VERIFICATION OF THE SUPERPAVE GYRATORY

Ndesign COMPACTION LEVELS

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VERIFICATION OF THE SUPERPAVE GYRATORY

Ndesign COMPACTION LEVELS

Brian Douglas Prowell

A Dissertation

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VERIFICATION OF THE SUPERPAVE GYRATORY

Ndesign COMPACTION LEVELS

Brian Douglas Prowell

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Signature of Author

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Date of Graduation

VITA

Brian Douglas Prowell, son of Harry Douglas and Judy Prowell was born February 12, 1967 in Harrisburg, Pennsylvania. He graduated from Mechanicsburg Area Senior High School in May 1985. He entered George Washington University in August 1985. After a rocky start where he found a greater passion for rowing than for engineering, he entered the Pennsylvania State University in 1987, and graduated with a Bachelor of Science in Civil Engineering in August 1990. He then worked for Kidde Consultants as an engineering intern from August 1990 until December 1990. In January 1991, he entered Graduate School at Virginia Polytechnic Institute and State University, and graduated with a Master of Science degree in Civil Engineering in August 1992. In August 1992, he married Marcia Ann Votour, the daughter of Paul and Carol Votour. From August 1992 until May 1993 he was employed as an instructor in the Civil Engineering Department at Virginia Polytechnic Institute and State University. He was then employed as a research scientist and later senior research scientist by the Virginia Transportation Research Council, the research arm of the Virginia Department of Transportation from May 1993 until July 2001. He has been employed with the National Center for Asphalt Technology at Auburn University since August 2001 as a research engineer and later as an assistant director. Marcia and Brian have been blessed with two children since moving to Auburn, Alabama, Benjamin Thomas Prowell, born September 25, 2001, and Katherine Elizabeth Prowell, born May 13, 2004.

DISSERTATION ABSTRACT

VERIFICATION OF THE SUPERPAVE GYRATORY

Ndesign COMPACTION LEVELS

Brian Douglas Prowell

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The original Superpave Ndesign table contained 28 levels based on a limited laboratory experiment. This was later consolidated to 4 levels based on the sensitivity of mixture volumetric properties to Ndesign; however, these data were not verified as being correct for field conditions. An experiment was conducted to verify the Ndesign levels in the field. Samples were collected, tested and analyzed from 40 field projects. The projects were selected in a total of 16 states. The projects represent a wide range of traffic levels, binder grades, aggregate types, and gradations. Each project was visited at the time of construction and at 5 additional times after construction. The 40 pavements

studied in this project appeared to reach their ultimate density after two years of traffic. A fair relationship was determined between the as-constructed and the density after two years of traffic. The high temperature PG binder grade was found to significantly affect pavement densification, with stiffer binders resulting in less densification. The ultimate in-place densities of the pavements evaluated in this study were approximately 1.5 percent less than the densities of the laboratory compacted samples at the agency specified Ndesign.

The number of gyrations to match the ultimate in-place density was calculated for each project in this study. The calculated values for the two compactors used in this study differed by approximately 20 gyrations. This was attributed to differences in their dynamic internal angle. The predicted gyrations, adjusted to a dynamic internal angle of 1.16 degrees showed good agreement between the two machines.

A relationship was developed between predicted Ndesign and design traffic for the projects which were not constructed using PG 76-22. Although there was a great deal of scatter in the data, the scatter was expected. The predicted gyration levels were generally less than those currently specified.

All of the projects in this study were very rut resistant. The maximum observed rutting for the field projects was 7.4 mm with an average rut depth for all of the projects of 2.7 mm after 4 years of traffic.

Based on the densification and performance data the Ndesign levels can be reduced for higher traffic levels and the Ninitial and Nmaximum criteria can be eliminated.

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CHAPTER 1 INTRODUCTION AND RESEARCH APPROACH

1.1 BACKGROUND

The Superpave mix design system, a product of the Strategic Highway Research Program (SHRP), was released in 1994. The Superpave mix design system for hot mix asphalt (HMA) includes: binder specifications, aggregate property specifications, design gradation ranges, a laboratory compaction procedure, specifications for volumetric properties and an evaluation of moisture sensitivity. These specifications are to act in concert to provide a system of checks and balances to ensure the resulting HMA is durable and rut resistant. Durability would include such performance parameters as resistance to low temperature and age related cracking, resistance to raveling or other surface wear and resistance to moisture damage. Rut resistance refers to resistance to permanent deformation resulting from shear flow of the hot mix asphalt; permanent deformation or rutting of the subgrade due to insufficient pavement structure is not included. The Superpave Mix Design System was designed to account for differing traffic and environmental conditions.

Central to the Superpave mix design system is the Superpave gyratory compactor (SGC). The SGC is used to compact trial HMA mixtures to a design number of gyrations in the laboratory in order to allow an evaluation of the volumetric properties of the compacted sample. The volumetric properties evaluated include: air voids, voids in mineral aggregate (VMA), voids filled with asphalt (VFA) and dust to effective binder content. Two additional parameters are included to examine the rate of densification: density at an initial number of gyrations (Ninitial) and density at a maximum number of gyrations (Nmax). The laboratory design air content is supposed to be related to the ultimate field density of the HMA.

Ultimately, the overall performance of an HMA pavement is highly dependent on the pavement structure and the construction quality. The pavement structure is evaluated in the pavement thickness design procedure, a separate topic. The ability to construct the HMA pavement layers should be, as much as possible, considered in the mix design procedure. The purpose of this research was to verify the relationship between laboratory testing and field performance with regards to the SGC, and, where needed, to provide alternative recommendations.

1.2 RESEARCH PROBLEM STATEMENT

When the Superpave mix design system was initially released in 1994, it included 28 different design gyration (Ndesign) levels for the SGC, representing seven traffic levels for each of four climates (*1*). Traffic levels were represented by equivalent 18-kip single axle loads (ESAL) accumulated during a 20-year design life. Differing climates were represented by the average 7-day high air temperature for the project site. Ndesign increased as either design ESAL or high air temperature increased.

In 1999, the Federal Highway Administration Superpave Mixture Expert Task Group recommended a consolidation of the original 28 Ndesign levels to 4 Ndesign levels (the author was present at this meeting). This consolidation was primarily based on research conducted in two studies (*2,3*). The consolidation eliminated differing

Ndesign levels for differing climates and reduced the design traffic to 5 ranges, 2 of which utilize the same Ndesign level. One of the studies did not address the magnitude of the Ndesign levels with respect to field performance, but rather differences in the gyration levels which resulted in significant differences in the resulting volumetric properties (*2*). The other study was based on the performance of a limited number of field sections (*3*). The American Association of State Highway and Transportation Officials (AASHTO) adopted the recommended changes to the SGC compaction procedure of the Superpave mix design procedure in 2000 (*4*).

There is still concern that the current Ndesign levels do not maximize field performance. The optimum asphalt content for a given blend of materials is selected at 4 percent air voids, based on laboratory samples compacted to Ndesign, assuming the resulting mixture meets the other criteria of the Superpave mix design system. The asphalt content of HMA is critical to its performance, too much asphalt and the mixture is likely to suffer excessive permanent deformation. Too little asphalt and it maybe difficult to achieve field compaction; and the pavement may develop premature cracking, raveling and/or other distresses related to durability. The locking point concept has been proposed as an alternative to Ndesign. The locking point is believed to represent the point where the aggregate skeleton "locks" together and further compaction results in aggregate degradation.

1.3 OBJECTIVE

The three objectives of this research were 1) to evaluate the field densification of pavements designed using the Superpave mix design system, 2) to verify or determine the

Ndesign levels to optimize field performance, and 3) to evaluate the locking point concept.

1.4 SCOPE

This study included a literature search and extensive laboratory and field testing. Samples were collected, tested and analyzed from 40 field projects at the time of construction. The projects were selected in a total of 16 states. The projects represent a wide range of traffic levels, binder grades, aggregate types, and gradations. Each project was visited at 5 time intervals after construction: 3 months, 6 months, one year, two years and four years. Coring and distress surveys were conducted at each evaluation interval. In total, approximately 4,085 SGC samples and 5,670 cores were tested. Data obtained from the SGC samples and field cores, as well as traffic data provided by the agencies were analyzed to provide recommendations for the Ndesign compaction levels and use of the locking point as an alternative to Ndesign.

CHAPTER 2 LITERATURE REVIEW

Literature was reviewed for this study related to the history of HMA design, the densification of HMA pavements, gyratory compaction, Ndesign and the locking point concept.

2.1 A BRIEF HISTORY OF HMA MIX DESIGN PRIOR TO SUPERPAVE

2.1.1 Proprietary Mixes

The first asphalt pavement constructed in the United States (U. S.) was built in Newark, New Jersey in 1870 (*5, 6*). This pavement was constructed with asphalt binder and rock asphalt imported from Europe (*6*). In 1876, President Grant appointed a commission of the U. S. Army Engineers to recommend paving materials for Washington, D. C (*5*). Based on this study, the first "sheet asphalt" pavement was constructed later that same year on Pennsylvania Avenue using Trinidad Lake Asphalt, clean sand and mineral filler (*6*). Amzi Alonzo Barber purchased the rights to collect and remove Trinidad Lake Asphalt. Barber was awarded a portion of the Washington, D. C. paving contracts. In 1883, he formed the Barber Asphalt and Paving Company. E. B. Warren was one of the founders of the Barber Asphalt Company, which was engaged in the import of Trinidad Lake Asphalt. Captain Francis V. Greene was an Assistant

Engineer in charge of paving Washington, D. C. He later joined and became president of the Barber Paving Company. Barber-Greene was one of the early manufacturers of paving equipment (*5*).

Two other paving companies were organized by members of the Warren family: Warren-Scharf Paving Company (1884) and The Warren Chemical and Manufacturing Company. In 1899, the Barber Asphalt Company, the Warren Chemical and Manufacturing Company and the Warren-Scharf Paving Company all merged. Hveem (*5*) referred to this group as the "Asphalt Trust". The remaining independent, National Asphalt Company, was brought into the group as the General Asphalt Company of America. Barber eventually withdrew from the trust to establish the A. L. Barber Company, which maneuvered to secure the rights to Bermudez Lake Asphalt, another natural asphalt source found in Venezuela.

Until the beginning of the 20th century, there is little evidence of design procedures or standardized tests. The asphalt "trust" mainly produced sheet asphalt using fluxed Trinidad Lake Asphalt. In 1905, the first textbook on asphalt pavements was published by Clifford Richardson (*5, 6*). Mr. Richardson, a chemist by training, began his career with the U. S. Department of Agriculture. He then became engineer inspector for the District of Columbia and later was employed by the Barber Asphalt Paving Company (*7*). Richardson proposed the following specification for sheet asphalt (*8*):

1. Asphalt penetration of 30 to 90 (0.1 mm) at 78°F for the surface course and 20 units higher for the binder or leveling course.
2. The mixture consist of refined natural asphalt, fluxed to the above consistency, sand of an appropriate grading, and mineral filler such as

rock dust or Portland cement. In this case refinement refers to the removal of water and excess organic matter.

3. The sand has 100 percent passing the No. 10 screen, at least 15 percent passing the No. 80 sieve and at least 7 percent passing the No. 100 screen. The sand contains less than 1 percent clay. The sand is to be mixed with 9.5 to 12.0 percent asphalt.

The penetration test was a recent invention, prior to which time asphalt consistency was evaluated by chewing. H. C. Bowen of the Barber Asphalt Paving Company invented the Bowen Penetration Machine in 1888. A. W. Dow, an inspector for the District of Columbia, designed another version of the penetrometer in 1903. Dow also invented the ductility test. Aggregate gradations, the penetration test for asphalt consistency and asphalt content determination by extraction using carbon disulfide made up the early asphalt tests (*5, 8*). The one test Richardson mentions to aid in the determination of optimum asphalt content is the Pat Test. The Pat Test consisted of a visual examination of a piece of Manila paper which had been pressed against a sample of HMA. A light stain indicated too little binder; a heavy stain indicated too much binder; and a medium stain indicated the optimum asphalt content (*9*).

The first HMA, which incorporated coarse aggregate, originated in 1901 with a patent application by Frederick J. Warren for "Bitulithic" pavement. A second patent was issued in 1903. Bithulithic pavements used tightly specified dense gradations with a maximum aggregate size of up to 3 inches. The large aggregate size tended to result in low asphalt contents, as compared to sheet asphalt. Also, the dense gradation allowed the use of softer asphalt cement resulting from the refinement of petroleum oil, mainly from

California, termed oil asphalt (*5, 6, 10*). A patent for “Warrenite” pavement, which incorporated a thin layer of sheet asphalt laid on top of hot Bitulithic pavement soon followed (*5, 9*). The sheet asphalt tended to prevent the steel rimmed wheels of the day from fracturing the large coarse aggregate particles found in the Bitulithic pavement, and allowing water to enter the pavement. Since the sheet asphalt was placed in a thin layer, it was not as prone to rutting as pavements constructed solely of sheet asphalt.

The City of Topeka, Kansas developed a mix consisting of sheet asphalt with a limited amount of ½ inch coarse aggregate added in an attempt to avoid paying royalties on the Warren Brothers patents. This mix became known as the “Topeka” mix. In 1912, The Warren Brothers filed suit against the City of Topeka for patent infringement. The federal court in Topeka, Kansas ruled that it was possible to construct an asphalt pavement that did not infringe on the Warren Brother’s patents if the nominal maximum aggregate size was less than ½ inch (*5, 6, 10*). Davis (*10*) credits this ruling for the predominance of small (less that ½ inch) top size aggregate surface mixes used today.

From 1900 until the early 1920’s the majority of the asphalt pavements constructed were constructed with one form or another of proprietary HMA. Davis (*10*) notes, that there was little incentive for the companies, such as the Warren Brothers, to explain their design procedures. From 1920 until 1940, the use of HMA pavements continued to grow. During this period pavements were typically designed with one of four techniques (*6*):

1. Sheet asphalt produced by Richardson’s or similar procedures,
2. Bitulithic, Warrenite or one of the other HMA mixes patented or trademarked by the Warren Brothers,

3. The Skidmore method which was similar to the Warren Brother's mixes, but had the addition of mineral filler to fill voids, or
4. The Hubbard-Field Method developed by Prevost Hubbard and Frederick Field (described below).

2.1.2 Hubbard-Field

Prevost Hubbard and Frederick Field developed a mix design method for the fine fraction (100 percent passing the No. 10 screen) of sheet asphalt and sand base mixes. The maximum load required to force a 2 inch diameter by 1 inch tall compacted sample through a 1.75-inch diameter orifice was plotted as a function of asphalt content. The maximum load was termed a "stability" value. The method was reportedly still in use by several states in the 1970s (*5, 6, 9, 11*).

From the late 1930's through approximately 1960, the modern philosophies of HMA mix design were developed, including: Hveem, Marshall, Texas Gyratory, and Corp of Engineers Gyratory Testing Machine.

2.1.3 Hveem Method

Francis N. Hveem was first exposed to asphalt as a young employee of the California Division of Highway. In 1927 he oversaw his first oil-mix job. Oil-mixes were road oil, slow curing cutback asphalt, mixed with gravel using a grader and rolled. Shortly thereafter, Hveem transferred to the Central Laboratory in Sacremento, California. By 1929, Hveem observed that coarser gradations tended to require less road oil than finer gradations and made the connection that the surface area of the aggregate varied with gradation. Hveem identified a method for calculating (estimating) the surface area of aggregate developed by a Canadian engineer, Captain L. N. Edwards for Portland cement concrete mixes (*5, 12*). Hveem realized that in addition to surface area, the optimum asphalt content, or at least the point where the optimum asphalt content was exceeded and stability decreased was affected by the surface texture of the aggregate. A "surface factor" was used by Hveem in combination with the calculated surface area to determine the optimum asphalt content. Although an experienced engineer could adjust for texture and absorption of various aggregates, Hveem later developed the centrifuge kerosene equivalent (CKE) test to estimate the surface constant (a combination of surface area, absorption and adjustment for surface texture) of the fine aggregate. A 100 g sample of the fine aggregate (100 percent passing the No. 4 sieve) was saturated in kerosene. The sample was then subjected to 400 times gravity in a centrifuge (*13*) [later this was reduced to 200 times gravity (*11*)], after which the aggregate was weighed to determine the percent of kerosene retained by mass of dry aggregate. If the fine aggregate type was similar to the coarse aggregate, then the bitumen index or the quantity of asphalt required to coat one unit of the area of aggregate could be determined directly from the CKE test; otherwise a separate test could be performed to determine the surface factor of the coarse aggregate (*13*). The coarse aggregate absorption test was performed by soaking a sample of the coarse aggregate in S. A. E. 10 oil for five minutes, and then allowing the sample to drain for 15 minutes at 140°F before determining the percent of retained oil. The coarse aggregate surface factor was used to correct the fine aggregate surface factor. These procedures, either the surface area calculation or the surface factors could be used to estimate optimum binder content. Correction factors were also included for aggregate specific gravity and the viscosity of the asphalt. Hveem did observe that a smaller film thickness of asphalt was required for smaller particles than for larger particles. Hveem stated that the CKE method indicated the optimum asphalt content in 95 percent of cases (*5, 13*).

Hveem also wanted to evaluate the stability of the HMA. He hypothesized that depending on the roughness and angularity of the aggregate, the film thickness at which the particles would become overly lubricated by the asphalt and therefore unstable would vary (*13*). Hveem was not satisfied with the Hubbard-Field method in use at that time. This led to the development of the first Hveem stabilometer in 1930. The stabilometer evolved into a hydraulic device into which a compacted sample of asphalt was loaded. The sample was loaded vertically on its flat surface and the radial force transmitted to the surrounding hydraulic cell is measured. The stability value is calculated according to Equation 1:

$$S = \frac{22.2}{\frac{P_h D_2}{(P_v - P_h)} + 0.222} \qquad (1)$$

where,

P_v = vertical pressure (400 psi),

P_h = horizontal pressure at a vertical pressure of 400 psi, and

D_2 = displacement of sample in number of turns of handle.

The use of the stabilometer required a compacted sample 4 inches in diameter and 2.5 inches tall. Initially an impact compaction method, consisting of an 8-lb hammer dropped 5 inches which applied blows to a 2-inch diameter tamper around the perimeter of the mold, was used. Vallerga and Lovering (*12*) state, "This method was used for several years, but when cores were cut from the pavement and the Stabilometer value compared with specimens of the same material compacted in the laboratory, it was found that the laboratory specimens invariably had a considerably higher stability." This led to the development of the kneading compactor which pneumatically loads a tamping foot with a cross section of one quarter of the mold area while rotating the mold 1/6 of a turn between each tamp. It was felt that the "kneading action produced by the foot (not covering the entire surface) would realign aggregate particles in a similar manner to a rubber tire roller or car.

The optimum asphalt content by the Hveem method was determined using a pyramid scheme. First, the asphalt contents for which moderate to heavy bleeding were observed on the surface of the compacted sample were eliminated. Next, any asphalt contents that failed the minimum stability value were eliminated. Finally, the highest asphalt content that had at least 4 percent air voids was selected as the optimum (*11*).

Vallerga and Lovering (*12*) quote Hveem's own summary of his mix design philosophy in 1937 as follows,

> "For the best stability, a harsh, crushed stone with some gradation, mixed with only sufficient asphalt to permit high compaction with the means available.
>
> For greatest resistance to abrasion, raveling, aging and deterioration, and imperviousness to water, a high asphalt content, broadly speaking, the richer the better.
>
> For impermeability, a uniformly graded mixture with a sufficient quantity of fine sand (fine sand is more important than filler dust).

For non-skid surfaces, a large quantity of the maximum sized aggregate within the size limits used.

For workability and freedom from segregation, a uniformly graded aggregate.

To reduce the above factors to as simple a consideration as possible, it seems to be the best rule to use a dense, uniformly graded mixture without an excess of dust and to add as much oil or asphalt as the mixture will tolerate without losing stability."

[Currently, we would describe "uniformly" graded as "well" or "dense" graded].

Graphically, this philosophy is summarized in Figure 2.1.

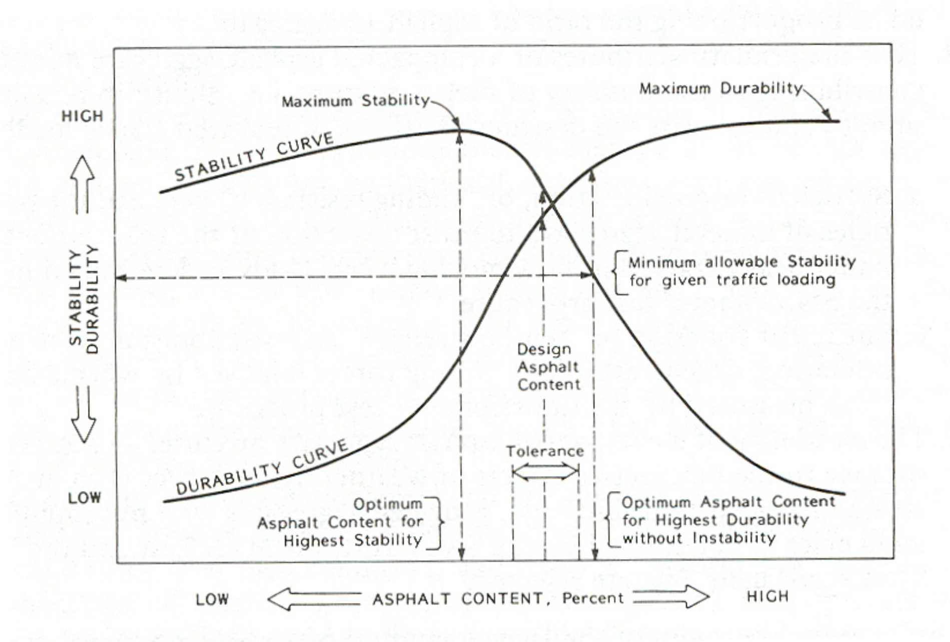

Figure 2.1. Stability and Durability as a Function of Asphalt Content (*12*).

2.1.4 Marshall Mix Design

Bruce G. Marshall began the development of what later became known as the Marshall mix design procedure around 1939 while employed by the Mississippi State

Highway Department (*11*). Marshall developed the stability test; flow measurements were added by the U. S. Army Corps of Engineers. Marshall was retained by the Corps during their studies (*6*). Initially, samples of HMA for the stability and flow tests were compacted with a modified American Association of Highway Officials (AASHO), California Bearing Ratio (CBR) field hammer. The modified AASHO hammer consisted of a 10 pound hammer (weight) dropped 18 inches; the load was transferred to the sample through a 1.95-inch diameter foot. Samples were compacted in a 4-inch diameter mold with a target compacted height of 2.5 inches. The initial compaction effort was 15 blows of the modified AASHO distributed across one face of the sample followed by a 5000 pound static load held for 2 minutes (*14*).

The Corps of Engineers was charged with selecting a method of HMA mix design to deal with the increasing tire pressures found on military aircraft. Aircraft weights began increasing during World War II. As the weight of the aircraft increased, tire pressures were also increased to minimize the size of the landing gear. At the beginning of World War II, tire pressures were approximately 100 psi. By the end of World War II, tire pressures had increased to approximately 200 psi. Currently, some military aircraft have tire pressures of 350 psi (*15*).

In a previous study, the Tulsa District of the U.S. Army Corps of Engineers recommended the Hubbard-Field method of HMA mix design. In 1943, the Waterways Experiment Station was charged with evaluating the Hubbard-Field method as well as a method utilizing the field CBR hammer (*14*). At this time the Marshall method had been used by some southern states for up to four years (*15*).

In the first phase of the study begun in 1943 (*14)*, comparisons were performed between the Hubbard-Field and Marshall mix design methods using a wide range of asphalt materials. From this study it was concluded that the Marshall Stability test gave comparable results to the Hubbard-Field stability test; further, the Hubbard-Field test was not readily adaptable to the field CBR equipment; and the Marshall apparatus was also more portable. Therefore, the Marshall method was selected for additional study to evaluate the following objectives (*14*):

1. For both sand asphalt and HMA evaluate the effect on test properties from:
    a. Aggregate gradation
    b. Type of filler
    c. Mixing temperature
    d. Penetration grade of asphalt cement
    e. Compactive effort.
2. Determine if there is a correlation between laboratory compaction and field compaction.
3. Determine the relationship between the Marshall method and the Hubbard-Field method.

The Marshall test properties selected for evaluation included stability and flow, total unit weight, aggregate unit weight, percent voids total mix, percent voids aggregate only (essentially voids in mineral aggregate) and percent voids filled with asphalt. In addition to evaluating asphalt mix design properties, the Corps were also charged with evaluating the required pavement thickness for three different wheel loads, 15,000 lb single, 37,000 lb single and 60,000 lb double on differing subgrade types.

Test sections were constructed to allow the laboratory properties to be compared with field performance. The test tracks were divided into 8 major sections to accommodate three mix types and three subgrade qualities. The three mix types were HMA, sand asphalt and double surface treatment. HMA sections utilized from both crushed limestone and uncrushed gravel coarse aggregate with a maximum particle size of ¾ inch. Siliceous sand from a river pit and from a Mississippi river sand bar were used for fine aggregate.

Three subgrade materials were used in the study: crushed limestone (high quality), sand-loess (medium quality) and sand-clay-loess (low quality) were used for the evaluation of the minimum required pavement thickness. Only the HMA produced with crushed limestone was placed on all three subgrade materials; the HMA produced with uncrushed gravel was only placed on the high quality crushed limestone subgrade. Each of the 8 sections, except the two double surface treatment sections, was further subdivided into three thicknesses, each 90 feet long. The total pavement thicknesses were 1 ½, 3, and 5 inches for the HMA and 2, 4, and 6 inches for the sand asphalt.

To evaluate the effect of filler on Marshall stability, each pavement thickness section was further subdivided into three 30 foot sections with three different levels of limestone mineral filler addition to the HMA or sand asphalt: none, some and high. Finally, at each level of mineral filler content, the HMA or sand asphalt was produced at three asphalt contents: that which produced the maximum stability using the previously described compaction procedure, and 10 and 20 percent below optimum. Previous experience with a test section in Marietta, Georgia indicated that the optimum asphalt content determined from the maximum stability value would be too rich (high in asphalt), leading to too low of in-place air voids under traffic. The sections for the different asphalt contents were 10 feet long. All of the main sections were produced with a 120-150 pen binder. By today's specifications, this is a very soft binder, probably softer than a PG 58-28. Additional studies, including the use of gap gradations were conducted in the turnarounds.

