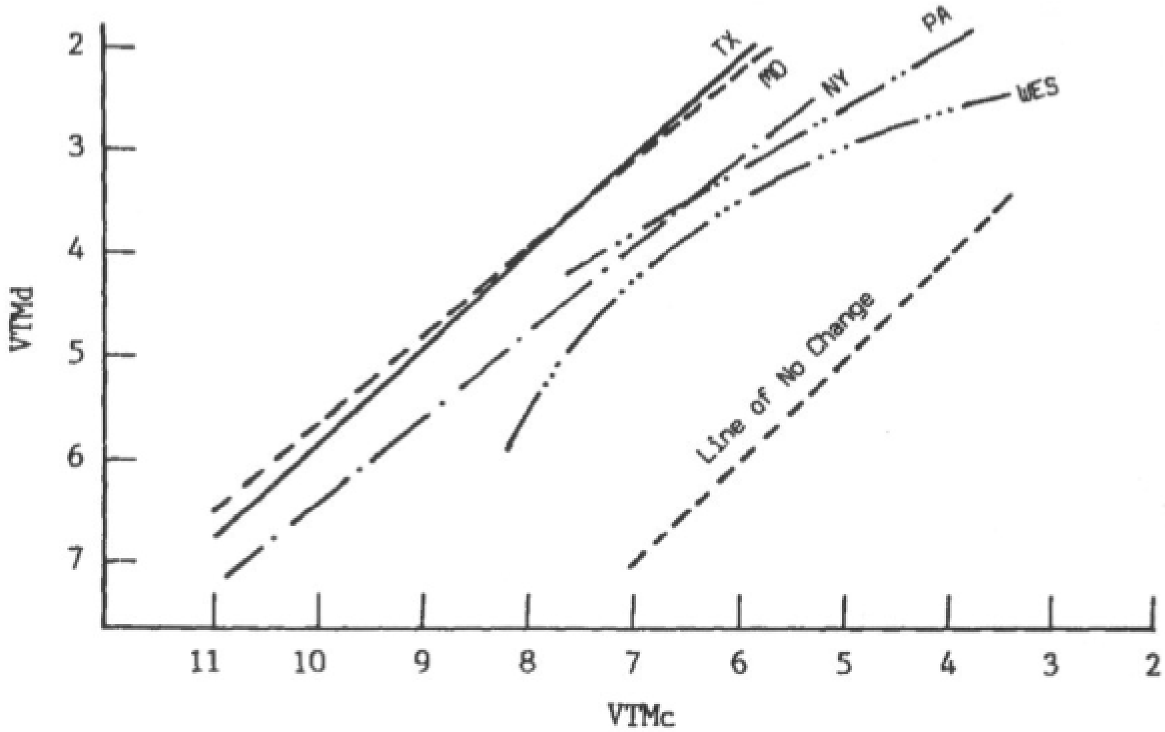

FIGURE 13 Relationship between Design and In-place Air Voids (34)