In total, the two straightaway sections were 850 feet long and 60 feet wide, allowing for a separate lane for each wheel load. It is interesting to note that the lanes were paved perpendicular to the direction of traffic. The ten foot width of the paving lane, which was 60 feet long, became the ten foot length of the test lane for a given wheel load.

Traffic loads were applied using a Model C Tournapull, essentially the engine and drive wheels of a modern scraper or pan. A 12-cubic yard scraper was loaded to provide 15,000 lbs load on each of its two wheels. This setup was used to provide 3500 coverages across an approximately 12-foot lane width with the 15,000 lb wheel load. A specially built cart was built to apply the 37,000 and 60,000-lb wheel loads. A single (for 37,000-lb load) or dual (for the 60,000-lb load) 56-in diameter wheel was mounted in the center of the cart (Figure 2.2). The load was applied to a 4-foot or 6-foot lane width for the 37,000 lb or 60,000 lb load, respectively. The cart had two additional wheels which were loaded to 10,000 lb each, but these as well as the Tournpull drive wheels (loaded to 14,000 lbs) tracked outside the test lanes. A total of 1500 coverages were applied with the 37,000 and 60,000-lb wheel loads. The net tire contact pressures were 106, 146 and 139 psi for the 15,000, 37,000, and 60,000-lb wheel loads, respectively. Net pressures were used to account for the block nature of the tire tread. The majority of the coverages were applied in warm weather.

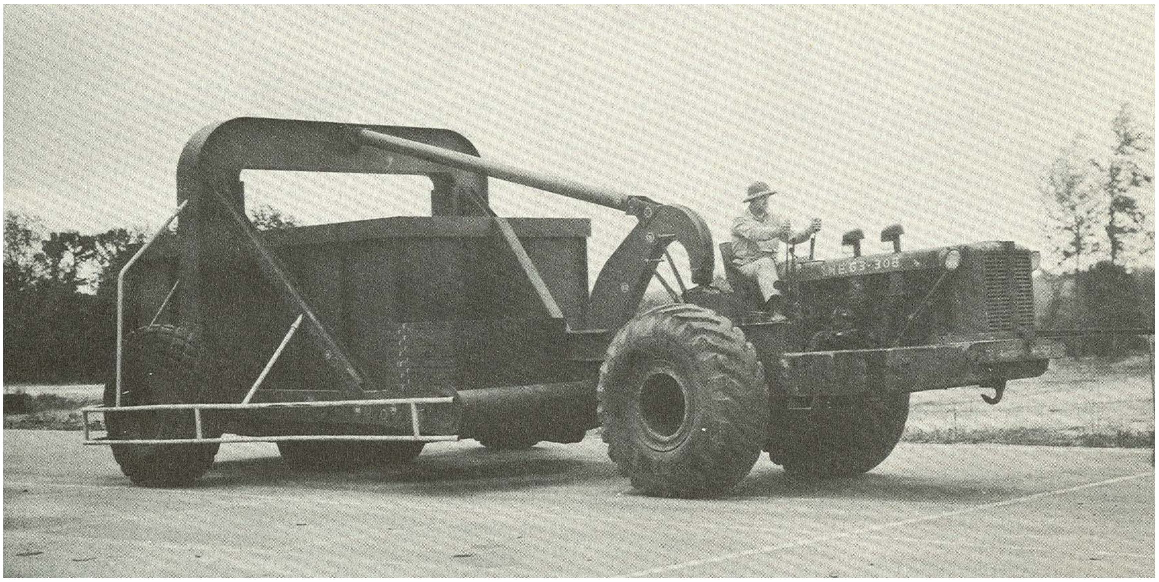

Figure 2.2. Model C Tournpull with Specially Built Loading Cart (*14*).

The performance of the test sections was monitored throughout trafficking by visual observations and coring. Visual observations included: tire printing (bleeding), rutting and shoving, cracking, settlement, roughness, upheaval and longitudinal movement. Four levels were used to quantify the observations: none, faint, well-defined and pronounced. The 4-inch diameter cores were tested for density and stability and flow.

The following is a summary of the conclusions from the Corps study which relate to this current study (*14*):

1. The test property relationships developed during construction and subsequent trafficking were similar to those developed from laboratory compaction.
2. There was an indication that the number of roller passes required to match the laboratory density varied with the mix type and asphalt content.

3. Aggregate gradation was believed to be of lesser importance than other factors in the design of good performing HMA.
4. In all cases, density increased with the application of wheel passes (Figure 2.3). Density increased rapidly at first, and then more slowly after the first few hundred passes. Regardless of initial, as-constructed, density, the densities of identical mixes subjected to three different wheel loads were nearly identical after 1500 passes.

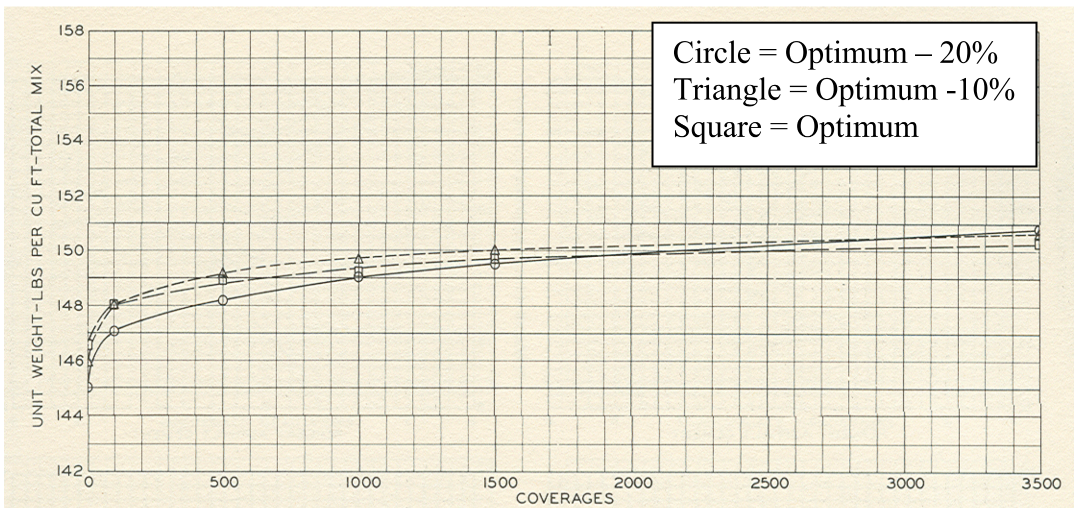

Figure 2.3. Traffic Compaction Data for Mix 11, Crushed Limestone with Medium Filler Content (*14*).

5. The range of asphalt content that produces satisfactory performance is approximately ± 1.0 percent.
6. The optimum asphalt content selected at 4 percent air voids and 80 percent VFA for HMA (6 percent air voids and 70 percent VFA for sand asphalt) was in reasonable agreement with those deemed acceptable based on the field test sections, but on the low end of the range.

7. The as-constructed density was approximately equivalent to the density obtained in the laboratory from the original compaction effort, 15 blows to a 1.95-inch diameter foot plus a 5,000 lb static load held for 2 minutes, as well as a modified compaction effort, 15 blows on each face with a 10-lb hammer falling 18 inches with a 3 7/8-inch diameter foot. This density was approximately 2 percent less than that obtained with 50 blows on each face with the modified compaction effort.
8. Tire pressure is more important than wheel-load in its effect on the performance of the pavement. No difference in performance was noted for net tire pressures ranging from 106 to 146 psi.

Additional studies were conducted to examine other compaction efforts that might account for the densification which occurred under traffic. From this effort, the familiar compaction effort, 50 blows to each face with a 12.5-lb hammer falling on a 3 7/8-inch diameter foot, was developed. This was later changed back to a 10 lb hammer. Five properties were selected for design: stability, flow, unit weight, air voids and VFA. Flow was only used as an evaluation of the plasticity of the mix (maximum value of 20). The optimum asphalt content from the remaining four parameters were averaged to determine the design asphalt content.

In summary the Corps of Engineers (*14*) note, "The results of this study indicate that the quantity of asphalt is the most important factor in a paving mixture. Where there is too much asphalt in the mix the resultant pavement will "flush" and the pavement will rut and shove under traffic. Too little asphalt produces a brittle pavement that will crack and ravel. From the standpoint of durability, it is desirable to include as much asphalt as possible."

As mentioned previously, aircraft tire inflation pressures continued to increase in the late 1940's and early 1950's. Tire pressures doubled from the approximately 100 psi net tire pressure used in the first field study to 200 psi. White reports (*15*), additional tests were conducted on the original test sections using both 30,000 lb wheel load with a 200 psi tire pressure and 15,000 lb wheel load with a 240 psi tire pressure. From these efforts it was determined that 69 blows from a 10-lb hammer falling 18 inches on a 3 7/8-inch diameter foot were appropriate for the increased tire pressures. This was later adjusted to the 75-blow Marshall.

McLeod (*16*) first suggested the concept of designing for minimum VMA to ensure durability in 1956. VMA is the total void space filled with either air or asphalt between the compacted mineral aggregate, which is believed to be related to durability. He argued that VMA and VFA should be calculated with the effective binder content and aggregate bulk specific gravity to avoid errors with absorptive aggregates (*16*). In 1957, McLeod reaffirmed his belief that the effective binder content and aggregate bulk specific gravity should be used to calculate the VMA and air voids of the compacted HMA sample (*17*). McLeod stated: "Values for percent voids in mineral aggregate and for percent air voids can be defined precisely for compacted bituminous paving mixtures that are made with non-absorptive aggregates." He added: "For compacted paving mixtures that contain absorptive aggregates, values for percent voids in the mineral aggregate and for percent air voids, should be calculated by means of (a) the ASTM bulk specific gravity of the aggregate, and (b) the effective bitumen content of the paving mixture."

McLeod's objections to the use of apparent and effective aggregate specific gravities (which are substantially easier to measure) result from their failure to differentiate between the portion of the binder that is coating the aggregate particle and the portion of the binder that is absorbed in the aggregate. Without this differentiation, it is difficult to relate observations from the laboratory design to field performance in terms of both permanent deformation and durability. In 1962, the Asphalt Institute published a new version of MS-2 that included the first "modern" version of the Marshall mix design procedure including volumetric analysis based on effective binder content (*18*).

Eventually, mechanical Marshall Hammers were developed to reduce the effort required by the operator to produce samples. These tended to produce less compactive effort than a hand-held hammer. This is attributed to the operator moving the handle during compaction, producing a slight kneading action (*19*). The Marshall mix design procedure was expanded to include 1 ½ inch maximum aggregate by developing a 6-inch diameter mold with a 75-blow compaction effort (*20*). By 1984, 38 out of 50 states were using the Marshall mix design procedure to design HMA.

Leahy and McGennis (*6*) provide a rare quote of Marshall's own mix design philosophy:

> "The ultimate result in the improvement of aggregate gradation is the reduction of the VMA. VMA should be reduced to the lowest practical degree. This reduction results in a superior pavement structure as well as to reduce the quantity of asphalt required in the mixture. No limits can be established for VMA, for universal application, because of the versatile application of bituminous materials to many types and gradations of aggregates."

2.1.5 Texas Gyratory Method

In 1939, the Texas Highway Department initiated a research program into the design and field control of HMA (*21*). The first goal of the research was to develop a means of compacting samples in the laboratory. The following criteria were listed for the laboratory compaction method:

1. Method must be adaptable to field control of HMA mixes.
2. The method should yield essentially the same density that is obtained in the finished pavement. Since pavements continue to densify under traffic, the laboratory density should approximately match the "ultimate" density after some time on the road, "and is the goal of any compaction method."
3. The aggregate breakdown that occurs during laboratory compaction should approximate the degradation that occurs in the field.

A number of compaction devices were evaluated. These methods applied shear to the surface of the sample. It was desirable to develop a method that applies shear throughout the sample while holding the faces of the sample, to which compressive forces are applied, parallel. The Texas Gyratory Molding Machine was developed from this effort. Using this device, Ortolani and Sandberg (*21*) state, "The aggregate is oriented into its most dense position by applying specimen shear at low initial pressures."

The original Texas Gyratory Molding Machine consists of two loading heads that are held parallel to one another. The lower loading head is connected to a 30 ton jack. The molding cylinder has two 24-inch handles attached at a 75-degree angle to one another (Figure 2.4). The handles are used to manually impart the gyratory action; a guide ring limits the mold's vertical movement to ½ inch. First a 50-lb compressive load is applied to the sample; then the handles are used to impart a gyratory action until 3 revolutions were completed. This is to be repeated until movement of the molding cylinder is extremely difficult. At this point, one stroke of the jack handle should increase the gauge pressure to 100 lbs. This indicates the sample has reached the proper degree of compaction.

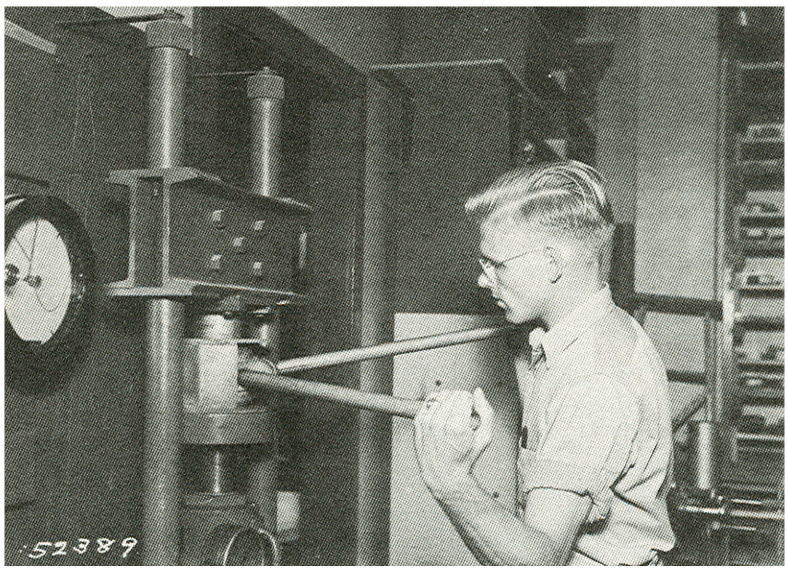

Figure 2.4. Manual Texas Gyratory Molding Machine (*21*).

In 1945, the Texas Highway Department took over 400 cores from around the state from pavements which were 1 to 12 years old in order to compare in-place pavement densities to those determined using the Texas Gyratory Molding Machine. In-place densities at the time of construction were also available; these averaged 3.8 percent less than the density of the samples compacted in the Texas Gyratory Molding Machine. The cores which were taken after 1 to 12 years of traffic averaged 0.8 percent less than the laboratory samples. There was variability in the data. One coarse-graded pavement's density was 3.3 percent less than the laboratory compacted samples after one year of traffic. Another base layer, approximately 3 inches deep in the pavement structure, was 2.3 percent less than the laboratory compacted samples (*21, 22*).

The Texas Gyratory Molding Machine was later automated. In 1974, the method was adopted as ASTM D 4013, "Standard Test Method for Preparation of Test Specimens of Bituminous Mixtures by Means of Gyratory Shear Compactor (*23*)." When using the Texas Gyratory Compactor, the number of gyrations is variable in groups of three gyrations applied at one gyration per second. First, a 50 psi vertical pressure, termed the gyration pressure, is applied to the sample. Next, the sample is gyrated three times at an angle of 6 degrees. At this point if one stroke of the hydraulic pump increases the vertical pressure to 150 psi, the gyrations are complete. Otherwise, the pressure is reduced to 50 psi and the sample is gyrated three more times. This process is repeated until one stroke of the hydraulic pump causes the vertical pressure to increase to 150 psi. Finally, the vertical pressure is increased to 2500 psi at the rate of one stroke per minute. This is termed the end pressure. Once 2500 psi is reached, the pressure is immediately released and the sample extruded (*24*).

2.1.6 Corps of Engineers Gyratory Compactor

McRae (*25*) presented the development of the Corps of Engineers Gyratory Compactor to simulate the in-place pavement densification which occurred under channelized high-pressure tire traffic. The goals of this research were to develop a compactor that could simulate in-place pavement density after traffic as well as produce laboratory samples with Marshall Stabilities similar to those obtained from cores.

Stabilities of samples compacted with the Marshall hammer tended to be higher than the stabilities of pavement cores of the same mixture tested at the same density. This was believed to be related to differences in the aggregate orientation.

The Corps of Engineers Gyratory Compactor was based on the Texas Gyratory Molding Machine, discussed previously. The gyratory action is provided mechanically by a pair of rollers riding on a flange connected to a sleeve surrounding the samples mold (Figure 2.5). The arm, to which the two rollers are affixed, is rotated by an electric motor. The initial angle of gyration can be adjusted using a thumb screw attached to the lower roller. The pressure of the upper roller is adjustable using an air over oil chamber. A hydraulic jack is used to provide a variable vertical pressure, up to 300 psi, on the sample. The combined action produces a "fixed-deformation variable

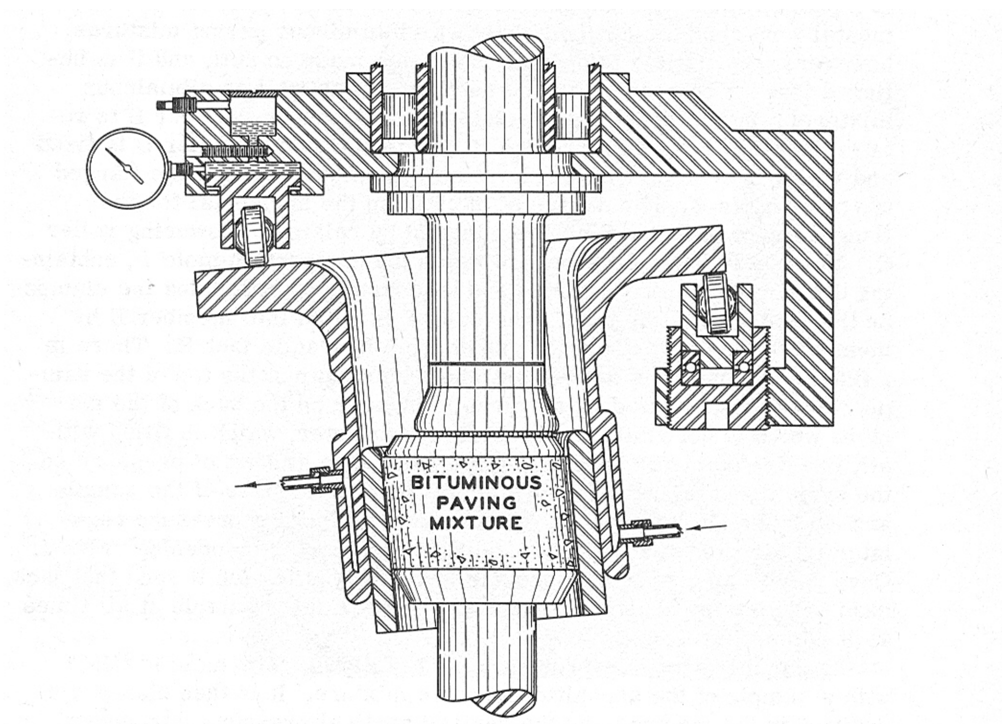

Figure 2.5. Schematic of Compaction Head for Corps of Engineers Gyratory Compactor (*25*).

stress" type compaction. The sample is compacted at a rate of five gyrations per minute. Later models included a heated jacket around the sample mold.

Figure 2.6 shows a comparison between the densities of samples compacted with varying laboratory compaction efforts with both the Marshall Hammer and Corps of Engineers Gyratory Compactor and field densities after varying levels of accelerated loading. The author notes that the as-constructed density was approximated by both the 50-blow Marshall and 5 gyrations with a 100 psi vertical load of the Corps of Engineers Gyratory Compactor (left side of Figure 2.6). The author also notes that the in-place pavement density after 2615 coverages exceeded even 150-blow Marshall samples; however, the in-place density could be exceeded by 60 gyrations at either 200 or 300 psi. It was also noted the Marshall stabilities of samples produced with the Corps of Engineers Gyratory Compactor more closely approximated those of field samples (right side of Figure 2.6).

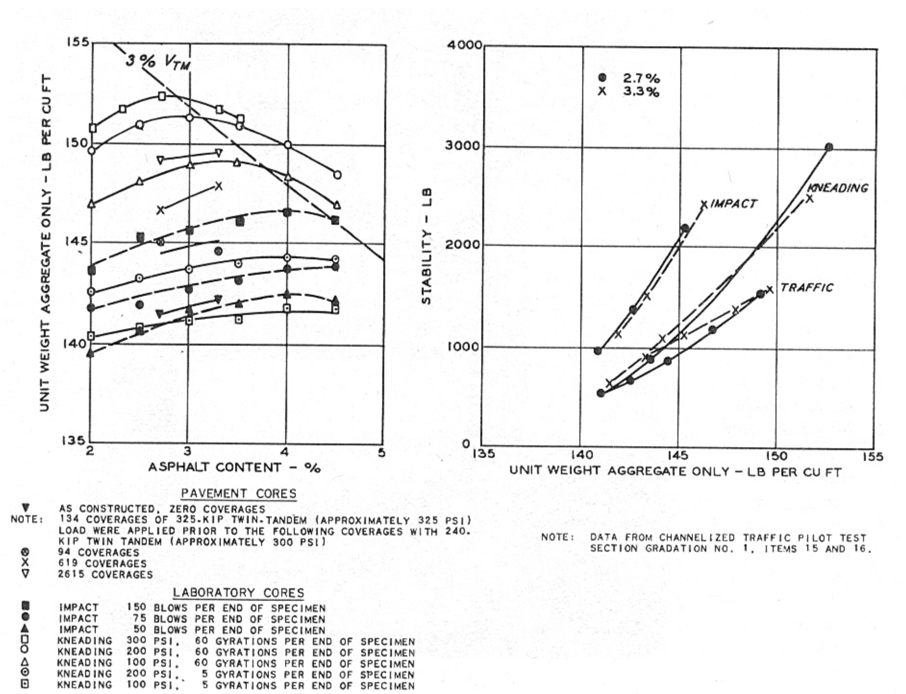

Figure 2.6. Comparison of Laboratory and Field Density and Stability Values (*25*).

The author goes on to outline a framework for selecting the optimum asphalt for HMA. A plot of aggregate density versus asphalt content can be used to determine the asphalt content at which the mix becomes plastic. As the compaction effort increases, the asphalt content at which the mix becomes plastic decreases. This is graphically illustrated in Figure 2.7. The ratio of the stress on the upper oil roller versus the vertical stress might be another indicator of mix stability.

In 1958, McRae and McDaniel (*26*), reported on additional advancements with the Corps of Engineers Gyratory Compactor. Rate of gyrations was studied and observed to have little effect on sample density. The machine was modified to record the gyratory motion of the sample during compaction. Initially, the angle of gyration would decrease from the level set prior to beginning the test; indicating densification of the mix. This densification would be a combination of that which occurs at the time of compaction and that which occurs under traffic. The pressure in the oil roller would increase during this phase. When a critical density was achieved, the specimen would become plastic and the angle of gyration would again increase and the oil-roller pressure would drop. It was believed that the number of gyrations before this occurred could be related to traffic. Recommendations were also developed to prepare samples with similar densities to samples compacted with the Marshall Hammer: 50-blows was approximately equivalent to samples compacted in the gyratory with a 100 psi vertical pressure and 1 degree initial angle compacted to 30 gyrations and 75-blows was approximately equivalent to samples compacted in the gyratory with a 200 psi vertical pressure and 1 degree initial angle compacted to 30 gyrations.

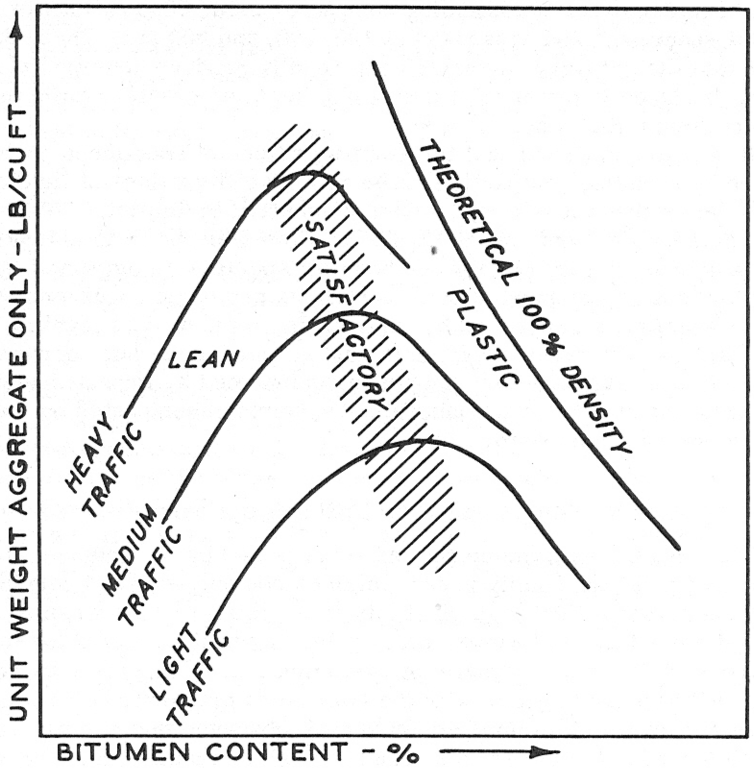

Figure 2.7. Aggregate Density as a Function of Asphalt Content and Compaction Level (*25*).

The Corps of Engineers Gyratory Compactor was later renamed the Corps of Engineers Gyratory Testing Machine (GTM) and adopted in 1974 as an ASTM D 3387, "Standard Test Method for Compaction and Shear Properties of Bituminous Mixtures by Means of the U. S. Corps of Engineers Gyratory Testing Machine (GTM) (*23*)". Additional research led to the development of an air roller to replace the oil roller which allowed for a "variable stress and variable shear strain testing capability" (*27*).