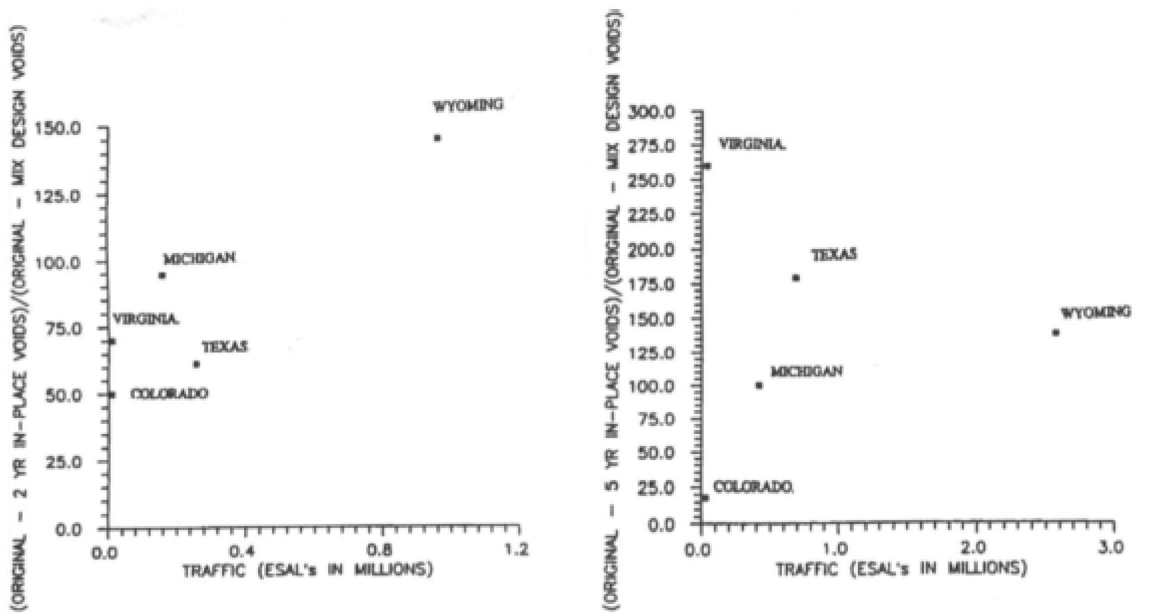

FIGURE 14 Densification for AAMAS Projects after Two and Five Years of Traffic (35)

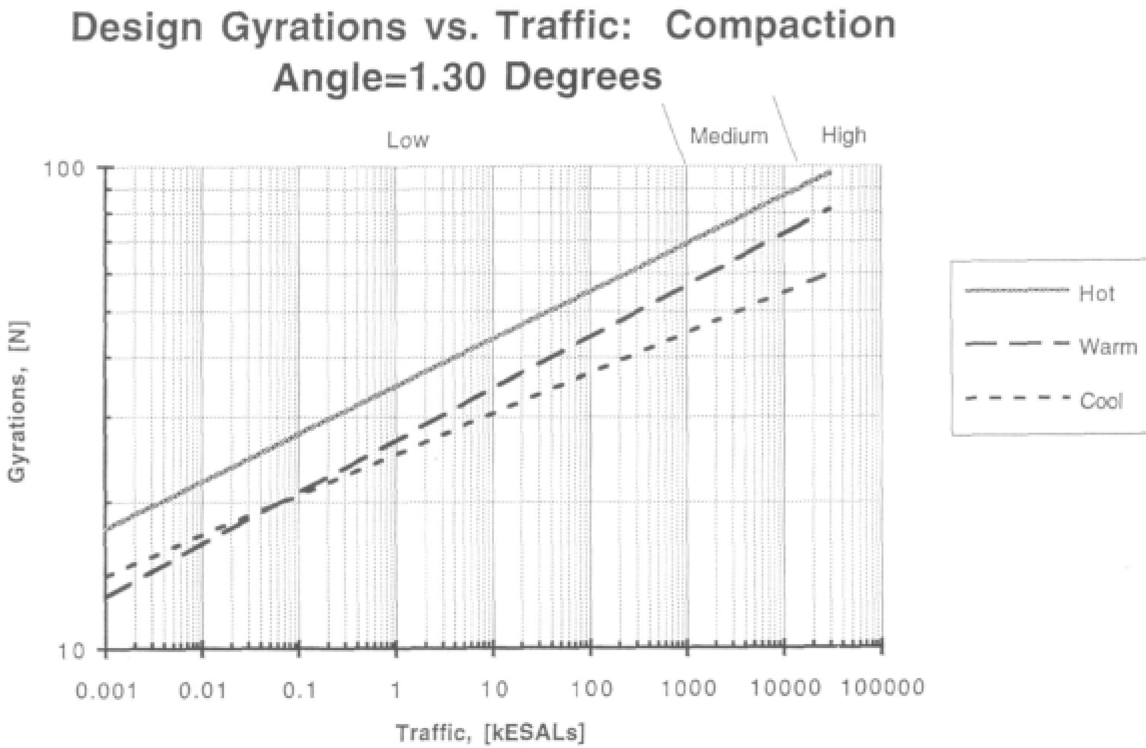

FIGURE 15 N_{design} versus Traffic for a Gyration Angle = 1.3 Degree (*37*)