2.1.7 French Design Procedure

Bonnot (*28*) outlined the framework of the French mix design procedure for HMA. The French use their Gyratory Shear Compacting Press (PCG) to evaluate the workability of HMA. Similar to the Texas Gyratory Molding Machine and the GTM, the ends of the HMA sample are held parallel during compaction with the mold forming an oblique cylinder. One end of the sample is fixed and the other describes a cone as shown in Figure 2.8. The sample is compacted in a 160 mm diameter mold with a final sample height of approximately 150 mm. During compaction, a vertical compressive pressure of 0.6 MPa (87 psi) is applied to the sample and the angle of gyration is fixed at 1 degree from vertical. The sample height and the force required to maintain the 1 degree gyratory angle are recorded with each gyration. Assuming a fixed sample mass and mold diameter, the density of the sample can be estimated at each gyration. Samples are generally compacted to 200 gyrations at a rate of 6 gyrations per minute.

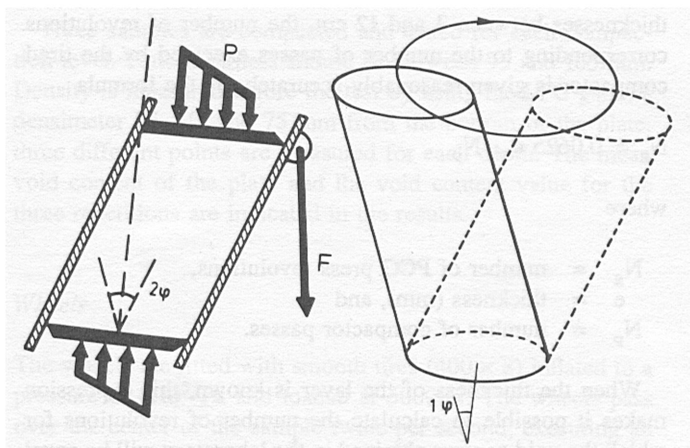

Figure 2.8. Compaction Principle of the PCG (*28*).

Correlations studies were conducted between the density obtained with the PCG and the in-place density achieved with a rubber tired roller at a given layer thickness. Equation 2 was developed for comparing the field compaction for lifts ranging in thickness from 3 to 12 cm to an equivalent number of gyrations in the PCG.

$$N_g = k \times e \times N_p \tag{2}$$

where,

N_g = number of PCG gyrations,

k = factor for compactor type; 0.0625 for rubber tired rollers and 0.25 for 10 ton vibratory rollers operating at 25 to 30 Hz,

e = layer thickness, (mm), and

N_p = number of rubber tired roller passes.

Using this equation, it is possible to estimate the obtainable in-place density using a given compaction effort. For instance, the achievable density of a 38 mm thick surface mix using 8 passes of a vibratory roller would be estimated at 76 gyrations of the PCG. The target in-place air voids (air voids = 100 – percent of theoretical maximum density) varies with climate, it is lower (3 to 4 percent air voids) for a cold mountainous region than it is for a hot region (6 to 7 percent air voids). If the air voids at the calculated number of gyrations is too high, the mix is unworkable and may be adjusted by:

- Increasing asphalt content,
- Increasing filler content,
- Substituting rounded fine aggregate, or
- Other gradation changes such as gap grading.

If the air voids are too low, the mix could be made stiffer by doing the opposite.

The PCG is used to develop the initial job mix formula. Additional performance testing is conducted depending on the application and may include: resistance to permanent deformation, predicted fatigue life, and resistance to moisture damage. Depending on the design conditions, these tests may be used to modify the design or simply verify minimum performance. Samples for performance testing are produced not with the PCG but with a compactor using a laboratory scale rubber tired roller. Samples may be sawed or cored from the resulting slab.

2.2 SUPERPAVE GYRATORY COMPACTOR

2.2.1 Selection of the SGC for the Superpave Mix Design System

One of the tasks faced by the SHRP researchers during the development of the Superpave Mix Design System was the selection of a laboratory compaction procedure. In the introduction to the selection process, Cominsky et al. (*29*) note, "compaction is considered the single most important factor affecting the performance of asphalt pavements. Hughes (*30*) stated, "It is important that the density of laboratory-compacted specimens approximate that obtained in the field in terms of (a) the structure of the mix and (b) the quantity, size, and distribution of the air voids."

Consuegra et al. (*31*) conducted a study on laboratory versus field compaction as part of the NCHRP project on the development of the Asphalt Aggregate Mixture Analysis System (AAMAS). Consuegra et al. (*31*) describe a major objective of their study to, "ensure that laboratory mixtures will be fabricated in a manner that adequately simulates field compaction and, consequently, will yield reliable engineering properties." Thus, two goals emerged, matching field air voids and matching the engineering properties of field compacted samples. [This author notes that the engineering properties of laboratory compacted samples are probably influenced by both aggregate orientation and the degree of aggregate degradation during compaction].

The research on the AAMAS system was completed in 1991, three years prior to the completion of the Superpave mix design system (*32*). The AAMAS research was linked to the SHRP research to develop the Superpave system. AAMAS included a study to select a laboratory compaction procedure by Consuegra et al. (*31*). Loose mix was sampled from five projects, one each in Colorado, Michigan, Texas, Virginia, and Wyoming and approximately 25 field cores were taken from each project immediately after construction. Five laboratory compaction devices were used in the study: mechanical Marshall Hammer, California Kneading Compactor, Arizona vibratory-kneading compactor, Texas Motorized Gyratory Shear Type Compactor and mobile steel wheel simulator. Three of these methods were discussed previously. The Arizona vibratory kneading compactor compacted samples with a rapid impact load (1,200 cycles per minute) and low contact pressure with the sample tilted at a slight angle (1 degree from vertical) to the applied load. The mobile steel wheel simulator used in this study was obtained from the Federal Highway Administration (FHWA). It consisted of curved foot that applied a static load to the sample. The curved foot consisted of a segment of a circle, simulating the action of a steel wheel static roller.

The laboratory compactive efforts with the five devices were varied to achieve the average in-place density determined for each of the field projects. The required compactive effort for the Marshall Hammer varied from 20 to 47 blows per face to match the in-place air voids. Initially, the researchers planned to reduce the number of gyrations with the Texas Gyratory shear Compactor; however three gyrations, the minimum that can be used with the Texas Gyratory, resulted in lower air void contents than the field cores. Therefore, the gyration pressure and end pressure were varied to match the field air voids. The gyration pressure was varied from 25 to 100 psi; 50 psi is the Texas standard. The end pressure was varied from 0 to 2500 psi; 2500 psi is the Texas standard. The Texas project required the least and the Virginia project the most compaction effort to match the field in-place air voids at the time of construction.

The engineering properties of the pavement cores and laboratory samples were evaluated by means of indirect tensile strength at 41, 77, and 104 °F, repeated load indirect resilient modulus, and indirect tensile creep. The average differences and mean square error (MSE) between the test results on field cores and laboratory compacted samples were used to assess the best compaction method. MSE equally weights the variance of the test results and the square of the bias of the test results between the field and lab compacted samples. Based on these analyses, no single compaction method always provided the best match with the test results for the field cores; however, the Texas Gyratory Shear Compactor was consistently better. The following lists the ranking of the compaction devices (*31*):

1. Texas Gyratory Shear Compactor,
2. California Kneading Compactor,
3. Mobile steel wheel simulator,
4. Arizona vibratory-kneading compactor,
5. Marshall Mechanical Hammer.

In addition to the evaluation of the engineering properties of samples produced using various compaction methods as compared to field cores, Von Quintus et al. (1991) present comparisons on compactability, laboratory and field air voids after two years of traffic, and aggregate orientation. Both the Marshall Hammer and the Texas Gyratory produced the same compactability rankings as observed in the field. Based on MSE, the California Kneading Compactor best matched the field air voids after two years followed by the Marshall Hammer, Texas Gyratory Shear Compactor, Arizona vibratory-kneading compactor and mobile steel wheel simulator. The mobile steel wheel simulator and Texas Gyratory Shear Compactor best simulated aggregate orientation as compared to the field cores. Based on these results and limited testing with the GTM, the AAMAS researchers (*32*) recommended either the Texas Gyratory Shear Compactor or the GTM for producing laboratory compacted samples for design and performance testing.

The SHRP A-003A Contractor, the University of California at Berkley (*33*), conducted a study of the effects of laboratory compaction procedure on the rutting and fatigue properties of HMA. Three compactors were evaluated in the study: the Texas Gyratory Compactor, California Kneading Compactor and rolling wheel compactor. In addition, limited testing was conducted with the Corps of Engineers GTM and the Exxon Rolling-Wheel Compactor. Sixteen HMA combinations were evaluated in the study: two asphalt sources (same grade), two aggregate types (granite and chert), two asphalt contents (optimum based on California Kneading Compactor and optimum plus either 0.5 percent [granite] or 0.7 percent [chert]), and two target air void contents (4 and 11.5 percent). The optimum plus asphalt contents approximate that obtained from a 75-blow Marshall design. Two primary tests were performed to evaluate the effect on rutting: static creep and shear creep; both tests were performed at two temperatures (40 and 60 °C) and two stress levels (varied). Beam fatigue tests were performed on samples prepared using the California Kneading Compactor and the rolling wheel compactor. Since beam samples cannot be prepared with the Texas Gyratory Compactor, diametral fatigue tests were also performed using samples compacted with all three compaction methods. Fatigue tests were conducted in constant stress mode at two stress levels and two temperatures (0 or 4 °C and 20 °C).

The California Kneading Compactor consistently produced the most rut-resistant samples and the Texas Gyratory the least rut-resistant samples. Dynamic modulus testing indicated that samples compacted with the California Kneading Compactor were in fact stiffer than samples compacted with the Texas Gyratory Compactor. This agreed with the findings from the AAMAS study (*29*). All three devices ranked all of the experimental variables in the same order, e.g., the granite aggregate was more rut resistant than the chert aggregate was. The California Kneading Compactor was more sensitive to aggregate type (angularity), than the Texas Gyratory Compactor was. The greater rut resistance of samples compacted with the California Kneading Compactor was believed to be related to the development of greater aggregate inter-particle contact.

The Texas Gyratory Compactor consistently produced samples which had longer fatigue lives than those samples compacted in the California Kneading Compactor; the rolling wheel compactor samples produced an intermediate ranking between the two. The ranking of the experimental variables were different for samples compacted with the three different compactors. The Texas Gyratory Compactor was believed to be more sensitive to asphalt type than the California Kneading Compactor, but only slightly more sensitive than the rolling wheel compactor.

Limited comparisons were performed with field cores from two projects in California. Testing with the Corps of Engineers GTM indicated that two different types of gyratory compactors could produce samples with very different engineering properties.

Samples produced with the two different rolling wheel compactors were similar. SHRP A-003A researchers (*33*) recommended the rolling wheel compactor. The researchers emphasized the importance of having a single compaction procedure. This author believes that their decision was partially based on their desire to have a compaction procedure which could produce flexural beam fatigue samples. This study was later criticized for not having been correlated to field performance (*29, 34*).

Based on the results from the AAMAS and SHRP A-003A studies, SHRP commissioned a third study which was conducted by Texas A&M University, the SHRP A-001 contractor (*29*). Five pavement sites were selected from the SHRP Special Pavement Studies (SPS)-5 and SPS-6 field tests. Approximately 30, 4-inch diameter cores were taken from each section. The average in-place air voids at the five sites varied from 3 to 8 percent, with a variation at each site of 2 to 5 percent. Four laboratory compaction devices were chosen for evaluation: the Texas Gyratory Compactor, Exxon Rolling Wheel Compactor, mechanical Marshall Hammer, and Elf Linear Kneading Compactor. The complete matrix of tests for all sites were only performed with samples compacted using the Texas Gyratory Compactor and the Exxon Rolling Wheel Compactor. The laboratory compacted samples were produced with laboratory prepared HMA. Laboratory compaction effort was varied to produce a range of air voids. This was somewhat difficult with the Exxon Rolling Wheel Compactor, which produced lower than expected sample air voids. Six tests were used to evaluate the engineering properties of the HMA: indirect tensile strength at 25 °C, resilient modulus at 0 and 25 °C, Marshall Stability, Hveem Stability, repeated load cyclic creep at 40 °C and compressive strength at 40 °C. Only the indirect tensile strength, resilient modulus, and

Marshall Stability tests were conducted on samples compacted with the Marshall Hammer; HMA from only two sites were compacted and tested with the Elf Linear Kneading Compactor (*34*).

Linear regressions were used to determine slope and offset values between air voids (x variable) and the test result (y variable) for the field cores and samples compacted with the various compactors for each site. Statistical analyses were performed to compare the slope and intercepts for a given test between the field cores and samples compacted with each of the laboratory compactors used. The Texas Gyratory Compactor produced samples equivalent to field cores in 24 of 33 cases (73 percent). The Exxon Rolling Wheel compactor and the Elf Linear Kneading Compactor produced samples with equivalent properties to field cores in 18 of 28 and 9 of 14 cases, respectively (both 64 percent). The Marshall Hammer produced samples with equivalent properties to field cores in only 10 of 20 cases (50 percent). The numbers of differences between the different compactors were not statistically different at the 5 percent significance level. The authors note that the differences between the field cores and laboratory compacted samples were relatively small. They also note that the Texas Gyratory Compactor is more convenient, faster and cheaper for producing samples at a given air void level than the rolling wheel compactors were. Based on this study, the Texas Gyratory Compactor was recommended for the production of laboratory specimens (*34*).

Based on the AAMAS study, the research conducted by Button et al. (*34*) and the work completed by the French with the PCG, the SHRP researchers elected to use a gyratory compactor for the production of routine testing samples (*29*). Further, the SHRP researchers selected a protocol similar to the French PCG. As noted previously, the PCG compacts samples at six gyrations per minute. The SHRP researchers desired to compact samples as fast as possible to decrease testing time (4 samples compacted to 200 gyrations takes approximately one half day at 6 gyrations per minute). As noted previously, McRae and McDaniel (*26*) found the effect of gyration rate to be insignificant up to 10 gyrations per minute. Therefore, the SHRP researchers designed an experiment to assess the effect of gyration rate on the resulting volumetric properties of the compacted sample.

A single aggregate source and a single asphalt source were used in the experiment. Samples were compacted at optimum and optimum ± 1.0 percent asphalt content. Samples were compacted at 6, 15 and 30 gyrations per minute. Volumetric properties evaluated included optimum asphalt content, air voids, VMA and VFA. Air void contents of 4.4, 4.5 and 4.0 percent were reported, respectively, for 6, 15 and 30 gyrations per minute. Statistically, these values were not different. Therefore, the SHRP researcher selected a gyration rate of 30 gyrations per minute to minimize testing time (*29*). The initial characteristics of the SHRP Gyratory Compactor were selected as follows:

1. Angle of gyration = 1 degree,
2. Vertical pressure = 600 kPa (87 psi),
3. Speed of gyration = 30 rpm.

The development of the design compaction level, Ndesign will be discussed later in the report.

2.2.2 Studies to Evaluate Factors Affecting Gyratory Compaction

Prior to the conclusion of the SHRP research program, initial studies were conducted to compare specifications for gyratory compactors and their effect on the resulting sample properties. A study was conducted to compare a SHRP Gyratory compactor, built by the Rainhart Company, a modified Texas Gyratory Compactor and a Corps of Engineers GTM (*29*). The SHRP Gyratory Compactor could be used to compact both 4-inch and 6-inch diameter samples. The angle on the Texas Gyratory Compactor was adjusted to 1 degree, and a frequency controller was added to allow the compaction speed to be set to 30 rpm. A single aggregate source, binder source, and gradation (19.0 mm NMAS) were used for the study. Samples were compacted at optimum asphalt content and optimum ± 1.0 percent. Two replicates were compacted in the SHRP and Texas Gyratory compactors and three replicates were compacted in the Corps of Engineers GTM. A larger study is described to compare the SHRP Gyratory and modified Texas Gyratory, but the results are not presented.

Based on the French concept of reporting the log of gyrations (x-axis) versus sample density (y-axis) reported by Moultier (*35*) in reference (*29*), three parameter were identified to compare the compactors: C_{10}, C_{230} and K, where, C_{10} is the sample density at 10 gyrations, C_{230} is the sample density at 230 gyrations, and K is the slope of the densification line. The parameters are illustrated in Figure 2.9. Changes in sample asphalt content are expected to affect the compaction curve as illustrated in Figure 2.10.

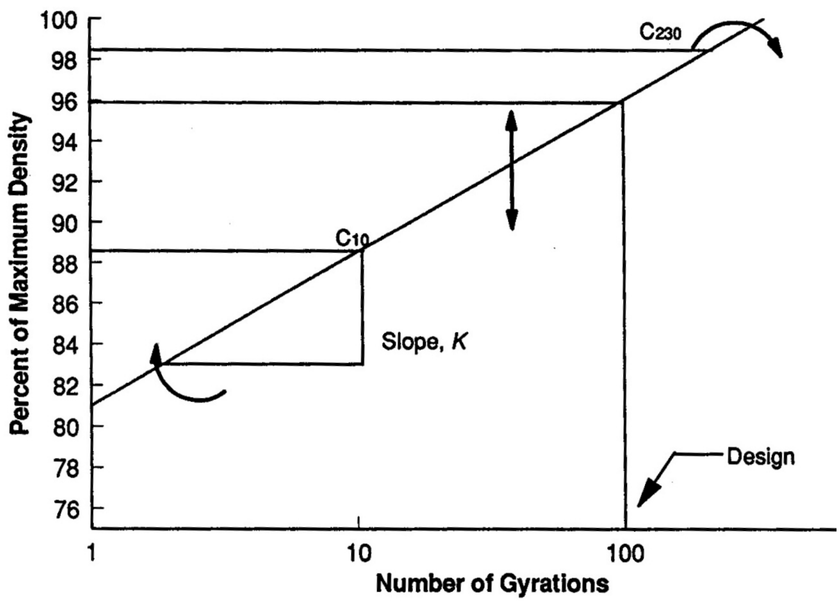

Figure 2.9. Typical Gyratory Compaction Curve (*29*).

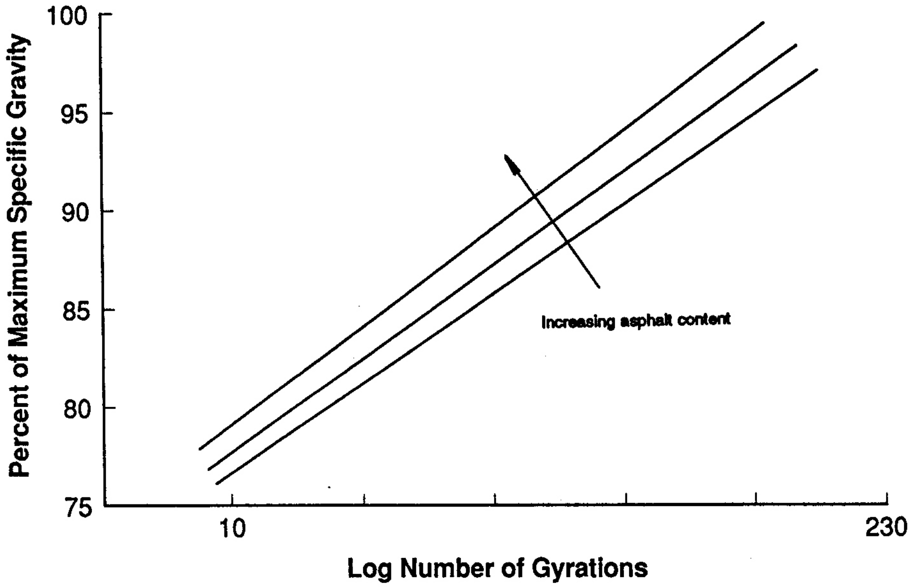

Figure 2.10. Effect of Asphalt Content on Compaction (*29*).

The results of the experiment to compare the three gyratory compactors are shown in Table 2.1. For the optimum minus samples, the corps of Engineers GTM produced significantly higher sample densities than the SHRP Gyratory at C_{10} and all other samples at C_{230}. At optimum plus, the compacted sample densities were significantly different at C_{10} for all three compactors; at C_{230} the Corps of Engineers GTM results and 6-inch diameter SHRP Gyratory results were significantly different from each other and significantly different from the other samples. Thus, it was concluded that the different gyratory compactors did not compact the same.

TABLE 2.1 Comparison of Densification Parameters from Gyratory Compactors (*29*)

AC%	Parameter	Gyratory Compactor			
		SHRP		Modified Texas	Corps GTM
		4-inch	6-inch		
Optimum Minus	C_{10}	83.4	84.4	85.4	86.8
	C_{230}	92.0	91.3	92.4	93.7
	K	6.281	5.039	5.100	5.059
Optimum	C_{10}	85.6	86.4	87.1	89.0
	C_{230}	95.2	94.4	95.0	96.5
	K	7.100	5.958	5.858	5.531
Optimum Plus	C_{10}	88.5	88.8	90.0	91.6
	C_{230}	99.0	98.0	99.0	99.4
	K	7.732	6.772	6.598	5.724

It was observed that the modified Texas Gyratory Compactor had an angle of gyration of 0.97 degrees (external) while the SHRP Gyratory Compactor had angles of 1.14 and 1.30 degrees, respectively, when compacting the 6-inch and 4-inch diameter samples. Cominski et al. (*29*) concluded, "A variation in the angle of compaction of ± 0.02 degrees resulted in an air voids variation of ± 0.22 percnt at 100 gyrations." This difference resulted in a change in optimum asphalt content of ± 0.15 percent. Based on this research, the specification for angle of gyration was changed to 1.0 ± 0.02 degress.

The differences in compaction with the Corps of Engineers GTM were attributed to the manner in which the angle is induced. The angle of gyration for the Corps of Engineers GTM is fixed at only two points, one of which (the oil roller) allows the angle to vary if the pressure in the roller is exceeded, while the SHRP and modified Texas Gyratory Compactors fix the angle at three points.

In 1994, two models of SGC's were initially approved as meeting the specifications for the SHRP (now called Superpave) Gyratory Compactor or SGC by the FHWA in a pooled fund purchase for state departments of transportation: the Pine Instruments Company (Pine) model number AFGC125X and the Troxler Electronic Laboratories, Inc. (Troxler) model number 4140 (*36, 37*). A study conducted by the Asphalt Institute (*38*) compared these two compactors with the modified Texas gyratory compactor used to develop the Superpave criteria during the Strategic Highway Research Program and a prototype Rainhart (SHRP) Compactor. Three samples of each of six blends were compacted in each compactor at optimum asphalt content. At Ndesign, the Pine compactor produced similar results to the Modified Texas compactor and the Troxler compactor produced results similar to the Rainhart Compactor. The Pine Model AFGC125X produced significantly higher densities than the Troxler Model 4140 did in five of six comparisons. After the completion of this study, modifications were made to both the Pine and Troxler SGCs.

Subsequently, both the Pine Model AFGC125X and Troxler 4140 SGCs were included in a ruggedness study to evaluate AASHTO TP4 (*39*). The ruggedness study was conducted according to ASTM C1067. As specified, seven factors were evaluated as part of the ruggedness study: angle of gyration, mold loading procedure, compaction pressure, precompaction, compaction temperature, specimen height, and aging period. A high and low level was selected for each of these factors. Due to the difficulty in obtaining exact external angles of gyration and exact specimen heights, some tolerance was allowed for both of these parameters. The low range for external angle of gyration varied from 1.22 to 1.24 degrees and the high angle varied from 1.26 to 1.28 degrees. The specification for the angle of gyration had been changed to 1.25 ± 0.02 degrees in 1994 during the original Ndesign experiment (*29*). This will be discussed later in the document. Fixed batch masses of 4500 and 5000 g were used to produce sample heights of approximately 110 and 120 mm. Four 19.0 mm NMAS mixes representing two aggregate types (crushed limestone and crushed river gravel) and two gradations (coarse and fine) were used in the experiment.

The range for compaction pressure, then specified as ± 3 percent or ± 18.0 kPa, caused significant differences in three of five laboratories for one or more mixes (4 cases, total). Marginally significant differences were found in seven of twenty cases for the height extremes. Additional analysis of the data indicated that the actual differences (approximately 12 mm) exceeded the 10 mm target difference. The 12 mm difference caused marginally significant differences for the fine graded mixes. Therefore, it was recommended that the existing tolerance on sample height in AASHTO TP4 be relaxed from ± 1 mm to ± 5 mm (*39*).

The two ranges for external angle of gyration only resulted in a significant difference in one in twenty cases. As anticipated, higher angles did produce denser specimens, but regression analysis indicated that only one percent of the difference in sample density was explained by the change in angle and the relationship was not significant (*39*). Both compactor types responded similarly to all seven of the main effects. However, additional analyses indicated differences in sample density between the laboratories that used the Pine AFGC125X compactor and the laboratories that used the Troxler 4140 compactor. Paired comparisons using a t-distribution grouped the three labs using the Pine compactor together and the two labs using the Troxler compactors together for three of the four mixes with the Pine compactors producing higher sample densities. There were three groupings for the fourth mix, but once again the Troxler compactors grouped together (*39*).

As the use of the SGC became widespread across the United States, several additional manufacturers have developed SGC's. In addition, both Pine and Troxler have developed new models of SGC's. This led to the need to develop a means of evaluating the new SGC's to ensure that they would produce results similar to the Pine AFGC125X and Troxler 4140. AASHTO TP4 did not contain a precision statement (*36*). Therefore, it was not clear what the acceptable difference between various SGCs should be.

To address potential differences between compactors, FHWA developed a standard protocol to compare compactors, which was approved by the FHWA Superpave Mixtures Expert Task Group, and is designated AASHTO PP35, "Standard Practice for Evaluation of Superpave Gyratory Compactors (SGCs)" (*36, 37, 40*). AASHTO PP35 consists of a comparison between a single unit of the new compactor versus one of the two original pooled fund compactors (Pine AFGC125X or Troxler 4140). The comparison consists of compacting six replicate samples for each of four mixes in both compactors. The mixes specified include: a 12.5 mm nominal maximum aggregate size (NMAS) mix, two 19.0 mm NMAS mixes (one coarse and one fine graded) and a 25.0 mm NMAS mix. The comparison is to be performed at one of the five Superpave Regional Centers (*36*). When evaluating new models, both Pine and Troxler performed the AASHTO PP35 comparisons against their respective original compactor (*37, 41*).