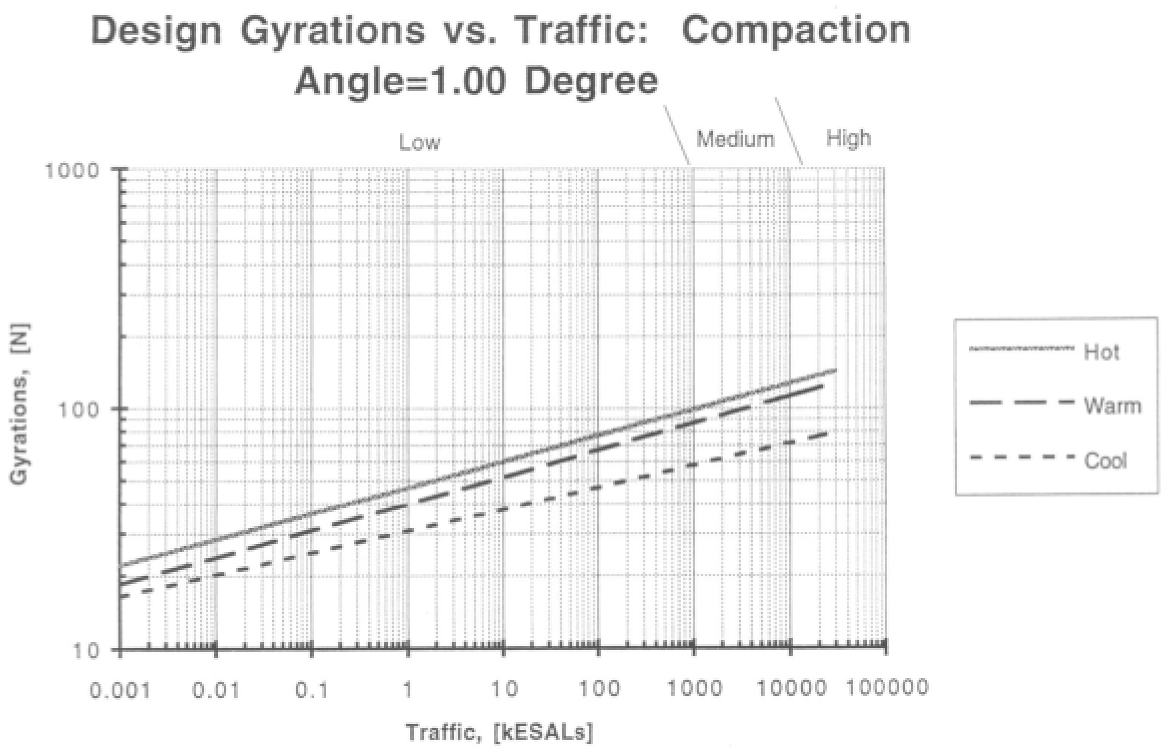

FIGURE 16 N_{design} versus Traffic for a Gyration Angle = 1.0 Degree (37)

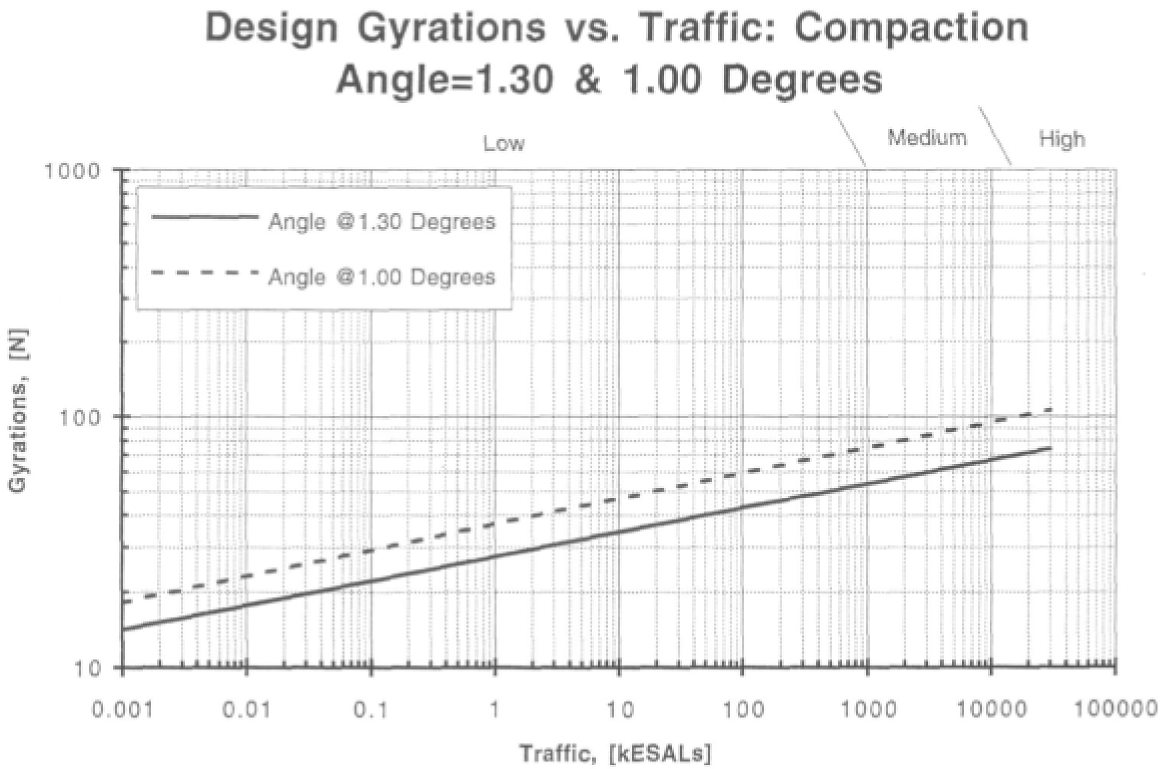

FIGURE 17 Average N_{design} versus Traffic for Gyration Angles of 1.3 and 1.0 Degrees (Hot and Warm Climates Only) (37)