Many agencies, throughout the country, have reported significant differences in the bulk specific gravity of compacted samples from different SGCs, which have been properly calibrated. Iowa Department of Transportation (*42*) completed a study to address this very concern. They evaluated four brands of SGCs: Pine AFGC125X, Troxler 4140, Test Quip Brovold and Interlaken Model 1. Four 19.0 mm nominal maximum aggregate size mixes, three coarse-graded mixes and one fine-graded mix were used in the study. All of the compactors were calibrated according to the manufacturer's recommendations prior to testing. The Troxler compactor was found to produce consistently higher densities at Ninitial. This was believed to be related to the manner in which the angle is induced. The Pine SGC consistently produced the highest density and the Interlaken SGC produced the lowest density at Ndesign. The Interlaken SGC produced the largest differences from the average density of all of the compactors.

2.2.3 Internal Angle of Gyration

The sensitivity of the density of SGC compacted samples to the angle of gyration was identified during the SHRP (*29*). The internal angle of gyration is defined as the angle of the interior of the mold wall relative to the top and bottom plates or platens. The platens are assumed to be parallel to one another. The gyration angle (internal and external) changes (generally decreases) with all types of compactors during compaction, primarily due to flexing of the SGC frame, but can be significant with some compactors. One source of compliance is believed to be the ram used to apply vertical pressure on the samples. One of the platens is generally attached to the ram. When the ram flexes during compaction, the platen supported by the ram may not remain parallel to the opposite platen. For these reasons, the gyration angle must be determined during compaction, preferably with a full-height HMA sample, not in the un-loaded (mold empty) condition.

The external angle of gyration is measured differently for each brand and many models (within a brand) of gyratory compactors. The Pine Model AFGC125X uses dial gauges and can measure the static (not gyrating) angle in both the loaded (with a full-height HMA sample) and unloaded condition. The Troxler 4140 uses a digital gauge to dynamically (while the compactor is gyrating) measure the offset of the turntable used to apply the angle in the loaded condition. No means for measuring the angle of gyration was supplied for the Rainhart compactors. All of the other compactors, Test Quip (Gilson or Pine AFGB1A), Interlaken, Pine Model AFG1A and Troxler 4141, use internal linear voltage displacement transducers (LVDT) to measure and display the external angle of gyration during compaction based on one to three points.   The numerous methods of measuring the external angle of gyration result in a lack of uniformity from one SGC to another.

The FHWA, in cooperation with Test Quip Inc., developed an independent device to measure the internal angle of gyration. The device is referred to as the Dynamic Angle Validation Kit (DAVK). The DAVK is placed inside the SGC mold with hot mix asphalt sample. A data acquisition system within the DAVK dynamically records the internal angle of gyration during compaction (*43*). A draft procedure (*40*) for evaluating the dynamic internal angle of gyration "Evaluation of the Superpave Gyratory Compactor's (SGCs) Angle of Gyration Using the FHWA SGC Angle Validation Kit" was developed by FHWA.

The DAVK unit is shown in Figure 2.11 with its accompanying NIST traceable calibration standard. The DAVK consists of a machined body designed to fit inside a SGC mold. Two probes connected to a single LVDT protrude through the body and rest against the mold wall. The base of the unit rests against the top or bottom mold plate. During compaction, the base of the DAVK is held tightly against the top or bottom mold plate and acts as a reference plane from which the internal angle of gyration is measured using the LVDTs. The DAVK body contains a data acquisition system and power source. The data acquisition system is programmed and the data downloaded to a notebook computer using software provided by the manufacturer.

The DAVK is designed to measure the internal angle of gyration along with a full height (115 mm tall) hot mix asphalt (HMA) sample (*43*). Figure 2.12 illustrates the possible measurements of angle of gyration. The external angle of gyration is defined as α. The internal angles of gyration are defined as δ_T (top) and δ_B (bottom) for the angle measured when the DAVK is placed above the HMA samples or below the HMA sample, respectively. The measured internal angle of gyration is different when the DAVK is placed at the top or bottom of the mold (*43, 44*). Therefore, δ_T and δ_B, as measured by the DAVK, should be averaged to determine an effective internal angle of gyration (δ_{AVG}) (*43 - 45*). The DAVK unit is approximately 77 mm tall. Certain SGC molds cannot accommodate the DAVK and a 115 mm tall (final height) HMA sample. This can be solved by extrapolation (*43*).

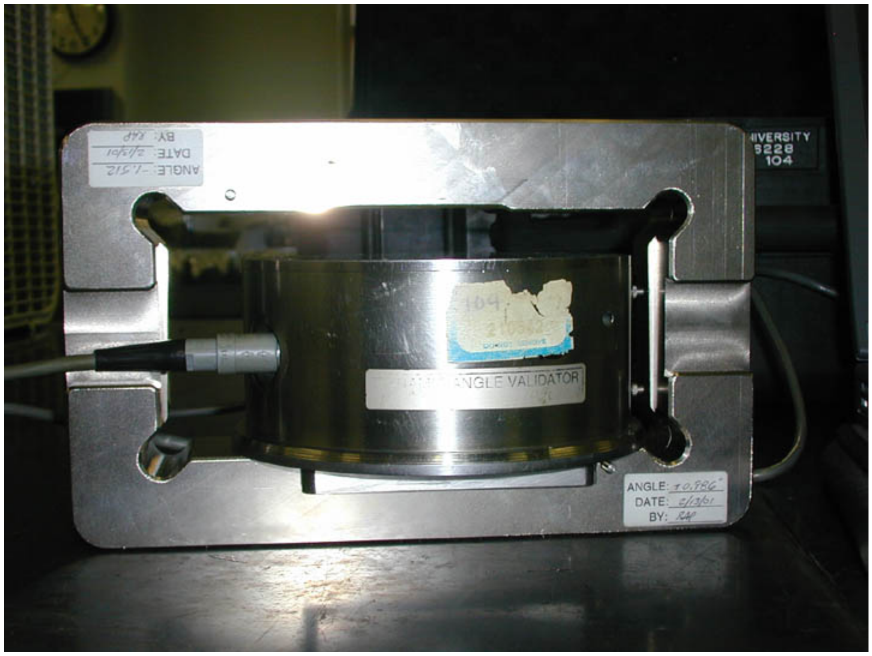

Figure 2.11. DAVK and Calibration Block.

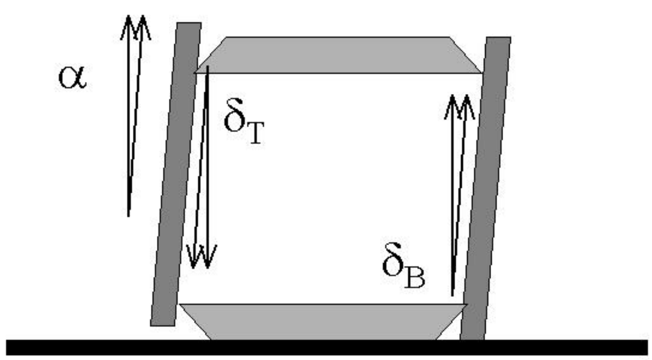

Figure 2.12. Definition of Internal and External Angle of Gyration.

To determine the internal angle of gyration by extrapolation, a series of HMA masses necessary to produce varying height samples are utilized. Typically, three sample masses are used (to produce three different height samples) for the extrapolation for which two replicates of each sample mass are compacted with the DAVK against the upper platen and two replicates with the DAVK against the lower platens. Research (*44, 46*) indicates an excellent linear relationship between sample height and internal angle of gyration with the DAVK at both the top and the bottom of the mold. Extrapolations to 115 mm are performed separately to determine δ_T and δ_B. δ_T and δ_B are then averaged to produce δ_{AVG}.

Studies have been conducted to relate the dynamic internal angle of gyration (DIA) to sample density. Dalton (*44*) conducted a study to evaluate the effect of DIA on compacted sample using two compactors, the Pine AFGC125X and the Pine AFG1A. Testing indicated that a change in internal angle of 0.1 degrees resulted in a change of 0.014 Gmb units or approximately 0.6 percent air voids for the Pine AFGC125X and a change in internal angle of 0.1 degrees resulted in a change of 0.017 Gmb units or approximately 0.7 percent air voids for the Pine AFG1A. The varying internal angles were artificially produced by inducing end plate deflections with machined tapers in the Pine AFG1A.

Dalton (*47*) reported on a second study where four compactors, adjusted to the same internal angle of gyration, compared favorably for nine of ten mixes representing a wide range of NMAS according to the criteria established for AASHTO PP35. Two of the four compactors allowed full height HMA samples to be compacted with the DAVK; one used precompaction and one used extrapolation. The results of this experiment indicated that the measured internal angle of gyration was independent of mix type.

FHWA conducted a study to determine the target and tolerance for the DIA. Al-Khateeb et al. (*48*) determined a target DIA of 1.16 degrees. The target was based on setting single articles of the original pooled-fund purchase SGCs, the Pine AFGC125X and Troxler 4140, to an external angle of gyration (using the manufacturer's calibration equipment) of 1.25 degrees as specified in AASHTO T312, and measuring the DIA using the AVK. Using a 12.5 mm NMAS Superpave mix, the average DIA was determined to be 1.176 and 1.140 degrees, respectively for the Pine AFGC125X and Troxler 4140 SGCs. Thus, set at an external angle of 1.25 degrees, the original pooled fund SGCs produced an average DIA of 1.16 degrees. The tolerance was determined to allow a maximum variability of approximately 0.10 percent design asphalt content or 0.25 percent air voids. Using the relationship developed between DIA and Gmb and a target change in air voids of 0.25 percent, the tolerance for DIA was determined to be ±0.03 degrees.

Prowell et al. (*49*) measured the DIA on 112 different SGCs in Alabama (seven different models). Three samples of a 19.0 mm NMAS mix were then compacted to 100 gyrations on each compactor for density determination. Regression analysis using all the data indicated an $R^2 = 0.37$. This indicates that although DIA explains part of the variability, other factors affect compacted sample density from one laboratory to another. Figure 2.13 shows the average internal angle of gyration versus the average Gmb values by compactor type for the 19.0 mm NMAS mix at 4.4 percent AC. A simple linear regression was performed with internal angle of gyration as a predictor for Gmb excluding

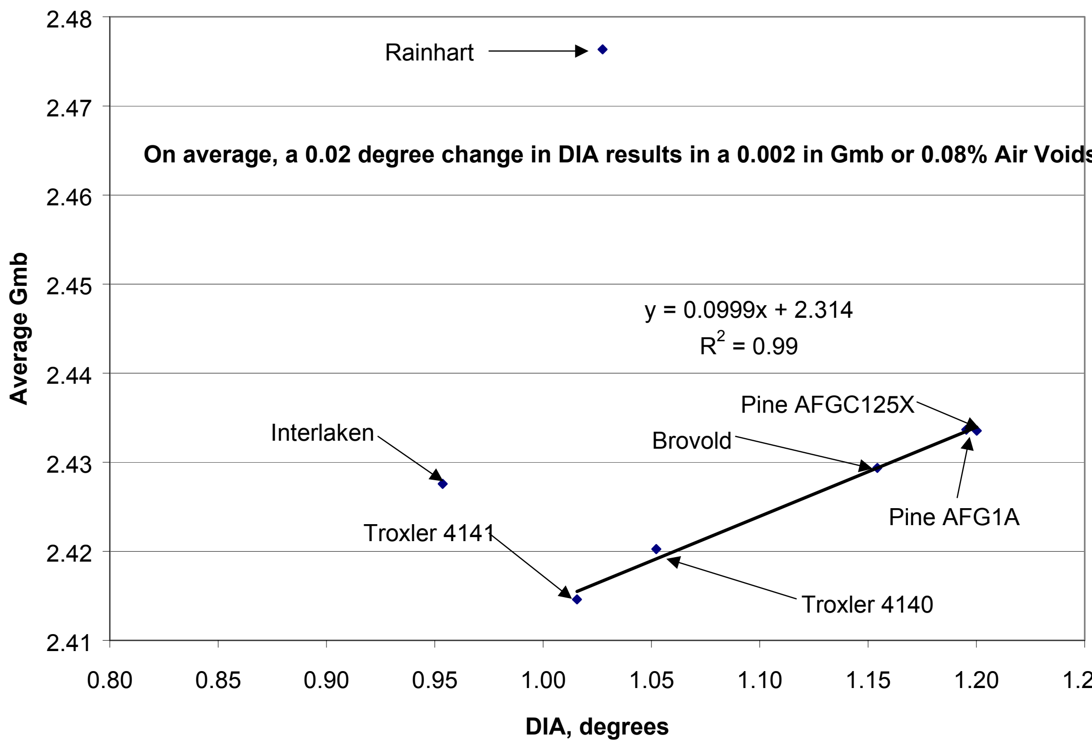

Figure 2.13. G_{mb} versus Average Internal Angle of Gyration (*49*).

the Interlaken and Rainhart data. The R^2 = 0.99 indicates on average an excellent relationship between average internal angle of gyration and average sample bulk density. The relationship shown in Figure 2.13 indicates that on average a change in 0.1 degrees of internal angle will result in a change of 0.010 G_{mb} units or a difference in air voids of approximately 0.4 percent. Therefore, a change of ±0.02 degrees as allowed by AASHTO T312 could produce a difference in air voids of approximately 0.08 percent or based on Superpave's rule of thumb (all things being equal, a 0.4% change in AC% results in a 1.0% change in air voids) approximately a 0.03 percent difference in design asphalt content.

2.3 DENSIFICATION OF PAVEMENTS UNDER TRAFFIC

A number of studies have been conducted to evaluate pavement densification under traffic. Though the general consensus is that pavements reach their ultimate density after the second or third summer, the results in research studies have varied. Additionally, some of these studies have tried to relate in-place density to laboratory compaction.

The first study to relate laboratory compaction to densification under traffic was the Corps of Engineers Study to develop the Marshall Method (*14*). As noted previously, accelerated loading was used to apply 3,500 passes of a 15,000 lb wheel load; 1,500 passes of a 37,000 lb wheel load; or 1,500 passes of a dual wheel configuration loaded to 60,000 lbs to test sections produced at various asphalt contents. It was noted that as-constructed density was approximated by 98 percent of the density of 50-blow Marshall samples. The 50-blow compaction effort appears to have been selected not on the basis of air voids after traffic, but by comparing the optimum asphalt content obtained with the various compaction efforts to visual assessments of the field performance of the various sections at different asphalt contents (*50*).

Dillard (*51*) tracked six Virginia sand asphalt pavements over a 100-week (2-year) period starting in 1952. Coring was conducted 5 times after construction on each of the 6 projects. The densification of 4 of the 6 projects, all sand asphalts, appears to have stabilized after one year, while the coarser mixes continued to densify in the second year. In 4 of 6 cases, 50-blow Marshall samples had a higher density than the pavement did after 2-years of traffic.

Twenty additional pavements, 13 HMA and 7 sand asphalt, were sampled the following year. Lift thicknesses ranged from ¾ to 1 ½ inches. HMA was sampled out of haul trucks at the HMA plant and compacted using 30, 50, and 75 blows; sand asphalt samples were compacted with 20, 35, and 50 blows. Cores were taken from each section between 1 to 4 months and between 13 to 16 months after construction. Comparisons were made between the core densities after 13 to 16 months and the Marshall sample densities compacted with the aforementioned blow counts. For the sand asphalt mixes, 30-blows appeared to provide the best correlation with in-place density; for 7 out of 13 sand mixes the mean 30-blow Marshall densities and in-place densities after 13 to 16 months of traffic were not significantly different. Figure 2.14 shows the data for the HMA mixes. The authors estimated that between 15 and 20 blows would best match the in-place density of the HMA. The authors noted the relative unimportance of traffic in the correlation between number of Marshall blows and in-place pavement density (Figure 2.14).

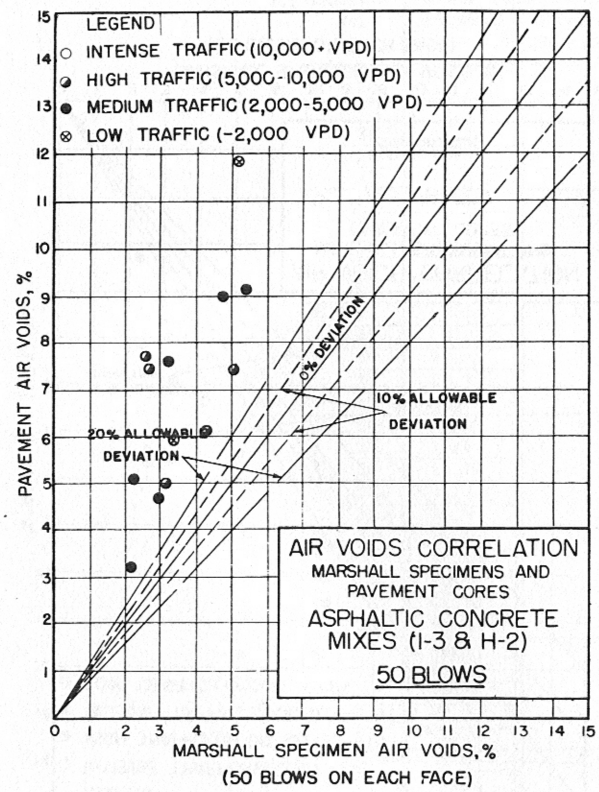

Figure 2.14. 50-Blow Marshall versus In-Place Densities (*51*)

Campen et al. (*52*) evaluated the densification of pavements placed in Omaha, NE between 1955 and 1959. The pavements were designed with a 50-blow Marshall compaction effort, with maximum aggregate sizes of 1/2, 5/8, and 3/4 inch. Primarily one mix design was used in each year; however, in 1957 the mix was altered from a 5/8 inch to a 3/4 inch maximum size. Laboratory samples were compacted and samples were sawed from the pavement immediately after construction. In 1960, samples were sawed from the pavements at the rate of 4 to 10 per mile. By 1960, 13 of 18 pavements had densified to ± 1.0 percent of the laboratory density, with 3 of those 13 pavements slightly exceeding the laboratory compacted sample density. The authors concluded the following:

1. Ultimate density is achieved in a few months in hot weather,
2. Initial density does not control ultimate density [this author noted a slight trend, $R^2 = 0.25$, when plotting the data],
3. The compacted density obtained from a 50-blow Marshall was not exceeded by heavy traffic,
4. Initial density affects the wear [raveling] of the pavement.

The authors note that rut resistance seemed to have been achieved at the expense of durability. The pavements placed in 1955 exhibited slight rutting and shoving at critical locations. Pavements placed after 1955 exhibited raveling, at times extreme raveling. The authors conclude (*52*), "In spite of all the scientific advancement the design of bituminous paving mixtures is still as much of an art as it is a science." This author believes that statement is still true to some extent today!

Graham et al. (*53*) tracked the densification of 47 test sections on 12 projects throughout New York over a two-year period including approximately 700 cores and 200 Marshall samples. Due to a lack of adequate traffic data, the authors did not attempt to relate traffic to pavement densification. Instead they presented the average densification of all of the sections with time. They concluded that the pavements densified significantly over the first year, but to a lesser degree over the second year (2.0% average increase in density first year versus 0.6% average increase in the second). Immediately after construction, 29 percent of pavements were less than 95 percent of Marshall density; after one year this was reduced to 8 percent and after two years it was reduced to 4 percent. [This author notes that 95 percent of Marshall density would be approximately 91 percent of theoretical maximum density.] An equation was developed to predict in-place air voids. The three most significant terms were volume of asphalt binder, deflection of the underlying pavement, and deviation of the aggregate gradation from the maximum density line.

Woodward and Vicelja (*54*) monitored the construction of Aviation Boulevard in Los Angeles, CA. Two lifts were placed, 3 inches (uncompacted) of 1 ½ inch maximum aggregate size base mix and 2 inches (uncompacted) of a ½ inch maximum aggregate size surface mix for a compacted thickness of 4 inches. The pavement was cored at the time of construction and 30, 60, and between 90 and 180 days after construction for a total of 169 cores. The average as-constructed density was 133 to 135 lbs/ft^3. Density increased approximate 3 lbs/ft^3 in the first 30 days; 1 to 1 ½ lbs/ft^3 in the next 30 days; and 1 to 1 ½ lbs/ft^3 in the final increment. Permeability tests and a large quantity of other data were collected but not reported.

Bright et al. (*55*) constructed 24 test sections on U. S. Route 64 west of Raleigh, NC. Two coarse aggregates, granite and gravel, were used to produce a ½ inch maximum size mix with an 85/100 pen binder. The lift thickness was 1 inch. The mixing temperatures in the test sections were altered (225, 250, 287, and 345 °F) to produce a range of mix viscosities from approximately 40 to 900 Saybolt Furol Seconds. The sections were cored at the time of construction and 4, 9, and 21 months after construction. Though the as-constructed densities varied, the in-place densities converged under traffic, except for the granite mix placed at 225 °F and the gravel mix placed at 250 °F. Binder was recovered from the cores for testing. Initially, the mix placed at lower temperatures exhibited less binder aging. However, the authors note that by 21 months less binder aging was noted in the sections with higher initial density.

Serafin et al. (*57*) tracked the pavement densification of 6 test sections representing 6 different binder sources (one grade) each subdivided into 5 sub-sections with varying binder content and compaction temperatures on one project in Michigan for 12 years. The pavement was subjected to approximately 8 million tractor-trailer passes during this period. An examination of the reported data indicates the pavement densification leveled off after 4 years of traffic.

Palmer et al. (*58*) reported on a continuation of the study conducted by Graham et al. (*53*) in New York. The pavement densities were tracked for a period of 5 years. The authors conclude, "If such a thing exists as "ultimate field density" of an asphalt concrete mixture, service time to attain this equilibrium may exceed 5 yr. [year] for New York State conditions, whereas studies elsewhere indicate leveling off of density after 1 to 4 yr. [years] of service (ultimate density being defined as that not exceeded with passage of further traffic and/or time)."

Epps et al. (*58*) conducted a study to try and determine the factors which affect the ultimate density of pavements with relation to the laboratory density determined with the Texas Gyratory Compactor. The study monitored pavement density on 15 projects in Texas over a two-year period. Based on previous studies, some of which have been discussed in this document, the following factors were suggested as affecting the ultimate pavement density (*58*):

1) "Degree of initial compaction
2) Material properties

    a) Aggregate absorption
    b) Aggregate surface characteristics
    c) Aggregate gradation
    d) Asphalt temperature-viscosity relationship
    e) Asphalt susceptibility to hardening
3) Mix design
    a) Asphalt content (film thickness)
    b) Voids in mineral aggregate
4) Weather conditions
    a) Air temperature variations (daily and seasonal)
    b) Date of construction
5) Traffic
    a) Amount
    b) Type
    c) Distribution throughout year
    d) Distribution throughout day
    e) Distribution in lanes
6) Pavement thickness."

The authors state (*58*), "The initial density of the pavement is dependent on the compactibility of the mix or the ease with which it can be compacted, the type of compaction equipment, the rolling sequence and procedure, and the timing of the compaction process."

Cores were taken from the sites after 1 day, 1 week, 1 month, 4 months, 1 year, and two years. Figure 2.15 indicates that pavements compacted to a higher initial density densified less under traffic than pavements compacted to a lower initial density did. The authors note the importance of season of construction as a pavement constructed in the fall or early winter will not densify until the onset of warm weather. Little densification was observed during colder months. The authors recommend the use of ESALs to account for the percentage and weight of trucks in the traffic stream. Figure 2.16 shows densification as a function of ESALs. The authors concluded that "Eighty percent of the total 2-year compaction, due to traffic and environmental effects, was complete within 1 year of service on all of the projects studied." They also noted that the ultimate pavement density (for a given project) tended to converge, even if the initial density varied.

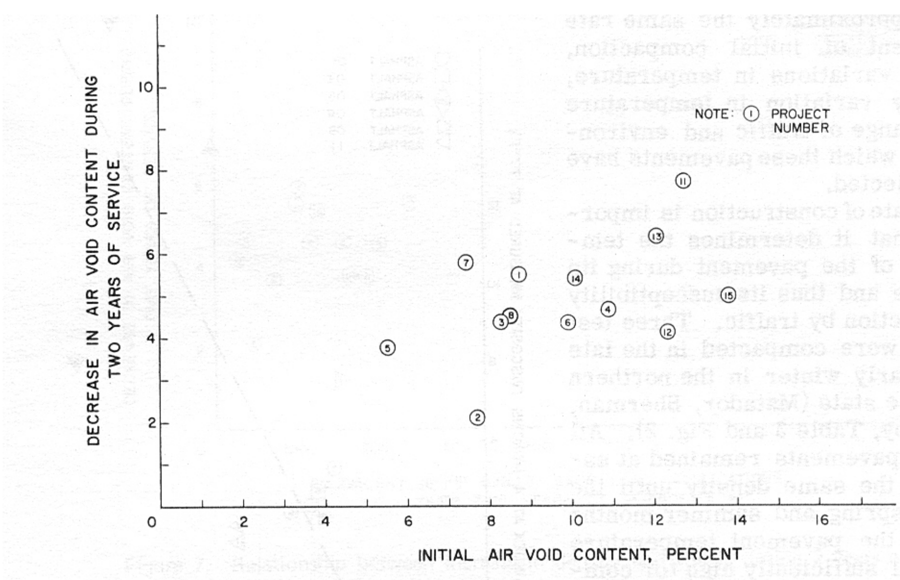

Figure 2.15. Densification as a Function of Initial Density (*58*).

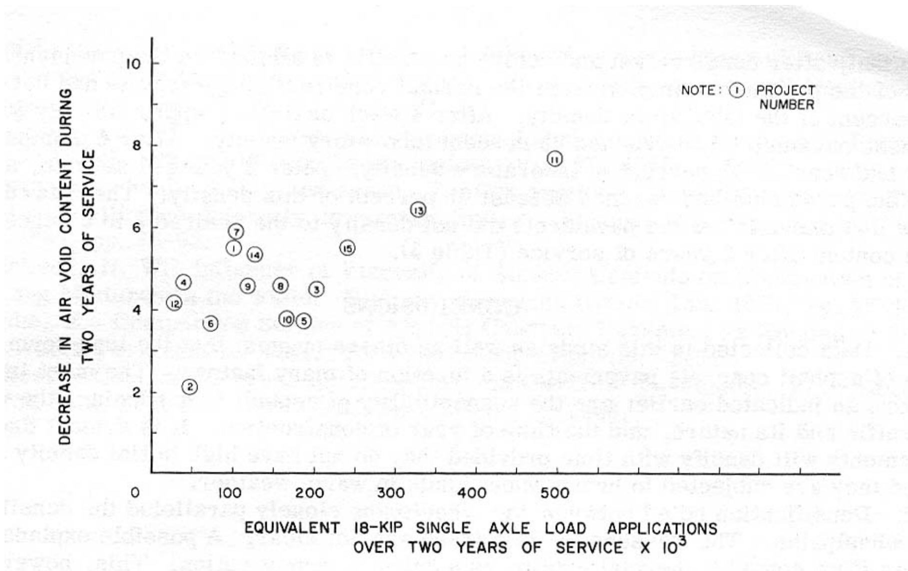

Figure 2.16. Densification versus ESALs (*58*).

Kandhal and Wenger (*59*) tracked the density and binder properties of 6 pavements in Pennsylvania over a 10-year period. The densification of the projects appears to have leveled off after a 4-year period. However, some densification continued on three of the projects up until 10 years. The authors suggest the use of a hyperbolic function to predict ultimate density based on early density measurements and indicate good results when this method was fit to the experimental data.

Brown and Cross (*60*) sampled 18 different pavements in 6 states. Thirteen of the projects rutted prematurely and 5 performed satisfactorily. The age of the rutted pavements ranged from 1 to 6 years, while the age of the satisfactory pavements ranged from 5 to 16 years old. Cores were taken from the sites and samples recompacted in the laboratory. The authors recommend dividing the in-place unit weight from cores by the recompacted unit weight to determine the relative amount of densification that has occurred. By plotting this value versus traffic, an estimate can be made of the amount of traffic required to reach the laboratory recompacted density.

Weak trends were noted between the 20th percentile of the in-place density and the accumulated traffic for both the surface and second layer of the pavement structure. Trends were also observed between the ratio of the in-place unit weight to the laboratory recompacted unit weight versus traffic for both the Corps of Engineers GTM and 75-blow Marshall samples. The best trend (R^2 = 0.50) was for the second lift recompacted with a 75-blow Marshall.

Hanson et al. (*61*) revisited 5-pavement sections that were included in the Asphalt-Aggregate Mixture Analysis System study (*32*), 5 years after construction. Pavement densities were monitored for a two-year period as part of the original study. A statistical comparison was performed between the measured densities at 2 and 5 years. The comparisons indicated significant differences in 20 out of 30 cases analyzed. As expected, in 16 of 20 cases where significant differences occurred, the air voids after 5 years of service were less than that after 2 years of service. It should be noted that of the 5 projects, 1 was a surface course, 2 were intermediate courses and 2 were base courses.

Stroup-Gardiner et al. (*62*) reported on a 5-year study of 16 projects in Minnesota representing a wide range of traffic loadings. For low volume roads (average daily traffic less than 10,000), the majority of any densification occurred in the first year after construction. For high volume roads, the authors found a decrease in density with time, which they attributed to moisture damage.

Brown and Mallick (*63*) reported on a 3-year study, which evaluated the densification of 6 projects in 5 states. Cores were taken from the projects at the time of construction and 1, 2 and 3 years after construction. An examination of the data indicates one project reached its ultimate density after 3 years, one project on a very low traffic road showed little change and the remaining 4 projects indicated additional increases in density between years 2 and 3. In summary, the literature seems to indicate that the majority of pavement densification under traffic occurs in the first 2 years. However, continued densification has been observed up to 4 and in some cases even 10 years after construction.

2.4 STUDIES RELATED TO Ndesign

2.4.1 Development of the Original Ndesign Table

The original Ndesign experiment was conducted by the Asphalt Institute as Task F of SHRP contract A001 (*64*). The experimental design was primarily developed by the Mixture Design and Analysis System (MiDAS) group consisting of: Ronald Cominski, Gerald Huber, Harold Von Quintus, and Matthew Witczak. The goal of the experiment was to determine the number of gyrations to 1) match the ultimate in-place density, targeted as 96 percent density (Ndesign), and 2) match the as-constructed density, targeted as 92 percent density (Nconstruction). The specifications for the SHRP Gyratory Compactor were discussed previously (*29*). Sections from the Long-Term Pavement Performance (LTPP) Studies General Paving Sections (GPS) were selected to determine Ndesign and Nconstruction. The in-place density at the time of construction was unknown for the GPS sections, so 92 percent density was assumed. This assumption was not expected to significantly affect the Nconstruction gyrations since only approximately 30 gyrations would be required to obtain 92 percent density.

Three hypotheses were identified for the experiment (*64*):

1. There was a correlation between lab compaction and field compaction,
2. There was a correlation between lab compaction with the gyratory compactor and field compaction (construction and traffic),
3. There was a linear correlation between an adjustable compaction parameter of the SGC and the density of the field cores.

The experiment was conducted as follows (*64*):

1. Select sites,
2. Collect cores and existing data on cores from Material Reference Library,
3. Separate Cores into paving lifts,
4. Measure bulk specific gravity of each lift,
5. Extract binder and recover aggregate,
6. Remix recovered aggregate with AC-20, short term age, and recompact,
7. Measure bulk specific gravity and maximum specific gravity of reconstituted mix,
8. Plot densification curves (gyrations versus density),
9. Tabulate and analyze data,
10. Recommend Ndesign values.

The experimental matrix is shown in Table 2.2. Two replicates (different pavements) were desired for each cell. The selected pavements were to be at least 12

TABLE 2.2 Experimental Matrix for Original Ndesign Experiment (*64*)

Lift / Traffic	Temperature								
	Hot (≥ 100°F)			Warm (≤ 90 < 100°F)			Cool (< 90°F)		
Traffic	Low	Medium	High	Low	Medium	High	Low	Medium	High
Upper	X	X	X	X	X	X	X	X	X
Lower	X	X	X	X	X	X	X	X	X

years old to ensure that they had reached their ultimate density. Only single replicates (sites) could be identified for the hot climate. Low traffic was defined as 20-year design traffic less than 1 million ESALs; medium traffic was defined as greater than 1 million to less than or equal to 15 million ESALs; and high traffic was defined as greater than 15 million ESALs. The 20-year design traffic was calculated according to Equation 3.

$$20\ Year\ DesignTraffic = 20 \times \left(\frac{Accumulated\ Traffic,\ ESALs}{Total\ Years\ in\ Service} \right) \tag{3}$$

The maximum design traffic included in the experiment was 32.1 million ESALs.

Fifteen, 12-inch diameter cores were collected for testing, one from each project. Two 4-inch diameter samples were compacted from each of the two selected lifts from each project. After completing the first round of compaction, the Asphalt Institute realized that the Rainhart SHRP Gyratory Compactor had erroneously been set to an angle of 1.3 degrees and not the 1 degree angle specified. Therefore, the compacted samples were re-extracted, remixed with virgin AC-20 and recompacted in the Rainhart Gyratory Compactor, now set to an (external) angle of 1 degree. No discussion was provided on the possible effects from aggregate breakdown which may have occurred during the first compaction cycle.

It was observed that the sample bulk specific gravities determined with ASTM D 2726 were approximately 2 percent higher than those estimated using the SGC sample height and mold diameter [Reference (*64*) actually says the reverse, but this is an error]. Two gyration levels were picked off of the plots of corrected sample density versus number of gyrations: Nconstruction = 92 percent density and Ndesign = the in-place pavement density. This author notes that the in-place density for two of the lifts, one upper and one lower, were less than 92 percent density after more that 12 years of traffic. No relationship was observed between traffic and gyrations for the lower lift. Therefore, the determination of Ndesign for the lower lifts was not reported. Figure 2.17 shows a comparison between the Ndesign levels determined at an angle of 1 and 1.3 degrees.

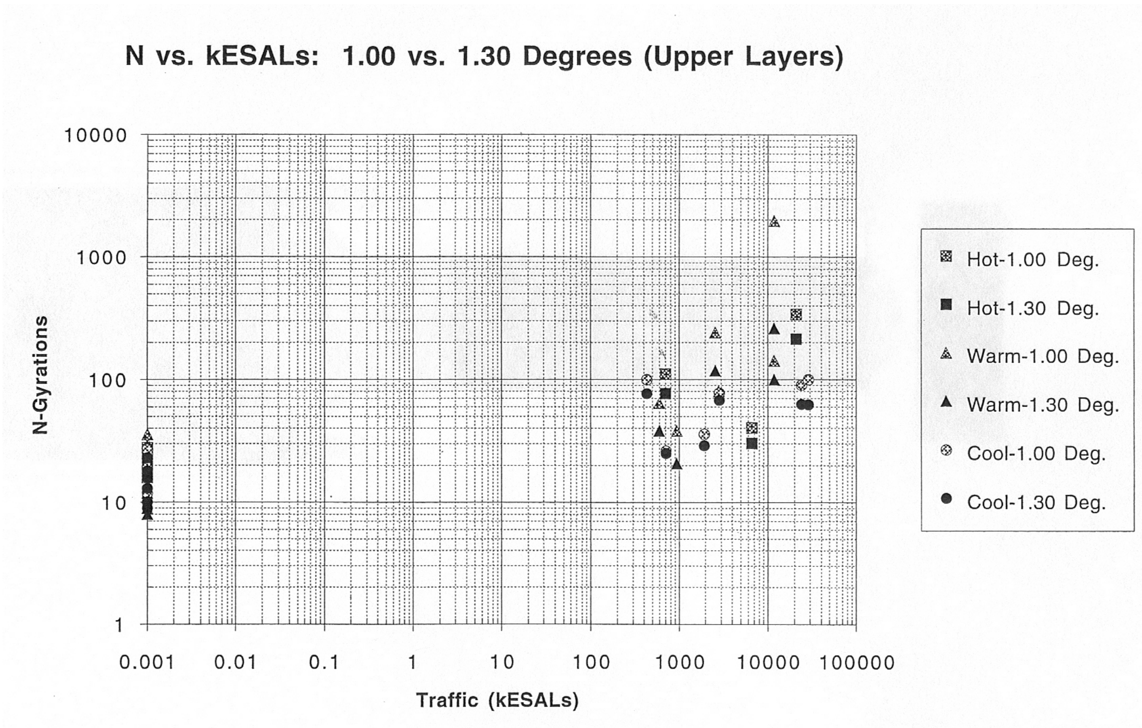

Figure 2.17. Comparison of Ndesign from Angles of 1 and 1.3 Degrees (*64*)

The complete data set consisted of 30 data points representing two gyratory samples from each of 15 pavements, 3 hot, 6 warm, and 6 cool. Linear regressions were performed between the logarithm (Log) of gyrations and the Log of 20-year ESALs. Regressions were performed on the whole data set, and the data set subdivided by climate. One sample, with an in-place density of 99.6 percent, was removed from the 6 warm climate data as an outlier. This level of density was not obtained after 230 gyrations. The models, subdivided by climate were recommended and are shown below with their pertinent statistical parameters (Table 2.3). The lack of fit statistic was not significant for this model. The climatic zones were redefined as average 7-day high temperatures of 44, 39, and 34 °C, respectively, for the hot, warm, and cool climates. Seven traffic ranges were identified, ranging from less than 0.3 to greater than 100 million ESALs.

TABLE 2.3 Ndesign Models (*64*)

Climate	Model	R^2	ANOVA *P-value*
Hot	$Ndesign = 10^{1.34276+0.10850 \times Log\ (Traffic,\ ESALs)}$	0.66	0.05
Warm	$Ndesign = 10^{1.26454+0.11206 \times Log\ (Traffic,\ ESALs)}$	0.69	0.00
Cool	$Ndesign = 10^{1.21211+0.09148 \times Log\ (Traffic,\ ESALs)}$	0.72	0.00

Note: analysis of variance (ANOVA)

It is clear that this was a limited experiment. It is noted that the MiDAS group desired to provide the best estimate possible, considering the time available and realized that future research would likely be needed to verify the estimates (*64*).

The next step in the development of the original Superpave Ndesign table was the determination of the numbers of gyrations for Ninitial (then termed N_{89}) and Nmax (then termed N_{98}) for each of the traffic levels and climatic zones (*29*). This was accomplished by translating the original compaction curves horizontally until the density at Ndesign corresponded to 96 percent (Figure 2.17). This translation is based on some of the principles investigated by Moultier (*35*). The ratio of Log (Nmax) to Log (Ndesign) and the ratio of Log (Ninitial) to Log (Ndesign) was determined for each compaction curve. The average ratios were 0.47 and 1.22 for Ninitial and Nmax, respectively. Based on this work, SHRP recommended the following equations (*29*):

$$Log\ \ Ninitial = 0.45 \times Log\ \ Ndesign \tag{4}$$

$$Log\ N\max = 1.15 \times Log\ Ndesign \qquad (5)$$

The density at Ninitial was specified as less than 89 percent to prevent tenderness during compaction and the density at Nmax was specified as less than 98 percent to prevent rutting at the end of service life.

An experiment was conducted to evaluate the SGC for field control (*29*). Changes in asphalt content, percent passing the 0.075 mm sieve, percent passing the 2.36 mm sieve, NMAS, and the ratio of natural to crushed fine aggregate were experimental variables. A partial factorial experiment was performed. Asphalt content, percent passing the 0.075 mm sieve, and the ratio of natural to crushed fine aggregate all had significant effects on the compaction curve. Based on this experiment, the SHRP researchers recommended the SGC for field control.

Finally, the prototype SHRP gyratory compactor was used to design 7 mixtures for nine pilot SPS-9 projects in 4 states: Arizona, Indiana, Maryland and Wisconsin. The sections were constructed in 1992 and 1993. Cominski et al. (*29*) state, "Although the original gyratory design specified an angle of gyration of 1°, a vertical pressure of 0.6 MPa (87 psi), and 30 rpm, problems were encountered on some SPS-9 mix designs. It became apparent that the 1° angle of gyration provided insufficient compaction effort for the air voids required at N_{design}." An example is provided for the Arizona SPS-9 project. The measured density at Ndesign was 90.8 and 92.0 percent, respectively for an (external) angle of 0.97 and 1.27 degrees at trial asphalt content of 4.1 percent. Thus the estimated asphalt content to achieve 4 percent air voids at Ndesign would have been 6.2 and 5.7 percent, respectively, at an (external) angle of 0.97 and 1.27 degrees. It is expected, but not stated, that the specified angle of gyration for the SGC was increased to 1.25 degrees due to concerns about the higher than expected design asphalt contents (*29*).

Table 2.4 presents the original Ndesign table. This author has never seen documentation of the decision to go from the three climatic levels presented by Blankenship (*64*) to the four levels provided in the original table. The Ndesign gyration levels for the 43 to 45 °C climate match the gyrations levels for the hot climate determined by Blankenship (*64*). The remaining levels appear to be interpolated.

TABLE 2.4 Original Ndesign Table (*1*)

Traffic (ESALs)	Design 7-day Maximum Air Temperature (°C)											
	< 39			39 - 41			41 - 43			43 – 45		
	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}
$< 3 \times 10^5$	7	68	104	7	74	114	7	78	121	7	82	127
$< 1 \times 10^6$	7	76	117	7	83	129	7	88	138	8	93	146
$< 3 \times 10^6$	7	86	134	8	95	150	8	100	158	8	105	167
$< 1 \times 10^7$	8	96	152	8	106	169	8	113	181	9	119	192
$< 3 \times 10^7$	8	109	174	9	121	195	9	128	208	9	135	220
$< 1 \times 10^8$	9	126	204	9	139	228	9	146	240	10	153	253
$> 1 \times 10^8$	9	143	235	10	158	262	10	165	275	10	172	288

Samples were to be compacted to Nmax and the density at Ndesign and Ninitial back calculated using the sample heights recorded by the SGC (Equation 6).

$$Density\ at\ Gyration\ n = Density\ at\ N\max \times \frac{Height\ at\ N\max}{Height\ at\ Gyration\ n} \qquad (6)$$

This is a simplified version of Equations 3-6, 3-7, and 3-8 presented by Cominski (*1*), produced by combining terms.

2.4.2 Research Related to Ndesign Conducted after SHRP

Following the completion of SHRP and the release of the Superpave mix design system, a number of studies have been conducted to compare the results of the Superpave mix design system to previously used design systems (such as Marshall or Hveem) and to refine the Ndesign levels. Sousa et al. (*65*) report on an early application of the performance based Superpave design on a project on Interstate 17 north of Phoenix, AZ. Two, 1 mile test sections were placed by the Arizona DOT. The mix was a three inch layer of a 19.0 mm NMAS mixture which was to be designed for 10 million ESALs in a 10-year design life. Rutting was to be limited to less than 10 mm over the design life. This appears to be the same mix discussed previously by Cominski et al. (*29*), which resulted in the angle for the SGC being increased from 1 to 1.25 degrees.

A fine-graded mixture was selected using a crushed gravel aggregate source with 95 percent one face crushed and 90 percent two face crushed. The mixture was produced with a modified PG 70-10 binder. The optimum binder content was selected based on tests with the repetitive simple shear test at constant height (RSST-CH) test conducted on the simple (later called Superpave) shear tester. The authors applied a factor of 8.97 to the design traffic of 10 million ESALs to determine a traffic level of 89.7 million ESALs with 95 percent reliability. Using this traffic level and a plot of asphalt content versus applied ESALs resulting in 10 mm of predicted rutting based on the tests conducted with the RSST-CH, an optimum asphalt content of 4.2 percent was selected. The RSST-CH tests appear to have been conducted at 3 asphalt contents, 4.0, 4.5, and 5.0 percent. By comparison, testing performed with the SGC on field mix resulted in 6.3 percent air voids at an Ndesign of 135 gyrations and 75-blow Marshall compaction effort, then used by

Arizona DOT, also resulted in 6.3 percent air voids. An optimum asphalt content of 5.2 percent was predicted with the SGC and later verified at 5.1 percent. (This author notes that an optimum asphalt content of 5.0 percent would have been determined using the design traffic of 10 million ESALs (50 percent reliability)). The authors conclude that samples compacted using rolling wheel compactor best match the performance properties of the field cores based on comparisons made with samples compacted in the California Kneading Compactor, Texas Gyratory Compactor, 2 SHRP Rainhart compactors and the Marshall Hammer.

Harman et al. (*66*) reported on testing conducted by the FHWA Office of Technology Applications (OTA) Mobile Laboratory. The lab conducted tests on four state agency paving projects to demonstrate field control with a prototype SGC. Comparisons were performed between SGC and Marshall compacted samples. A unique relationship was found between SGC and Marshall sample air voids for each project. Ndesign of 100 gyrations produced samples with lower air voids than 6-inch diameter 112-blow Marshall compaction did. The same held true for comparisons between Ndesign of 126 gyrations and 50-blow Marshall and comparison between Ndesign of 113 and 75-blow Marshall samples.

Gowda et al. (*67*) conducted a study to evaluate the sensitivity of volumetric properties and optimum asphalt content to the Superpave Ndesign levels resulting from variations in design traffic and climate. The authors were concerned by the small differences in Ndesign between some traffic and climate levels (Table 2.4). Four aggregate gradations were selected for the study; all coarse graded (passing below the restricted zone). Two aggregate sources were used in the study: a granite source accounted for three of the blends and a sandstone source was used for the fourth blend. Two binders were used in the study, a PG 64-22 and a polymer modified PG 76-22. Samples were compacted at three asphalt contents, 4.5, 5.5, and 6.5 percent.

Three replicate samples of each of the 24 combinations (4 mixes x 2 binders x 3 asphalt contents) were compacted to 288 gyrations (Nmax for > 100 million ESALs with a 7-day maximum air temperature of 43 to 45 °C). The volumetric properties at the 27 Ndesign levels were back calculated from these samples. Figure 2.18 shows the calculated VMA as a function of Ndesign. Note that for a given gradation, VMA changes by approximately 0.3 percent for a change in Ndesign of 10 gyrations.

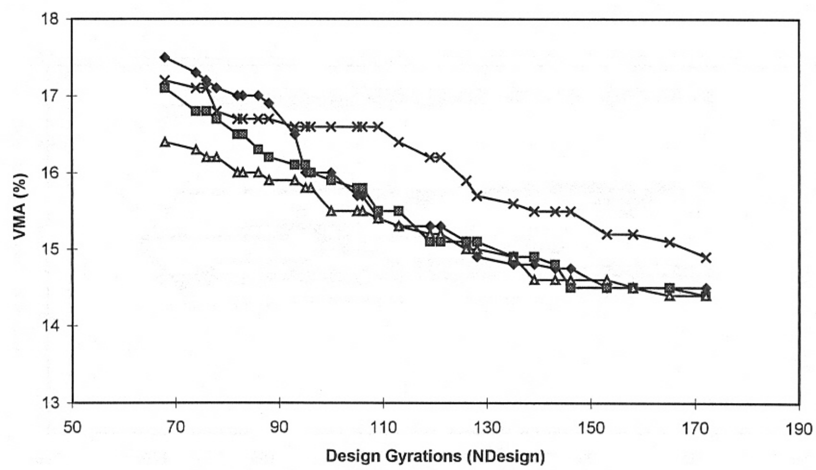

Figure 2.18. Variation in VMA with Ndesign for PG 64-22 (*67*).

Statistical analyses were conducted to compare the volumetric properties between 6 gyration levels that only varied by 1 to 2 gyrations (e.g. 95 and 96) and the mean mix design properties for the 4 climates. For the comparison of close gyration levels, statistically significant differences were observed 3 of 64 cases for VMA and for 2 of 64 cases for optimum asphalt content. For the comparison of the different gyration levels resulting from different climates, significant statistical differences were observed in 35 of

168 cases for VMA, 2 of 168 cases for optimum asphalt content and 8 of 168 cases for VFA. The authors concluded that Ndesign levels for differing design traffic which differ by 1 to 2 gyrations do not result in significantly different mix properties and that Ndesign levels from differing climates do not result in significantly different mix properties for a given traffic level.

Habib et al. (*68*) compared the Superpave and Marshall design procedures for the design of shoulder mix in Kansas. Five 19.0 mm NMAS blends were evaluated, produced from 4 aggregate stockpiles. The percentage of crushed limestone coarse aggregate was held constant and the percentage of coarse river sand varied in 5 percent increments to produce the 5 blends. All five gradations were coarse graded. Mixtures were prepared with an AC-10 (approximately PG 58-22). Samples were compacted in the SGC to Nmax = 104 gyrations. Volumetric properties were back calculated at Ndesign = 68 gyrations. Four of the five blends, evaluated using the SGC, failed VFA on the low side; the fifth failed dust to effective asphalt content on the low side. Marshall samples were compacted with a 50-blow effort for comparison. The Marshall samples met all of the Kansas DOT's criteria. It was observed that the optimum asphalt contents, VMA and VFA were all lower for the samples compacted in the SGC. The authors speculate that the Superpave Ndesign levels for low volume pavements are approximately 20 percent too high.

Mallick et al. (*69*) reported on the effect on volumetric properties of the restricted zone from mixes produced with crushed and partially crushed fine aggregate and the effect of back calculation on the volumetric properties of samples compacted in the SGC. As discussed previously, when Superpave was first adopted, samples were compacted to

Nmax and the volumetric properties back calculated at Ndesign. The back calculation uses a correction factor which is the ratio of the measured Gmb using AASHTO T166 to the Gmb calculated with the measured sample mass and estimated sample volume calculated based on the area of the gyratory mold (176.7 cm^2) times the sample height recorded by the SGC, cm. Testing conducted with dense and SMA gradations produced with a traprock aggregate indicated that the correction factor varied with the number of gyrations the sample was compacted to. In essence, the sample has more surface texture at lower gyration levels, resulting in a smaller measured volume. Figure 2.19 shows the error in measured air voids. Note that the back calculated air voids are higher than the air voids measured at a given Ndesign level, particularly for coarse graded mixes. This resulted in a slight reduction in optimum asphalt content for samples compacted to Ndesign as opposed to those compacted to Nmax where volumetric properties were back calculated at Ndesign.

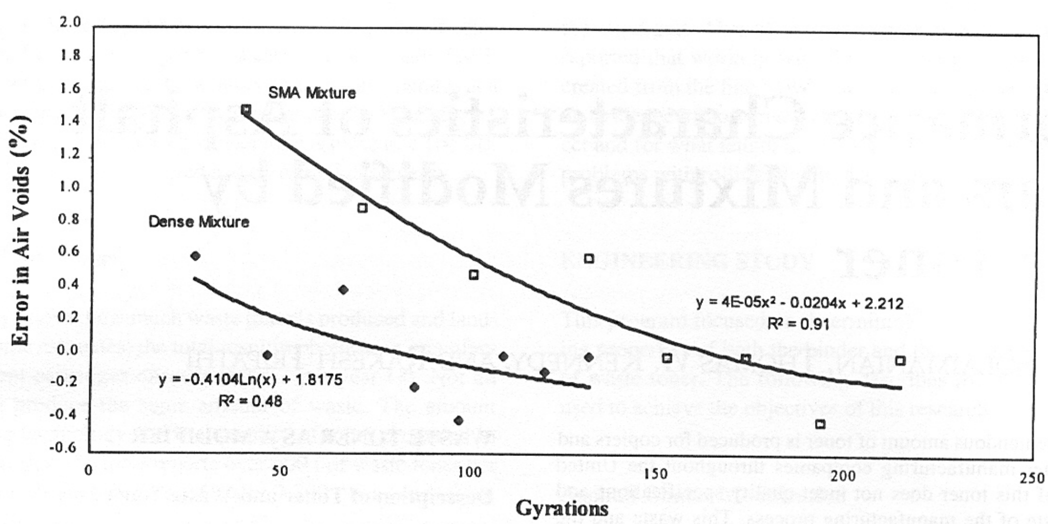

Figure 2.19. Error in Back Calculated Air Voids Versus Gyration Level (*69*).

Brown and Mallick (*63*) reported on a preliminary study to evaluate the Ndesign Table. Loose mix, aggregate and asphalt, and cores were sampled from six projects in five states in 1992 and 1993. The projects were located in Alabama (2), Idaho, New Mexico, South Carolina, and Wisconsin. The field mix and laboratory mix produced to match the field mix were compacted to a number of gyrations which produced approximately 99 percent density with an SGC. Samples were also compacted using 75-blows of a fixed base mechanical Marshall Hammer. A set of 12 cores were obtained at the time of construction and 12, 24 and 36 months after construction.

Good correlations were observed between the Log of accumulated ESALs and pavement density for 4 of 6 projects. The New Mexico project produced an $R^2 = 0.52$. This author notes that this may be related to the polymer modified AC 40 used for the project. The remaining projects used AC-20 or softer binders. The one of the two Alabama projects with a poor correlation received very little traffic, approximately 112,000 ESALs after 3 years.