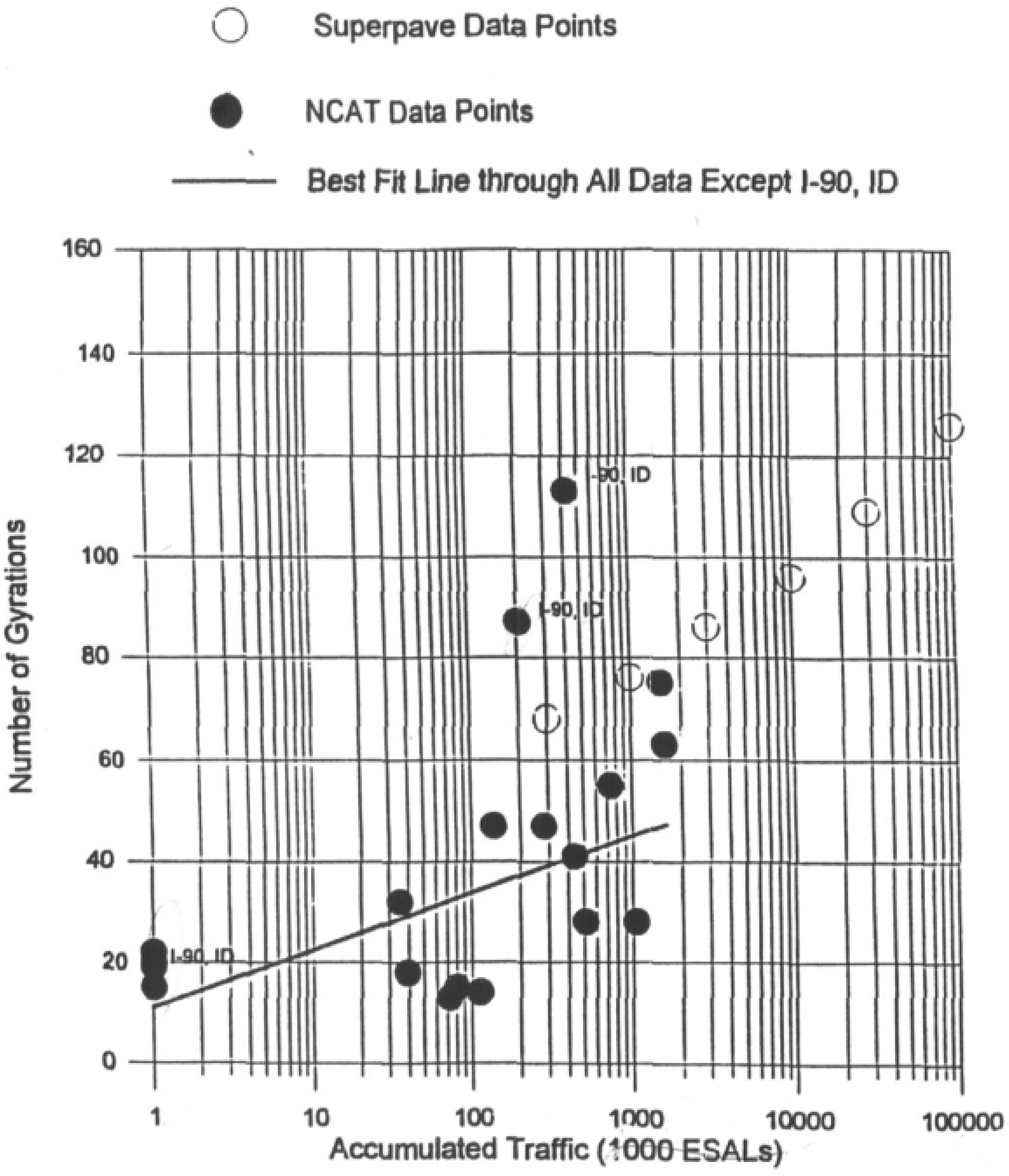

FIGURE 18 N_{design} versus Traffic (ESALs) (39)