On average, the reheated mix was observed to have approximately 1 percent lower density than the laboratory prepared mix did. The difference decreased with increasing gyration levels. The average of the reheated field mix and the laboratory prepared mix were used to estimate Ndesign for each project. The results from one project, I-90 in Idaho, were discarded since it began to rut after two years. The Ndesign values from this study predicted to match the in-place density after three years were approximately 30 gyrations less than those determined during SHRP. (This author notes that some of this difference might be attributed to the 1 degree angle used during SHRP and the 1.25 degree angle used in this study). The SGC samples had approximately 1.5 percent higher density than the 75-blow Marshall samples.

Forstie and Corum (*70*) performed an initial evaluation of Ndesign for the Arizona DOT. The authors note three concerns about the SHRP Ndesign experiment:

1. The angle of gyration used to develop the original Ndesign table was 1 degree, but an angle of gyration of 1.25 degrees was later selected by SHRP without modifying the Ndesign table,
2. The original Ndesign experiment was performed using 100 mm diameter specimens whereas SHRP later specified 150 mm diameter samples,
3. The mixes used in the original Ndesign study were predominately fine graded whereas coarse graded mixes were more predominant when Superpave was first implemented,
4. The Ndesign study was based on only two cores per project (actually one (*64*), there were two cores per cell except for the hot climate).

The authors present a comparison of the Ndesign levels determined in the original Ndesign experiment (Table 2.5) based on Reference (*64*) for angles of gyration of 1 and 1.3 degrees. Notice that Ndesign is between 27 and 46 gyrations less at an angle of gyration of 1.3 degrees.

TABLE 2.5 Comparison of Ndesign Levels for Hot Climate for 1 and 1.3 Degrees (*70*)

Design Traffic (Million ESALs)	Predicted Ndesign	
	External Angle = 1.30°	External Angle = 1.0°
0.5	64	91
3.0	77	111
10.0	87	127
30.0	97	143

Cores were taken from six in service pavements which had been subjected to 2 to 5 years of heavy interstate traffic. The in-place density was determined for the wheel path cores. The asphalt was extracted using the ignition furnace and the aggregate recovered. The actual mix correction factor for the ignition furnace was unknown. The recovered binder was remixed with binder of the same grade as had been used previously and compacted to the appropriate Nmax using a Troxler SGC after which the sample densities were back calculated at Ndesign. The Gmb values for the SGC samples were an average of 0.037 units higher or 2.3 lbs/ft^3 higher than the in-place core densities. The SGC densities were also calculated at the Ndesign value for 1.3 degrees. This reduced the difference between the laboratory compacted samples to 0.012 Gmb units or 0.7 lbs/ft^3. Two possible flaws in the study noted by the authors were 1) the ignition furnace asphalt contents were approximately 0.3 percent higher than those later obtained by solvent extraction, and 2) changes to the recovered aggregate specific gravity were noted resulting from the ignition furnace.

Buchanan (*71*) conducted much of the research which supported NCHRP 9-9, "Refinement of the Superpave Gyratory Compaction Procedure." The major objectives of this research were to determine whether, and to what extent, the Ndesign compaction matrix could be consolidated from the original 28 levels determined during SHRP, and secondly to evaluate the back calculation of Ndesign from Nmax. The first objective was evaluated by examining the effect of Ndesign on volumetric properties. An evaluation of the parameters of the SGC: gyration angle, vertical pressure, and gyration speed, was not included in this research.

An experimental matrix was developed for the research which included four aggregate sources, two gradations and six Ndesign levels. The aggregate sources included: New York Gravel, Georgia Granite, Alabama Limestone, and Nevada Gravel. Both gradations were 12.5 mm NMAS; one was fine graded, and one was coarse graded; neither passed through the restricted zone. The gyration levels consisted of the lowest (68) and highest (172) in the original Ndesign table, three intermediate gyrations levels (93, 113, and 139), and 40 gyrations. Based on previous work, it was felt that a lower level of gyrations may be required for low volume roads. A single binder, PG 64-22, was used in the experiment. Three asphalt contents were used to bracket Ndesign. The samples were compacted to Ndesign (not Nmax). Separate samples were compacted to Nmax for three Ndesign levels and compared to results from the Asphalt Pavement Analyzer. Some of the samples did not meet all of the volumetric requirements.

The data indicated that optimum asphalt content, VMA, and VFA all decreased with increasing Ndesign; the coarse-graded mixes were more sensitive that the fine-graded mixes were. ANOVA was performed to determine which of the experimental factors affected VMA. All of the main factors (e.g., Ndesign, aggregate source, and gradation) and their interactions were significant. Duncan's multiple range comparison procedure was conducted to compare the measured VMA resulting from the differing Ndesign levels. The analyses were conducted separately for the coarse-graded and the fine-graded mixes. For both gradations, the differing Ndesign levels used in this study resulted in significantly different VMA at the 5 percent significance level.

An evaluation was performed of the need for the differing gyration levels for the differing climatic zones in the Ndesign table. The argument was made that the average

7-day maximum temperature is less than 39 °C for the majority of the United States. Further, where higher temperatures exist, a stiffer binder would likely be used. Statistical comparisons were conducted using a Student's t-test between the resulting VMA calculated for each aggregate source and gradation between the Ndesign climatic extremes for a given traffic level (e.g., 68 versus 82 gyrations, respectively for < 39 and 43 to 45 °C). No significant differences were observed for 41 of 56 comparisons. For the 15 comparisons which were significant, the average absolute difference in VMA was 0.57 percent. Based on these analyses, the differing Ndesign levels as a function of climate were eliminated from the Ndesign table, collapsing the table from 28 to 7 levels.

Since the coarse-graded mixes were more sensitive to Ndesign than the fine graded mixes were, the VMA results for the coarse-graded mixes were evaluated to further consolidate the Ndesign table. The average difference in VMA between Ndesign levels was 0.32 percent for the coarse-graded mixes and 0.18 percent for the fine-graded mixes. A VMA range of 1 percent was selected for differing Ndesign levels. This would result in a difference in optimum asphalt content of approximately 0.45 percent for the coarse graded mixes. Thus three levels of Ndesign were proposed 70, 100 and, 130 gyrations. A fourth Ndesign level, 50 gyrations, was proposed for low volume roads.

None of the mixes included in this study failed the Nmax criteria. Further, it was determined that compacting samples to Nmax and back calculating the volumetric properties at Ndesign can result in errors of up to 0.8 percent air voids. Therefore, it was recommended that samples be compacted to Ndesign for the determination of volumetric properties. Separate samples could be compacted to Nmax after the optimum asphalt content is determined. Table 2.6 presents the revised Ndesign table recommended by Buchanan (71).

TABLE 2.6 Revised Ndesign Table Proposed by Buchanan (*71*)

Design Traffic Level (million ESALs)	Gyration Levels			% Gmm @ Ninitial	% Gmm at Nmax
	Ninitial	Ndesign	Nmax		
<0.1	6	50	74	< 91.5	< 98.0
0.1 to < 1.0	7	70	107	< 90.5	
1.0 to < 30.0	8	100	158	< 89.0	
> 30.0	9	130	212	< 89.0	

Anderson et al. (*3*) conducted an evaluation Ndesign based on the sensitivity of engineering properties to changes in Ndesign. This research had four tasks (originally five, but one was abandoned because it duplicated NCHRP 9-9):

1. Examine the performance of in-place Superpave pavements designed with the original SHRP Ndesign table,
2. Select a validated performance test for rutting,
3. Determine the sensitivity of the performance test to changes in Ndesign,
4. Recommend a new Ndesign table.

Six Superpave mix designs were developed using two aggregate types, crushed limestone and crushed gravel, and three Ndesign levels, 70, 100, and 130 gyrations. All of the mixes were 12.5 mm NMAS. The gradations of the three blends for each aggregate source were varied to produce a VMA slightly above the minimum (14.0 percent). This was done based on the assumption that since binder is the most expensive component of HMA, the mix designers will alter the gradation to reduce VMA as Ndesign decreases. The resulting mixes had measured VMA ranging from 14.2 to 14.6

percent and optimum asphalt contents of either 4.6 or 4.7 percent. Samples were produced with a single unmodified PG 70-22.

The rutting properties of the mixes were evaluated using two tests performed in the Superpave Shear Tester (SST): frequency sweep at constant height (FSCH) and RSCH. FSCH is conducted by applying a small shear stress to the samples which results in a shear strain of less than 0.0005. Tests are conducted at ten frequencies: 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. Highway traffic speeds are generally represented by the results at 10 Hz. The complex shear modulus (G*) is the ratio of the applied shear stress to the resulting shear strain. Higher G* values at a given temperature indicate a stiffer mix. FSCH testing was conducted at two temperatures 50 and 60 °C. RSCH is performed by applying a haversine shear stress of 69 kPa with a 0.1 second load and 0.6 second rest period (1.4 Hz) for 5000 cycles. The test result is reported as the accumulated permanent shear strain after 5000 cycles. Testing was conducted at 60 °C.

It was observed that G* (10 Hz) was significantly higher for the limestone aggregate than for the gravel aggregate. For a given aggregate, there were no significant differences between the stiffness of the mix designed at 100 and 130 gyrations. G* (10 Hz) was significantly lower for both aggregate mixtures designed with Ndesign = 70 gyrations. There was a general trend of decreasing shear stiffness with decreasing Ndesign. It was believed that this trend is related to changes in the aggregate skeleton. [Alternatively, this author believes it could be related to the degree of contact developed between the aggregate particles, similar to the results observed for the kneading compactor compared to the other compactors by Consuegra et al. (*31*) or simply more asphalt in the mixture]. For the RSCH test, the limestone aggregate was again identified as being more rut resistant. However, no significant differences were noted between the accumulated shear strain from the RSCH test for the mixes designed at different Ndesign levels. A study was also conducted to examine the sensitivity of VMA to Ndesign. Similar results to NCHRP 9-9 were noted. Finally, the authors note that based on experience, an increase in one high temperature binder grade, say from PG 70 to PG 76 will result in the same increase in mix G* as a change of 30 gyrations.

In 1999 at a meeting of the FHWA Superpave Mixtures Expert Task Group (ETG), Dr. Ray Brown and Mr. Mike Anderson presented the results of their respective studies on Ndesign. This author was a member of the ETG at that time and present at the meeting. Based on that meeting, a new Ndesign table was recommended and adopted by AASHTO in 2001. The revised Ndesign Table from AASHTO PP28 is shown below (Table 2.7) (*36*). In 2004, AASHTO PP28 was adopted as AASHTO M323 (*4*).

TABLE 2.7 Superpave Gyratory Compaction Effort (*36*)

Design ESALs (millions)	Compaction Parameter		
	Ninitial	Ndesign	Nmax
< 0.3	6	50	75
0.3 to <3	7	75	115
3 to < 30	8	100	160
≥ 30	9	125	205

In 1994, Colorado DOT initiated a study to compare the air void contents of laboratory compacted samples and in-place field projects (*72*). At the time the study was initiated, Colorado DOT was using the Texas Gyratory with variable end-point stresses for the differing traffic and environmental conditions within Colorado. Samples were taken from 25 sites at 22 projects, designed using the Texas Gyratory, and compacted in a

Pine SGC. The mix designs also met the Superpave design criteria. The projects were selected to cover a range of traffic and environmental conditions.

At the time of construction, loose mix was sampled and 3 samples each were compacted to the specified Ndesign and one level above and one level below the specified Ndesign. Fifteen cores were taken to determine the as-constructed density, 5 from the estimated position of the left-hand wheel path of the design lane and 5 cores just to the right and 5 cores just to the left of the estimated position of the left-hand wheel path. All but 3 of the 25 sites fell within the specified in-place density range of 92 to 96 percent, with an average density of 94.7 percent. Five cores were then taken from the left-hand wheel path on an annual basis for a period of five to six years. The in-place air void contents from the 3, 4, 5, and, 6 year cores did not change significantly. Therefore, it was concluded that the pavements reached their ultimate density after approximately 3 years of traffic.

Figure 2.20 shows a comparison between the laboratory compacted air voids at Ndesign and the in-place air voids after 3 years of traffic. Note from the figure that the in-place air voids are approximately 1.2 percent higher than the laboratory compacted samples at 4 percent air voids. Harmelink and Aschenbreber (*72*) in their recommendations state that the mixes are being designed at too low of an asphalt content for the environmental and traffic conditions in Colorado. Two options for adjustments suggested were: 1) lowering Ndesign and 2) adjusting the mix design air void content (less than 4 percent). It is noted that Colorado DOT uses 100 mm diameter molds in the SGC, which tend to produce lower density that 150 mm diameter molds would.

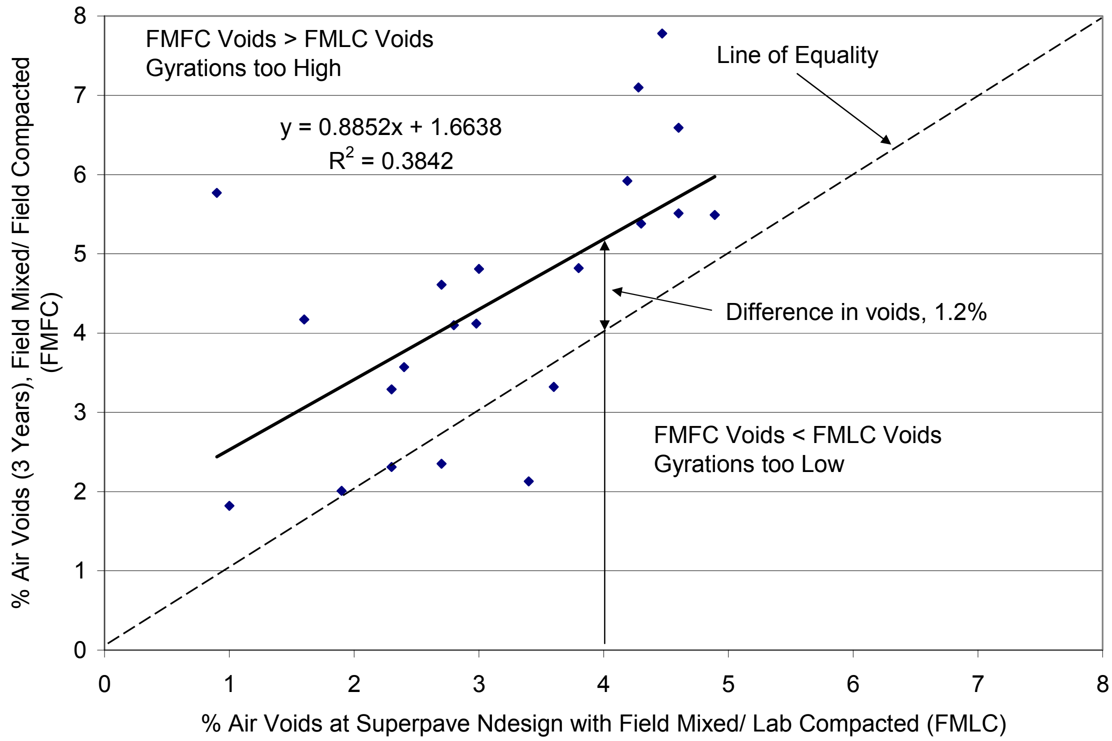

Figure 2.20. Comparison of Ndesign and In-Place Air Voids after 3 Years (*72*)

Watson et al. (*73*) conducted a study to verify the Ndesign levels for Georgia Department of Transportation. The objective of this study was to compare the performance of Georgia DOT's mixes designed using the Superpave and the Marshall mix design systems, both produced using PG binders and aggregates from the same source. From a list of 217 Marshall and Superpave projects, 16 Marshall designed and 16 Superpave designed projects were selected that matched closely in age, traffic, aggregate source, and geographical area. All of the projects were 12.5 mm NMAS. A pavement performance survey and coring was conducted at each site. Three cores were collected from each project, one in each wheel path and one from between the wheel paths. Quality control and quality assurance data were determined from historical records. Figure 2.21 shows a comparison of the in-place air voids in the wheel path. The average in-place air voids for the Superpave designed projects were 5.7 percent whereas the in-place air voids for the Marshall designed projects were 3.8 percent. Data from the quality assurance records indicated that the in-place air voids at the time of construction averaged 7.3 and 6.1 percent for the Superpave and the Marshall designed mixes, respectively. It should be noted that the Marshall and Superpave projects averaged 6.1 and 4.7 years old, respectively.

Figure 2.22 shows a comparison between the design VMA for the Superpave and Marshall designed mixes. The authors note that the average VMA for the Superpave designed mixes (14.9 percent) is almost 2 percent less than the average VMA for the Marshall designed mixes (16.8 percent). This occurred even though the gradations of the Marshall designed mixes were closer to the maximum density line than the gradations of

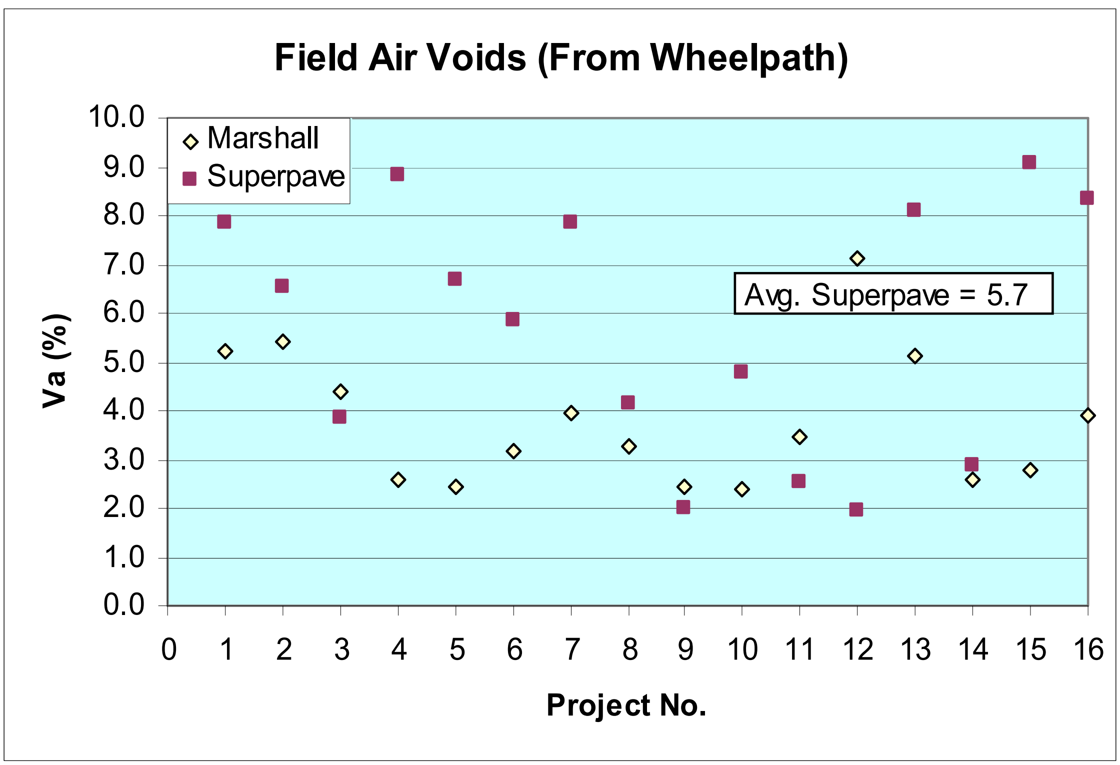

Figure 2.21. Comparison of Superpave and Marshall in-place Air Voids (*73*)

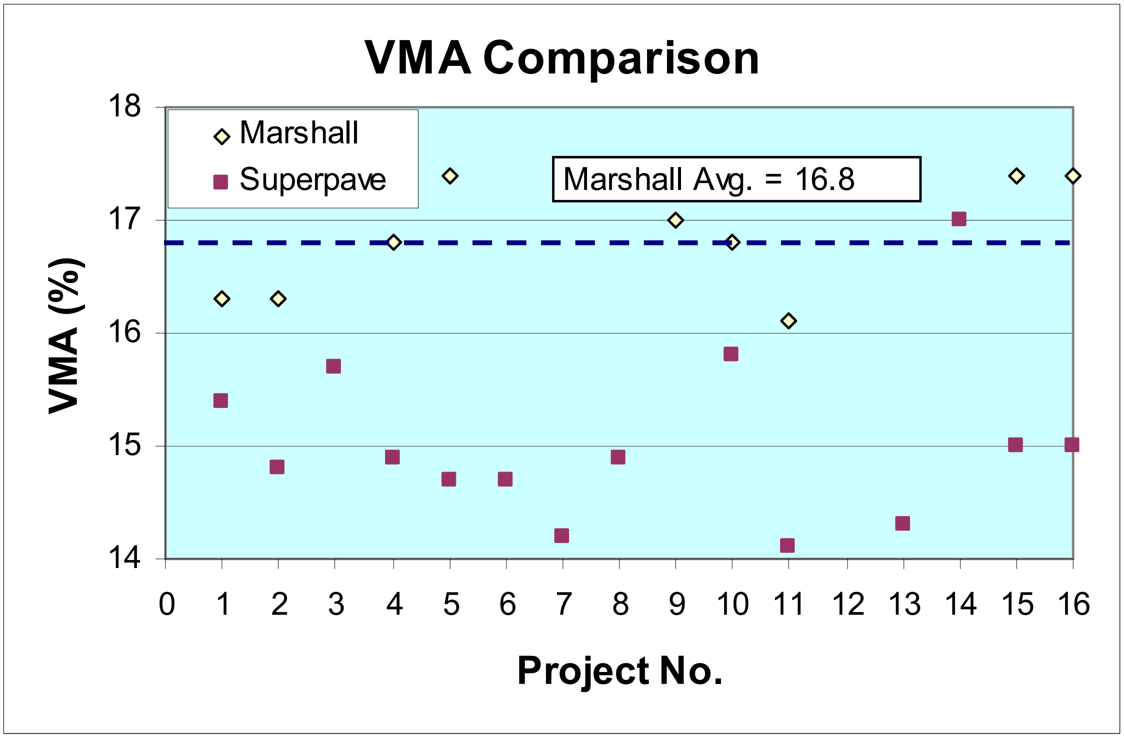

Figure 2.22. Comparison of Superpave and Marshall Design VMA (*73*)

the Superpave designed mixes were. This indicates the effect of the increased laboratory compaction effort with the Superpave mix design system. It should be noted that Georgia DOT used effective specific gravity to calculate VMA for both the Marshall and the Superpave designed mixes. The difference in design VMA resulted in the average asphalt content for the Superpave designed mixes being 0.34 percent less than that for the Marshall designed mixes.

Locking Point

Pine (*74*) proposed the "Locking Point" concept for the SGC. The locking point was likened to the growth curve conducted to determine the maximum number of roller passes in the field before the increase in in-place density leveled off or decreased. It was noted that mixes are not compacted with the same number of passes in the field because each mix is different. Rolling was stopped at the peak density before excessive aggregate degradation occurred.

The locking point concept was developed from comparisons made between three years of Marshall and Superpave data and field growth curves. Initially, the locking point was defined as the first gyration in a set of three gyrations of the same height which were preceded by two gyrations of the same height (0.1mm taller). It was believed to indicate the development of some degree of coarse aggregate interlock and be related to the density achieved in the field growth curves. It was noted that the standard deviation of the number gyrations equal to the locking point was less than the standard deviation of the number of gyrations to obtain 4 percent air voids.

Vavrick and Carpenter (*75*) discuss errors in the back calculated density from samples compacted to Nmax. A refined definition of the locking point is also presented where the locking point is defined as the first gyration in the first occurrence of three gyrations of the same height proceeded by two sets of two gyrations with the same height (each 0.1 mm taller) as illustrated in Table 2.8.

TABLE 2.8 Sample Gyratory Height Data Illustrating Locking Point Determination (*75*)

Gyration	1	2	3	4	5	6	7	8	9	10
60	111.9	111.9	111.8	111.8	111.7	111.7	111.6	111.6	111.5	111.5
70	*111.4*	*111.4*	*111.3*	*111.3*	***111.2***^{LP}	*111.2*	*111.2*	111.1	111.1	111.0
80	111.0	110.9	110.9	110.8	110.8	110.8	110.7	110.7	110.7	110.6

2.5 SUMMARY OF LITERATURE REVIEW

The first HMA (actually sand asphalt) was placed in the United States in 1876. Initially, optimum asphalt content was selected by experience. Several proprietary mixes were developed, and widely used. As the popularity of HMA grew, there developed a need for standardized tests to assist with the design and control of HMA. This was partially due to the fact that there were no longer enough experienced individuals to make decisions regarding the adequacy of a mix (*5, 6*).

One of the first tests applied to the determination of optimum asphalt content was the pat test, basically a visual assessment of the residual asphalt on a piece of Manila paper which had been pressed into a fresh sample of HMA (*9*). Hveem (*5*) recognized the relationship between aggregate gradation and optimum asphalt content, finer mixes generally require higher optimum asphalt contents because they have more surface area. In the 1930's researchers began to look for a laboratory compaction procedure which would produce sample densities similar to the ultimate density of the in-place pavement. Pavements were observed to densify under traffic for a period of 2 to 3 years or more. Later this search was expanded to include a laboratory compaction procedure which would produce samples with the same mechanical properties as field-compacted HMA (*5, 12, 14, 15, 21, 22*).

The most widely recognized study of this nature was that conducted by the Corps of Engineers during the development of the Marshall mix design procedure. More than 214 test sections representing 27 mixes were placed and tested with accelerated loading. Three wheel loads were used: 15,000, 37,000 and 60,000 lbs; 3500 passes were applied with the 15,000 lb load and 1500 passes with the remaining two loads. The filler content and asphalt content of each mixture were each varied at three levels. Based on field performance, optimum asphalt content for each mixture was recommended. The laboratory compaction effort that produced an optimum asphalt content that best matched those determined in the field was 50-blows (*14, 15*).

Hveem (*5*) placed less emphasis on sample air voids and more emphasis on stability, but did recognize the importance of air voids as they relate to durability. Texas conducted studies with the Texas Gyratory Compactor during the 1940's to verify that the laboratory compaction effort matched the ultimate pavement density. The density of cores taken 1 to 12 years after construction averaged 0.8 percent lower than the laboratory samples. The Corps of Engineers developed the GTM in response to even higher (up to 350 psi) tire pressures on military aircraft (*12, 25, 26*).

A general summary of the early design philosophies might be that HMA should be designed with the highest asphalt content (for durability) which does not result in stability or rutting problems. Marshall emphasized the importance of minimizing VMA by using the densest aggregate structure possible (*6*).

Numerous studies were conducted to monitor the densification of pavements, in situ (*14, 32, 50 – 63*). Generally, pavements were believed to reach their ultimate density under traffic after 2 to 3 years, with most of the densification occurring in the first year. Some studies observed densification over a longer period of time (up to ten years). Attempts were made to relate field densification to laboratory compaction, particularly with the Marshall method.

In the late 1970's and 1980's, rutting problems became more prevalent in the United States. This is somewhat attributed to the use of radial tires and increased tire pressure on trucks. To address these concerns, 50 million dollars was devoted to asphalt research in the SHRP program authorized by the Surface Transportation and Uniform Relocation Assistance Act of 1987 (*1*). Superpave was a product of the SHRP research program.

The gyratory compactor was selected for routine use in the Superpave mix design system for it ability to 1) produce samples with similar mechanical properties as field compacted HMA, and 2) for its convenience (*29, 31, 34*). Further, the French indicated a relationship between the number of gyrations and the layer thickness and number of roller passes in the field. The operational characteristics of the French Gyratory Compactor were adopted, with the exception that the speed of gyration was increased to 30 rpm (*28*).

An experiment was conducted during SHRP to determine Ndesign (*29, 64*). The premise of the experiment was three-fold, 1) there was a relationship between pavement densification and accumulated traffic, 2) there was a relationship between the densities of samples compacted in the SGC and in-place density, and 3) there was a linear relationship between Ndesign and design traffic. Fifteen pavements representing three climatic regions and three traffic levels were cored (one core each) which had been in service for more than 12 years. The density of the cores was measured and the asphalt extracted to recover the aggregate. The density at the time of construction was unknown and assumed to be 92 percent. No relationship was observed between pavement density and traffic for the lower lifts (> 100 mm); therefore these samples were not tested (*64*).

The recovered aggregate was remixed with virgin asphalt and two samples compacted to 230 gyrations for each mix. The number of gyrations which matched the in-place density was back calculated. A relationship was developed between design traffic (ESALs) and Ndesign. However, it was found that the angle of gyration of the SGC was 1.3 degrees not the specified 1.0. Therefore, the aggregates were again recovered, remixed and compacted in the SGC, now set to an angle of gyration of 1 degree. From this a table of Ndesign levels for three climates and 7 traffic levels was developed (*29, 64*). Later the SHRP researchers expanded this table to 4 climates (*29*). Late in SHRP, the angle of gyration was changed to 1.25 degrees. The Ndesign levels were not altered at this time even though angles had been demonstrated to affect Ndesign (*29*).

When Superpave was first released, researchers and agencies compared the results from the Superpave system using the SGC to the design systems they were familiar with, most frequently the Marshall system. The SGC was found to generally produce lower VMA, air voids and therefore lower optimum asphalt contents than the Marshall system did (*63, 66, 68, 70*).

Research indicated that significant differences did not exist between mix properties resulting from many of the Ndesign levels which were close together (*67, 71*). Errors were observed between the density at Ndesign back calculated from Nmax, as originally recommended in the Superpave system, and the density of samples compacted to Ndesign (*69, 71*). Significant research was conducted to confirm these findings which resulted in a consolidation of the Ndesign table from 28 to four levels and a change in practice from compacting samples to Nmax and back calculating volumetric properties at Ndesign to simply compacting samples to Ndesign for volumetric property determination. However, the consolidation of the Ndesign table was primarily based on sensitivity of volumetric properties and performance test results related to rutting of laboratory produced mixtures, not relationships with field performance (*2, 3, 71*).

Colorado DOT conducted a study that indicated that in-place air voids after 5 to 6 years of traffic were higher than those obtained at Ndesign using the SGC. Lower design gyrations or design air void contents were recommended (*73*). A study for Georgia DOT indicated that the design VMA of 12.5 mm NMAS Superpave mixes was approximately 2 percent less than Marshall designed mixes with corresponding aggregate sources (*74*). Illinois DOT developed the locking point concept to prevent the over compaction of and subsequent aggregate degradation in the SGC. The locking point was believed to be related to the maximum achievable density during construction (*75*).

The literature indicates that there is still concern that the Ndesign levels have not been optimized to maximize field performance. The original Ndesign table was based on a limited data set for which the as-constructed densities were not available. The Ndesign table was consolidated based on a laboratory study design to evaluate the sensitivity of volumetric properties to Ndesign. There is a need to verify the current Ndesign values and relate them to field densification and performance.

CHAPTER 3 RESEARCH TEST PLAN

3.1 RESEARCH TEST PLAN

In 1999, the Ndesign table was revised and consolidated from 28 to 4 levels. However, this consolidation was based on the sensitivity of both volumetric properties and a performance test related to rutting to Ndesign; it was not tied to field performance. There is still concern that the Ndesign levels, in some cases, may be too high. Two states have adopted a single gyration level to design mixes; one of these has been successfully used for more than four years. Therefore there is a need to validate the Ndesign levels with respect to field performance.

In order to validate the Ndesign levels, an extensive field study was conducted to relate Ndesign to the in-place densification of pavements under various traffic loadings while monitoring field performance. The approach selected for this study was similar to the approach used by Brown and Mallick (*63*). Experimental variables for the project included: Ndesign level, lift thickness relative to NMAS, gradation and PG binder grade. The original experimental plan is shown in Table 3.1. Forty projects were required to fill the experimental plan. The projects were geographically distributed across the United States as shown in Figure 3.1. Attempts were made to identify projects in the southwestern and northeastern United States. Projects in the southwest were typically overlaid with open-graded friction course and therefore not suitable for the study. Projects could not be identified in the northeast that could be sampled during the

TABLE 3.1 Test Plan for Field Densification Study

Gyration Level	Fine or Coarse Graded	Lift thickness / nominal maximum aggregate size								
		2			3			4		
		High Temperature Performance Grade								
		Normal	+1	+2	Normal	+1	+2	Normal	+1	+2
50	F	X			X			X		
	C	X			X			X		
75	F	X			X			X		
	C	X			X			X		
100	F	X			X	X	X	X	X	X
	C	X			X	X	X	X	X	X
125	F	X			X	X	X	X	X	X
	C	X			X	X	X	X	X	X

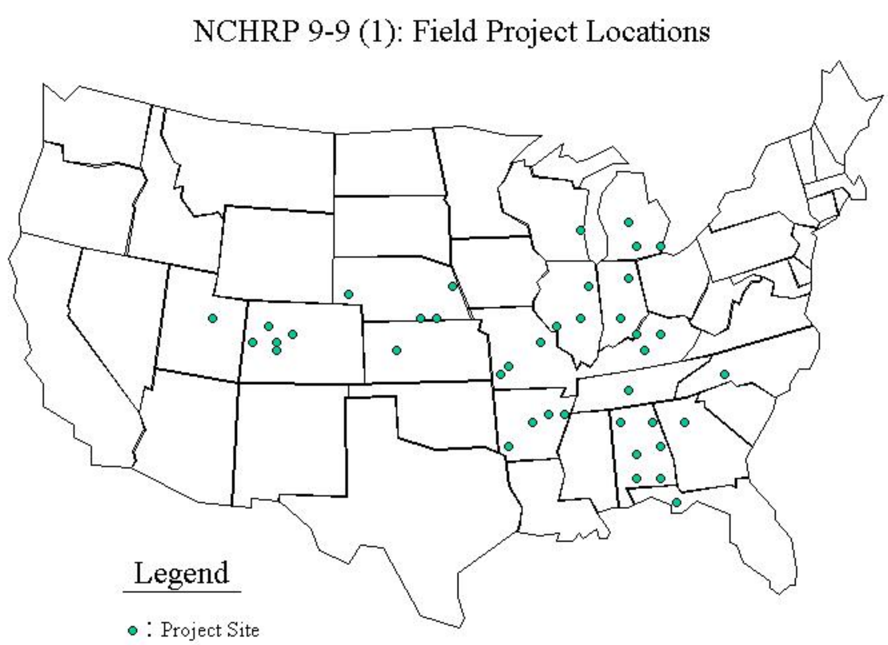

Figure 3.1. Location of Field Projects.

required timeframe. In 2000, twenty-two projects were visited and samples were obtained and tested. In 2001, the remaining eighteen projects were visited and samples obtained and tested. All of the mixes sampled were surface mixes.

For each project, the following testing and evaluation procedure was conducted:

1. Samples of loose mix were sampled from a truck at the asphalt plant; the corresponding location where the remainder of the mix was placed on the roadway was marked. Where possible, three samples were taken from each project, but in some cases only two could be obtained,
2. Three replicate pills (gyratory samples) were compacted to two different gyration levels, 100 and 160, without reheating, using two different SGCs in a mobile laboratory. This resulted in the compaction of 12 SGC pills per production sample, or 24 to 36 pills per project,
3. Samples were split and boxed for determination of maximum specific gravity (Gmm), asphalt content and gradation,
4. Three cores were taken from the right wheel path of the area marked on the roadway where the mix corresponding to a given sample was laid. This resulted in 6 (2 samples) to 9 (3 samples) cores per project at the time of construction,
5. Gyratory pills, cores, and loose mix for Gmm and asphalt content and gradation testing were brought back to NCAT for testing,
6. The following tests were run at the NCAT laboratory:
    a. Compacted sample specific gravity (Gmb) by ASHTO T166,
    b. Gmm by AASHTO T209,
    c. Asphalt content determination by AASHTO T164,

d. Washed gradation analysis by AASHTO T30,

7. The sites were revisited at approximately 3 months, 6 months, 1 year, and 2 years after construction. During each visit the following was conducted:
    a. Three additional cores were taken corresponding to each sample location at each project,
    b. The pavement condition was visually assessed.
    c. Rut depth measurements were taken adjacent to each core location with a 6-foot string line,
    d. The cores were shipped back to NCAT for specific gravity determination as described above.

Mix design and traffic information were also collected for each project. Brown and Mallick (*63*) indicated a difference between the compacted SGC sample density of reheated and laboratory prepared mix. Therefore, a mobile laboratory was mobilized to each site so that the SGC samples could be compacted without reheating. Previous research indicated differences in compaction between different brands and models of SGCs (*42, 49*). Therefore two SGCs, a Pine Model AFG1a and a Troxler Model 4141, were selected for the study. Although previous research had identified errors with the back calculation procedure (*69, 71*), it was deemed impossible to compact samples to all possible Ndesign levels. Two levels, 100 and 160 gyrations were selected to minimize the number of gyrations for which the sample density needed to be back calculated.

After two years, the project was extended to allow additional coring after four years. This was done to ensure that the pavements had reached their ultimate density. The same procedure as described in No. 7 above was used at the four-year interval. The collected traffic, in-place density, and SGC compacted sample density information was used to evaluate the relationship between Ndesign and field performance.

CHAPTER 4 TEST RESULTS AND ANALYSES

4.1 PROJECTS SELECTED

A summary of the projects selected for the study is shown in Table 4.1. The data in Table 4.1 are grouped by Ndesign level, corresponding to 50, 75, 100 and 125 gyrations. Within each category, the data are sorted by high temperature binder grade bumps and actual Ndesign level. The distribution of factors in Table 4.1 provides some interesting notes on the use of Superpave at the time the projects were sampled. Several states were still using the original Ndesign levels. These projects were grouped with the closest current Ndesign level. Only one project was identified with an Ndesign of 50 gyrations, 12 projects with Ndesign of 75 gyrations (68-86), 17 projects with Ndesign of 100 gyrations (90-109), 10 projects with Ndesign of 125 gyrations. Although only one project with an Ndesign of 50 gyrations was sampled, it will be shown later that the distribution of design traffic meets the intent of the experimental design.

Three different NMAS were sampled: 9.5 (11 projects), 12.5 (26 projects) and 19.0 mm (3 projects). The average lift thickness was determined from the average of the core thickness measurements at the time of construction. Fine- and coarse-graded mixes were separated by their percents passing the 2.36 mm sieve. Fine-graded mixes are defined as having the design percent passing the 2.36 mm sieve above (finer than) the maximum density line. Coarse-graded mixes are defined as having the design percent passing the 2.36 mm sieve below (coarser than) the maximum density line. Figure 4.1

TABLE 4.1 Summary of Project Information

Project ID	Roadway	NMAS	Avg. Thick., mm	Lift/ NMAS	Fine or Coarse Graded	Neat or Mod.	LTPP Grade		Grade Used		High Temp. Bump	Ndesign
							High	Low	High	Low		
KY-1	CR 1796	9.5	31.2	3	C	N	64	28	64	22	0	50
NE-1	Hwy 8	12.5	39.8	3	F	N	64	28	64	22	0	68
KY-3	CR 1779	9.5	27.1	3	F	N	64	28	64	22	0	75
MI-2	Hwy 50	9.5	39.9	4	F	N	58	28	58	28	0	75
MI-3	Hwy 52	9.5	32.4	3	F	N	58	28	58	28	0	75
UT-1	Hwy 150	12.5	38.7	3	F	M	64	22	64	34	0	75
NE-3	Hwy 8	12.5	51.2	4	F	N	64	28	64	22	0	76
CO-2	Hwy 82	12.5	53.3	4	F	M	64	28	64	28	0	86
CO-5	Hwy 82	12.5	44.3	4	F	M	64	28	64	28	0	86
AL-5	Hwy 167	12.5	33.7	3	C	N	64	16	67	22	0.5	75
FL-1	Davis Hwy	9.5	34.3	4	C	N	64	10	67	22	0.5	86
CO-1	Hwy 9	19	49.6	3	F	N	52	34	58	28	1	68
CO-4	Hwy 13	12.5	47.6	4	F	N	58	34	64	28	1	86
NE-2	Hwy 77	19	48.7	3	F	N	64	28	64	22	0	96
MO-2	Hwy 65	12.5	78.8	6	C	N	64	22	64	22	0	100
AL-6	Andrews Rd	19	33.0	2	F	N	64	16	67	22	0.5	95
AL-2	Hwy 168	12.5	43.1	3	C	N	64	22	67	22	0.5	100
AL-4	Hwy 84	12.5	54.1	4	C	N	64	16	67	22	0.5	100
AL-1	Hwy 157	12.5	43.2	3	C	N	64	16	67	22	0.5	106
IL-1	I-57	9.5	40.5	4	C	M	64	28	70	22	1	90
IL-2	I-64	9.5	44.5	5	C	M	64	22	70	22	1	90
IN-1	Hwy 136	12.5	44.1	4	C	N	58	28	64	22	1	100
KS-1	I-70	9.5	22.3	2	F	M	64	28	70	28	1	100
TN-1	Hwy 171	12.5	34.8	3	F	M	64	22	70	22	1	100
IL-3	I-70	9.5	45.7	5	C	M	64	28	70	22	1	105
NE-4	I-80	12.5	55.2	4	F	M	64	28	70	28	1	109
AL-3	Hwy 80	12.5	38.0	3	C	M	64	10	76	22	2	100
GA-1	Hwy 13	12.5	44.1	4	F	M	64	16	76	22	2	100
KY-2	I-64	9.5	33.9	4	C	M	64	28	76	22	2	100
WI-1	I-94	12.5	36.3	3	C	M	58	28	70	28	2	100
CO-3	I-70	12.5	50.6	4	C	M	64	22	76	28	2	109
IN-2	I-69	12.5	37.1	3	C	N	58	28	64	22	1	125
MI-1	I-75	9.5	35.6	4	C	N	58	28	64	22	1	125
MO-1	I-70	12.5	51.1	4	C	M	64	22	70	22	1	125
MO-3	I-44	12.5	48.4	4	C	M	64	22	70	22	1	125
AR-1	I-40	12.5	53.5	4	C	M	64	16	76	22	2	125
AR-2	I-55	12.5	51.0	4	C	M	64	16	76	22	2	125
AR-3	I-40	12.5	52.8	4	C	M	64	16	76	22	2	125
AR-4	I-30	12.5	56.8	5	C	M	64	16	76	22	2	125
NC-1	I-85	12.5	45.8	4	F	M	64	16	76	22	2	125

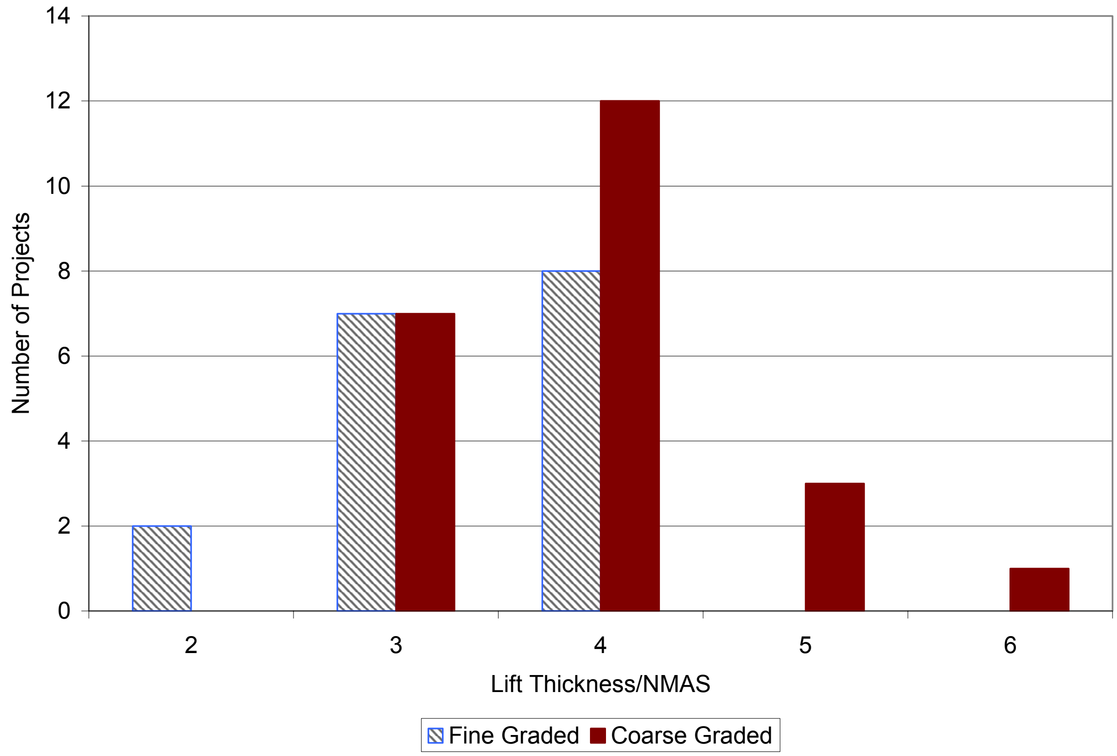

Figure 4.1. Frequency Distribution of Lift Thickness to NMAS by Gradation.

illustrates the distribution of lift thickness to NMAS ratio for the fine- and coarse-graded mixes. From Figure 4.1 it can be seen that there is a trend for thicker lift thicknesses for coarse graded mixes. Although the distribution of lift thickness to NMAS ratio does not exactly match the experimental design, it does indicate a representative distribution of field practice. The Ndesign of 75 gyration projects were predominantly fine-graded. Two-thirds of the Ndesign of 100 gyration projects were coarse-graded and all but one of the Ndesign of 125 gyration projects were coarse-graded. Therefore from this data set, it appears that higher gyration mixes are more likely to be coarse graded.

The climatic binder grade for each project was determined using LTPPBind Version 2.1 (*77*). The high temperature grade bumps were determined by comparing the climatic binder grade with that used on the project. As expected, high temperature binder bumps were predominantly found with higher Ndesign levels. Only two projects were identified with Ndesign of 100 gyrations that did not include a binder bump and all of the Ndesign of 125 gyration projects included at least one high temperature binder bump. Therefore, for design traffic levels greater than 3 million ESALs, the majority of state agencies included in this data set are using high temperature binder grades that are stiffer than the recommended climatic grade based on the LTPP weather station data. Binder bumps are recommended for slow moving traffic (less than 70 km/hr [44 mph]) and for 20-year design traffic volumes greater than 30 million ESALs (*4*).

4.2 TEST RESULTS

There are several important hypotheses for this project:

1. Pavement densification is related to traffic,
2. The laboratory design density should match the ultimate density in the field,

Therefore,

3. The laboratory compaction effort should be related to traffic.

Data from the 2000 NCAT Test Track (*78*) supports other hypotheses:

4. Binder grade, particularly modified binders, effects the rate of densification,
5. Densification (the majority of the "rutting" which occurred at the 2000 NCAT Test Track) only occurred when the air temperature exceeded 28 °C.

To address the hypotheses, test results are provided as they relate to the following:

1. Evaluation of the validity of the data,
2. Estimation of traffic at various sampling intervals,
3. Evaluation of densification under traffic,

4. Verification of Ndesign,
5. Evaluation of the locking point concept.

4.2.1 Comparison of Mixture Data to Design Job Mix Formula

Table 4.2 presents the job mix formula (JMF) gradation and asphalt content for each of the 40 projects. No JMF was available for project MI-1, constructed as a warranty project. Three solvent extractions were performed for each sample taken at each project according to AASHTO T164, resulting in 6 to 9 extractions per project depending on whether 2 or 3 samples were taken. Washed gradations were performed on the recovered aggregate according to AASHTO T30. The results from the six to nine extractions, representing two or three samples, respectively, were averaged for comparison with the JMF. Figure 4.2 shows the design versus average field gradations for the percent passing the 2.36 mm sieve. The 2.36 mm sieve is one of the control sieves for Superpave mixes. Lines have been added to the figure representing ± 4.5 percent from the job mix formula, chosen to represent typical allowed variability for the average of three samples. Four projects, KY-2, MI-2, NE-2 and UT-1, exceeded the ±4.5 percent tolerance on the 2.36 mm sieve. Figure 4.3 shows the design versus average field gradations for the percent passing the 0.075 mm sieve. Lines have been added to the figure representing ± 1.1 percent of the job mix formula, a typical tolerance for three samples for the percent passing the 0.075 mm sieve. The average percent passing the 0.075 mm sieve for fourteen projects exceeded the 1.1 percent tolerance. Four projects exceeded the tolerance by a large amount, CO-5, MO-2, and UT-1. Generally, dust content is expected to increase during production. However, only six of the fourteen

TABLE 4.2 Design Gradation and Optimum Asphalt Content (JMF)

Project ID	Percent Passing										Design AC%
	19	12.5	9.5	4.75	2.36	1.18	0.6	0.3	0.15	0.075	
AL-1	100	96	79	45	32	25	19	11	6	3.4	4.9
AL-2	100	99	86	47	30	20	15	9	5	3.4	5.3
AL-3	100	90	75	47	34	22	14	7	4	3.0	5
AL-4	100	93	78	47	34	25	19	12	6	4.3	3.65
AL-5	100	99	87	57	36	25	18	12	7	4.2	5
AL-6	99	87	78	66	49	38	25	14	7	4.6	5.25
AR-1	100	96	78	45	31	21	15	11	7	4.8	5.1
AR-2	100	93	83	40	29	22	16	13	9	5.4	4.9
AR-3	100	94	83	46	30	20	15	12	8	5.6	5.5
AR-4	100	95	84	55	37	25	18	11	7	4.6	5.5
CO-1	99	89	78	59	44	31	22	15	11	7.4	6.1
CO-2	100	96	85	60	45	34	24	17	11	7.6	5.5
CO-3	100	94	81	57	35	24	17	13	9	6.4	5.6
CO-4	100	100	89	56	36	27	20	NA	NA	6.5	5.3
CO-5	100	96	85	60	45	34	24	17	11	7.6	5.5
FL-1	100	100	97	65	40	29	23	14	9	5.3	5.7
GA-1	100	98	85	NA	38	NA	NA	NA	NA	5.0	4.8
IL-1	100	100	99	59	32	22	16	9	5	4.3	5.5
IL-2	100	98	90	57	34	22	14	9	7	5.5	5.5
IL-3	100	100	98	57	36	23	14	9	6	4.9	5.33
IN-1	100	100	91	59	39	NA	15	NA	NA	6.0	6.4
IN-2	100	100	95	58	43	NA	20	NA	NA	3.9	5.6
KS-1	100	100	90	54	38	25	17	11	7	5.0	5.7
KY-1	100	100	95	69	41	27	19	10	NA	5.0	5.8
KY-2	100	100	98	67	39	25	18	11	NA	4.5	5.8
KY-3	100	100	94	69	46	31	21	8	5	4.5	5.6
MI-1											
MI-2	100	100	100	83	63	40	28	19	10	5.7	6.8
MI-3	100	100	100	80	55	41	31	19	10	5.0	6.2
MO-1	100	97	85	49	29	17	10	6	4	3.1	5.5
MO-2	100	98	83	48	31	18	13	10	8	6.7	6
MO-3	100	98	89	52	28	18	12	9	7	5.7	6
NC-1	100	95	89	58	43	33	23	14	9	5.4	5.1
NE-1	100	95	90	78	49	30	23	12	NA	3.6	5.5
NE-2	99	90	81	62	41	27	19	11	6	3.4	5
NE-3	100	90	81	71	50	32	25	12	NA	3.5	5.3
NE-4	100	91	87	73	51	34	23	14	NA	6.1	4.8
TN-1	100	98	86	58	43	32	22	10	5	4.0	5.1
UT-1	100	100	89	70	62	45	31	15	NA	6.8	5.4
WI-1	100	98	90	62	39	26	17	9	5	3.5	5.1

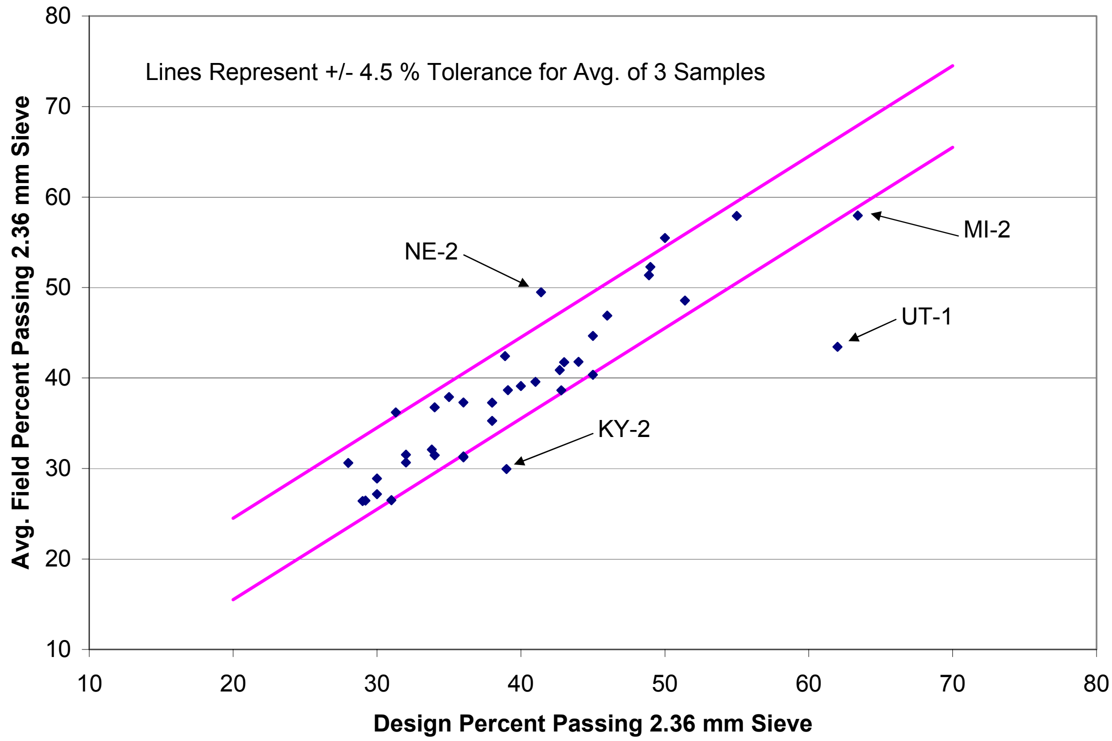

Figure 4.2. Design versus Average Field Percent Passing the 2.36 mm Sieve.

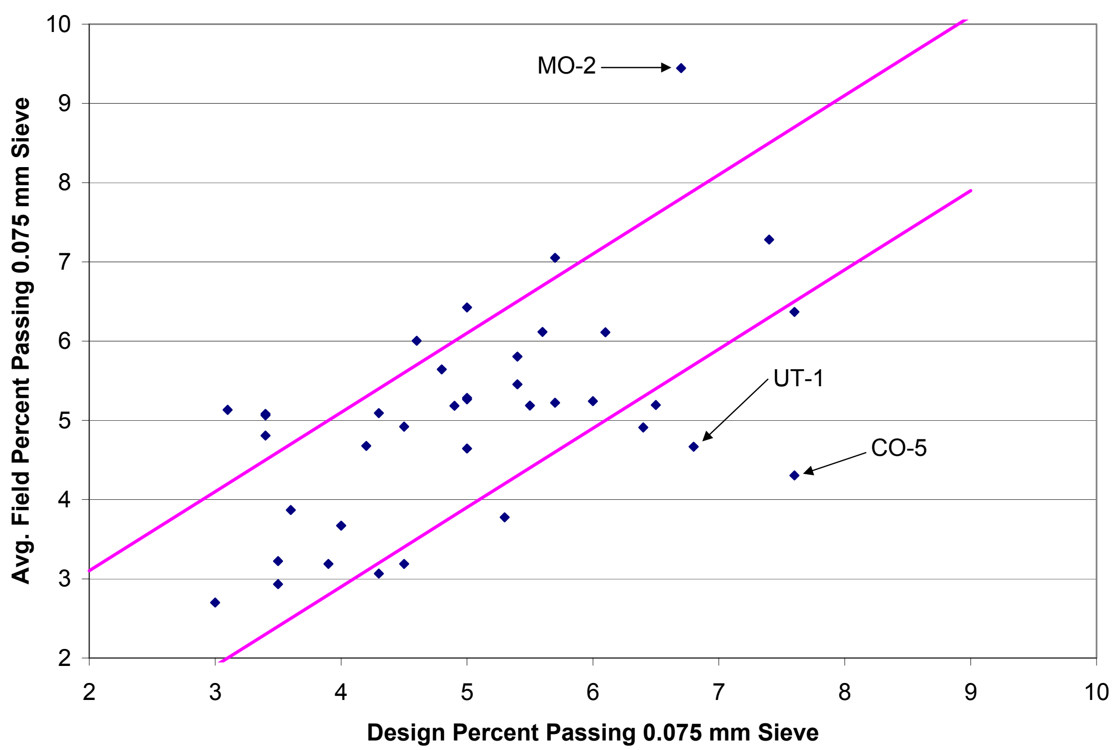

Figure 4.3. Design versus Average Field Percent Passing the 0.075 mm Sieve.

projects exceeding the 1.1 percent tolerance exceeded it on the high side. Figure 4.4 shows the design versus the average recovered asphalt contents for the field samples. Lines were added to the figure representing ± 0.33 percent asphalt from the job mix formula, a typical tolerance for the average of three samples. With one exception, the fourteen projects that fell outside of this range were all on the low side. Solvent extractions were performed, which may produce lower asphalt contents (incomplete recovery) compared to the ignition furnace that many agencies now use. Liquid asphalt is also the most expensive component in hot mix asphalt; contractors may tend to put in the least amount allowable by the specifications.

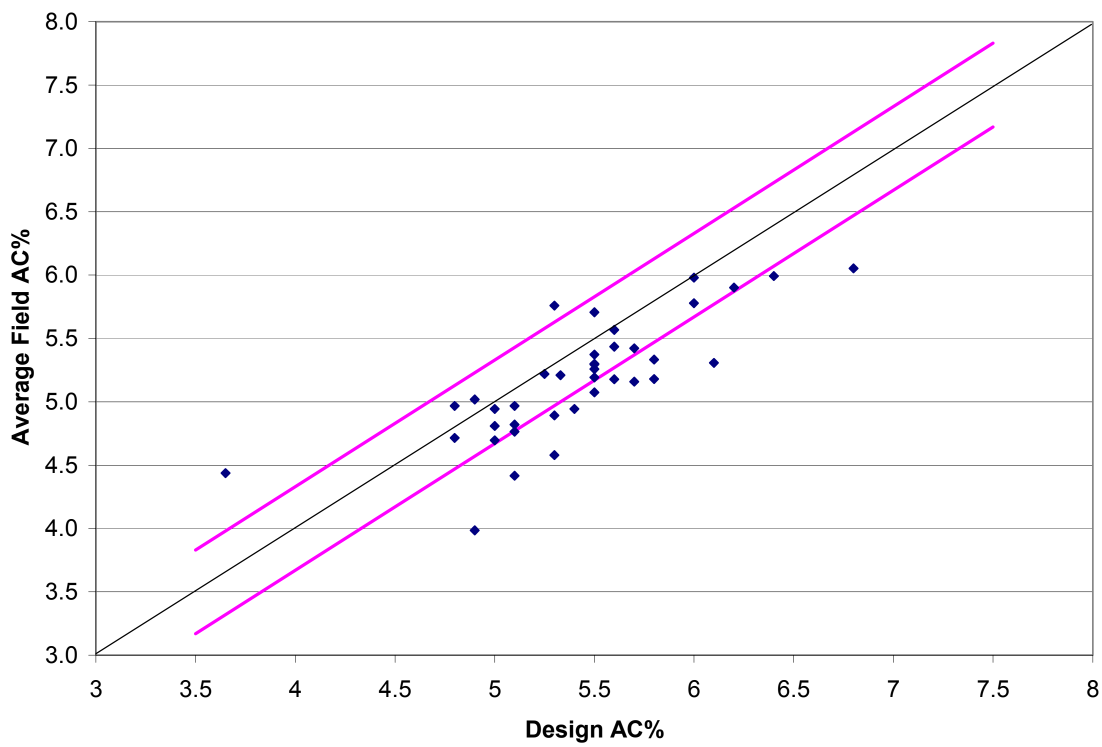

Figure 4.4. Design versus Average Field Asphalt Content.

4.2.2 Estimation of Traffic

Initially, traffic at the various sampling intervals was estimated by dividing the design ESALs reported by the agency by the design period and then multiplying by the elapsed time since construction. This method can produce varying degrees of error early in the life of the pavement depending on the growth rate used for the traffic. Traffic data were updated to reflect the actual traffic levels during the monitoring period. To obtain the best possible traffic estimates, the following procedure was used:

1. Determine average annual daily traffic (AADT) for the year the section was constructed.
2. Determine a growth rate. In some cases the growth rate was provided by the agency. In other cases it was fit from historical AADT data using Equation 7. The growth rate was fit using a least squares approach and Microsoft Excel's Solver routine.

$$AADT_N = AADT_C * (1+i)^N \tag{7}$$

where,

$AADT_N$ = Predicted AADT after N years,

N = number of years between when the project was constructed and the year of interest,

$AADT_C$ = AADT in the year the pavement was constructed (or repaved),

i = growth rate.

3. Determine the percent trucks. Some agencies measure a combined percentage of all trucks. Other agencies track separate percentages for single units (such as cube trucks) and multiple units (such as tractor trailers). Percent trucks or heavy

commercial vehicles were recorded as either a single percentage or as percent single units and multiple units. Multiple units generally represent vehicles with predominantly tandem axles except for the steer axle.

4. Determine a truck factor(s) to convert heavy vehicles to ESALs. In some cases agencies used a standard factor for either all trucks or separate factors for single and multiple units. In other cases agencies recorded the AASHTO vehicle classification or single and tandem axles load spectra. In these cases, a truck factor was calculated by multiplying the percentage of total repetitions in a load group by the corresponding equivalent axle load factor for that load group to determine a composite single unit factor and multiple unit factor.
5. Determine directional distribution and lane distribution factors. Directional distribution was generally assumed to be 0.5 unless AADT values were for a single direction or the agency recommended a specific value. Agency recommendations were used for the lane distribution factor. If none were provided, the recommendations provided in the AASHTO Design Guide (*79*) were used.

TABLE 4.3 Lane Distribution Factors

Number of Lanes in each Direction	Percent of 18-kip ESALs in Design Lane
1	100
2	80-100
3	60-80
4	50-75

6. The accumulated ESALs at each sampling period, as well as the ESALs for the specified design period are calculated according to Equation 8 or 9.

$$ESAL = (AADT_C + AADT_C \times (1+i)^N)/2 \times T\% \times TF \times D \times L \times 365 \times N \qquad (8)$$

$$ESAL = (AADT_C + AADT_C \times (1+i)^N)/2 \times \left(ST\% \times SF + MT\% \times MF + \left(100 - ST\% - MT\%\right) \times CF\right)$$
$$\times D \times L \times 365 \times N \tag{9}$$

where:

$AADT_C$ = AADT in the year the pavement was constructed (or repaved),

i = growth rate,

N = number of years (or fraction) between construction and sampling time,

T% = percent trucks,

TF = truck factor to convert trucks to ESALs

D = directional distribution factor,

L = lane distribution factor,

ST% = percent single unit trucks,

SF = single unit truck factor to convert to ESALs,

MT% = percent multiple unit trucks,

MF = multiple unit truck factor to convert to ESALs, and

CF = car factor to convert to ESALs

Table 4.4 summarizes the factors used to calculate the traffic at various sampling periods. Using the data in Table 4.4, the design traffic at the design interval specified by the agency and the accumulated traffic at each coring interval were calculated. The accumulated traffic at each coring interval was calculated using the actual dates that the coring occurred and not the targeted intervals, e.g. three months, six months, one year, two years and four years. The accumulated or design traffic for each of these intervals is shown in Table 4.5.

TABLE 4.4 Factors used to Calculate Accumulated ESALs at Various Intervals

Project ID	Roadway	Number of Lanes Both Directions	AADT	Growth Rate	% Trucks	% Single Units	% Combo Units	Directional Distribution Factor	Lane Distribution Factor	Combined ESAL Factor	Single Unit ESAL Factor	Combo Unit ESAL Factor	Car ESAL Factor	Design Period (Yrs)
AL-1	Hwy 157	4	7450	2.5%	20.0%			0.5	0.95	0.99				20
AL-2	Hwy 168	2	7077	2.5%	10.7%			0.5	1	0.99				20
AL-3	Hwy 80	4	10870	2.5%	19.0%			0.5	0.9	0.99				20
AL-4	Hwy 84	2	7120	2.8%	14.0%			0.5	1	0.99				20
AL-5	Hwy 167	2	3796	2.5%	10.0%			0.5	1	0.99				20
AL-6	Andrews Rd	2	1066	3.5%	2.5%			0.5	1	0.99				20
AR-1	I-40	4	31000	2.4%	27.6%	14%	5.3%	0.5	0.9		1.163	3.77	0.0002	20
AR-2	I-55	4	32000	4.7%	33.7%	19.3%	7.2%	0.5	0.9		1.163	3.77	0.0002	20
AR-3	I-40	4	33000	5.9%	51.8%	29.7%	11.0%	0.5	0.9		1.163	3.77	0.0002	20
AR-4	I-30	4	22750	5.1%	47.8%	27.4%	10.2%	0.5	0.9		1.163	3.77	0.0002	20
CO-1	Hwy 9	4	22193	1.9%		4.3%	0.5%	0.5	0.9		0.249	1.087	0.003	10
CO-2	Hwy 82	4	15893	2.0%		4.4%	2.0%	0.5	0.9		0.249	1.087	0.003	10
CO-3	I-70 Bus.	6	12581	1.5%		2.6%	0.8%	1	0.6		0.249	1.087	0.003	10
CO-4	Hwy 13	2	2279	1.8%		15.3%	10.8%	0.5	1		0.249	1.087	0.003	10
CO-5	Hwy 82	4	15893	2.0%		4.4%	2.0%	0.5	0.9		0.249	1.087	0.003	10
FL-1	Davis Hwy	5	37100	3.0%	2.0%			0.5	0.24	0.89				20
GA-1	Buford Hwy	4	13924	1.6%	8.3%			1	0.9	0.97				20
IL-1	I-57	4	17700	3.0%		2.3%	23.7%	0.5	0.9		0.36	1.32	0.0004	20
IL-2	I-64	4	23100	3.0%		4.8%	31.6%	0.5	0.9		0.36	1.32	0.0004	20
IL-3	I-70	4	19900	3.0%		9.1%	34.2%	0.5	0.9		0.36	1.32	0.0004	20
IN-1	US 136	2	14080	2.5%	2.1%			0.5	1	1.30				20
IN-2	I-69	4	30250	2.3%	27.3%			0.5	0.9	1.30				20
KS-1	I-70	4	5461	3.5%	27.8%			1	0.88	0.69				20
KY-1	CR1796	2	211	5.2%	7.9%			0.5	1	0.47				20
KY-2	I-64	4	14500	2.1%	18.7%			1	0.467	1.07				20
KY-3	CR1779	2	262	4.8%	7.7%			1	0.5	0.64				20

TABLE 4.4 Factors used to Calculate Accumulated ESALs at Various Intervals (Continued)

Project ID	Roadway	Number of Lanes Both Directions	AADT	Growth Rate	% Trucks	% Single Units	% Combo Units	Directional Distribution Factor	Lane Distribution Factor	Combined ESAL Factor	Single Unit ESAL Factor	Combo Unit ESAL Factor	Car ESAL Factor	Design Period (Yrs)
MI-1	I-75	8	60500	2.2%	5.0%			1	0.8	0.72				20
MI-2	Hwy 50	2	5500	1.5%	8.7%			0.5	1	0.61				20
MI-3	Hwy 52	2	7900	1.5%	7.6%			0.5	1	0.59				20
MO-1	I-70	4	18500	1.9%	34.9%			0.5	0.95	1.00				20
MO-2	Hwy 65	4	19400	5.3%	9.8%			0.5	0.95	1.00				20
MO-3	I-44	4	32750	2.5%	35.8%			0.5	0.95	1.00				20
NC-1	I-85	4	61346	2.6%		11.0%	24.0%	0.5	0.8		0.30	1.15	0.00	20
NE-1	Hwy 8	2	700	1.5%		7.7%	12.3%	0.5	1		0.25	0.89	0.00	20
NE-2	Hwy 77	2	2623	1.4%		4.9%	13.1%	0.5	1		0.23	0.91	0.00	20
NE-3	Hwy 8	2	1320	0.7%		4.2%	6.8%	0.5	1		0.25	0.89	0.00	20
NE-4	I-80	4	7506	3.6%		6.2%	52.8%	0.5	0.9		0.14	1.01	0.00	20
TN-1	Hwy 171	2	8800	4.87%		7.7%	2.3%	0.5	1		0.44	1.08	0.002	20
UT-1	Hwy 150	2	1013	3.0%	14.9%	9.5%	4.1%	1	1	0.55	0.36	0.56	0.0201	20
WI-1	I-94	6	81428	2.0%	6.8%			1	0.4	0.72				20

TABLE 4.5 Accumulated ESALs at Sampling Intervals

Project ID	Roadway	3 Months	6 Months	12 Months	24 Months	48 Months	20 Year Design ESALs
AL-1	Hwy 157	69,600	129,022	263,972	559,853	1,149,977	6,748,142
AL-2	Hwy 168	34,215	69,022	138,140	296,338	611,855	3,610,001
AL-3	Hwy 80	97,881	170,357	346,635	767,236		8,861,352
AL-4	Hwy 84	58,977	101,426	182,573	402,633		4,899,406
AL-5	Hwy 167	18,854	34,981	65,784	149,147		1,809,675
AL-6	Andrews Rd.	1,939	2,960	5,323	9,916	19,907	143,958
AR-1	I-40	690,394	1,131,450	2,110,407	4,619,146	8,120,222	48,726,562
AR-2	I-55	942,469	1,562,429	2,957,818	6,590,986	11,850,476	91,370,805
AR-3	I-40	956,294	1,936,956	4,141,677	9,974,122	18,576,489	170,842,507
AR-4	I-30	578,939	1,201,114	2,596,098	6,261,493	11,603,641	97,890,077
CO-1	Hwy 9	20,866	38,064	68,695	138,927	287,854	756,789
CO-2	Hwy 82	27,654	76,585	91,905	185,961	385,731	1,017,593
CO-3	I-70 Bus.	14,675	26,863	48,805	98,324	202,528	523,624
CO-4	Hwy 13	19,805	36,273	65,592	132,764	274,968	720,911
CO-5	Hwy 82	26,897	75,056	90,370	184,395	384,096	1,017,593
FL-1	Davis Hwy	8,117	16,784	30,420	62,813		811,658
GA-1	Buford Hwy	133,892	287,006	435,998	798,627	1,568,426	8,803,521
IL-1	I-57	252,510	449,723	948,145	1,963,241	3,970,500	26,285,917
IL-2	I-64	445,196	792,900	1,671,661	3,461,359	7,000,327	46,344,297
IL-3	I-70	365,925	699,160	1,541,346	3,256,535	6,648,086	44,466,336
IN-1	US 136	28,199	41,039	73,589	144,256	372,269	1,850,992
IN-2	I-69	688,995	957,471	1,827,656	3,586,718	9,265,105	45,150,555
KS-1	I-70	85,315	227,911	374,505	729,765	1,435,783	10,075,962
KY-1	CR1796	530	819	1,591	3,038	6,357	53,706
KY-2	I-64	181,101	278,340	539,117	1,016,831	2,061,494	12,438,605
KY-3	CR1779	857	1,334	2,608	4,988	10,412	84,028
MI-1	I-75	211,625	419,507	650,039	1,426,667	2,893,187	15,966,398
MI-2	Hwy 50	24,456	32,399	54,261	119,143	240,447	1,250,146
MI-3	Hwy 52	26,258	45,341	0	132,171	278,594	1,515,200
MO-1	I-70	493,003	884,139	1,306,076	2,541,928	4,778,697	27,546,007
MO-2	Hwy 65	107,389	224,065	349,533	734,786	1,462,700	12,517,675
MO-3	I-44	597,842	1,307,458	2,063,169	4,337,141	8,453,012	53,683,941
NC-1	I-85	692,210	1,427,287	2,889,164	6,040,907	12,565,156	73,918,507
NE-1	Hwy 8	4,441	10,481	16,872	37,057	67,176	383,385
NE-2	Hwy 77	16,728	39,363	63,672	140,411	255,199	1,450,960
NE-3	Hwy 8	4,183	10,424	17,010	37,683	68,179	365,719
NE-4	I-80	166,950	413,599	671,010	1,529,367	2,841,721	20,084,248
TN-1	Hwy 171	25,738	58,918	98,776	207,136	428,119	3,490,393
UT-1	Hwy 150	8,014	14,873	27,347	55,992	122,456	771,982
WI-1	I-94	345,088	494,711	597,614	1,316,468	2,557,478	14,614,748

Figure 4.5 shows a distribution of the 20-year design traffic for the projects sampled. Although the original experimental matrix was not evenly filled due to the availability of projects, Figure 4.5 indicates a good distribution of 20-year design traffic. There are only three projects with less than 300,000 ESALs, however, it is expected that there is not a strong relationship between traffic and pavement densification at such low traffic levels. There are 21 projects with design traffic between 3 and 30 million ESALs. Under the current AASHTO M 323, all projects with a design traffic level between 3 and 30 million ESALs would be designed with an Ndesign of 100 gyrations (*4*). The maximum 20-year design traffic in the SHRP Ndesign experiment was 32.1 million ESALs (*1*). Nine projects in this study had 20-year design traffic in excess of 30 million ESALs.

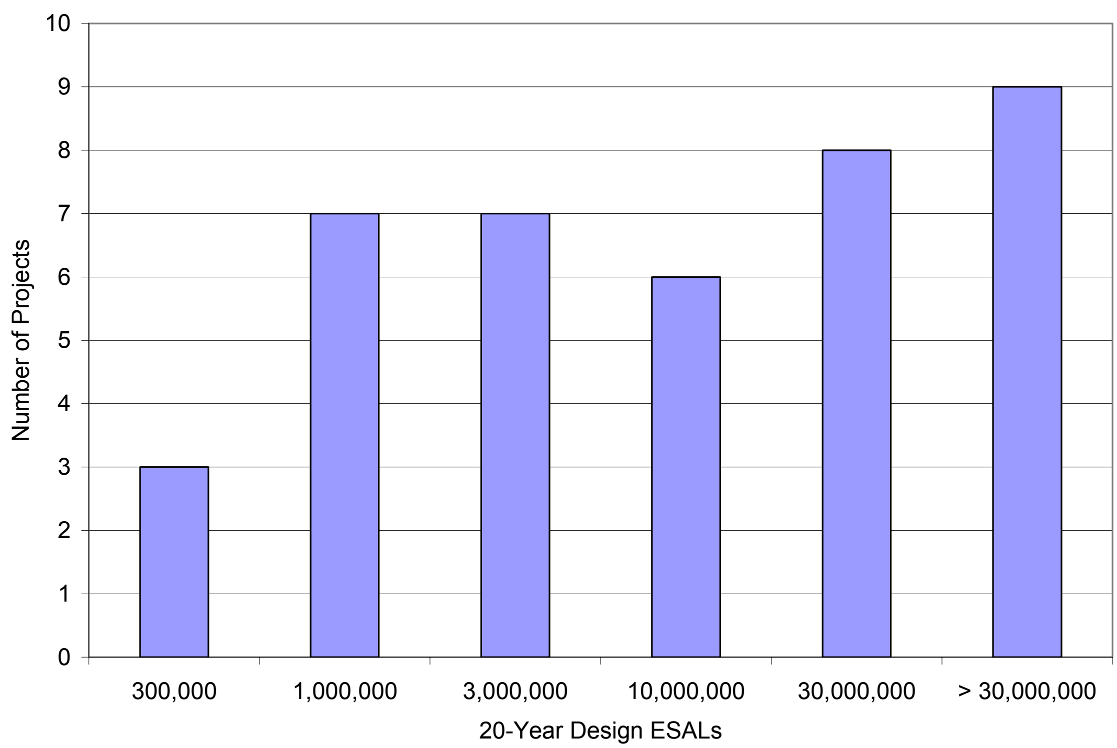

Figure 4.5. Distribution of 20-Year Design Traffic.

4.2.3 Pavement Densification

The in-place density of HMA may be the single factor that most affects the performance of a properly designed mixture (*30, 80*). A mediocre mix, well constructed with good in-place air voids, will often perform better than a good mix that has been poorly constructed (*30*). In-place density, between 92 and 97 percent of Gmm for surface mixes passing through or above the Superpave defined restricted zone will generally provide good performance (*80*). To limit permeability concerns, in-place density greater than 93 to 95 percent of Gmm may be required for larger nominal maximum aggregate size mixtures, stone mastic asphalt or coarse graded Superpave mixtures (*81*). In-place air voids that are too high may result in permeability to water and excessive binder oxidization, resulting in moisture damage, cracking or raveling (*80, 82, 83*). In-place density in excess of 97 percent of Gmm may result in permanent deformation or loss of skid resistance (*84*). Table 4.6 summarizes the average in-place densities for the projects at each of the sampling intervals through 2-years; the complete data are presented in the Appendix Table A.41 through A.80.

The average in-place as-constructed density for the 40 projects was 91.6 percent. Figure 4.6 shows a cumulative frequency distribution of the average in-place density for the 40 projects at the time of construction. From Figure 4.6, it is evident that 55 percent of the projects had in-place densities less than 92 percent of Gmm and 78 percent of the projects had in-place densities less than 93 percent of Gmm. This indicates that the in-place densities of the majority of the projects were less than desired. There may be a number of reasons for the as-constructed in-place densities being less than desired,

TABLE 4.6 Average In-Place Densities for Field Projects

Project ID	Roadway	Average In-Place Density, Percent Gmm				
		Construction	3 month	6 month	1 Year	2 Year
AL-1	Hwy 157	88.7	93.2	93.6	93.0	93.9
AL-2	Hwy 168	88.3	90.3	90.2	90.2	91.8
AL-3	Hwy 80	89.7	92.8	93.2	93.3	93.6
AL-4	Hwy 84	88.4	92.8	93.1	92.6	94.3
AL-5	Hwy 167	89.7	93.6	93.8	93.1	94.6
AL-6	Andrews Rd	91.8	93.1	92.7	93.1	93.3
AR-1	I-40	92.0	93.1	93.5	94.1	94.2
AR-2	I-55	89.4	90.9	91.4	91.8	91.8
AR-3	I-40	91.5	94.6	94.8	94.8	94.7
AR-4	I-30	90.9	94.2	93.5	94.5	94.5
CO-1	Hwy 9	93.8	96.9	96.5	97.2	98.1
CO-2	Hwy 82	94.7	96.6	96.6	96.9	97.1
CO-3	I-70	93.5	94.6	96.0	95.6	95.7
CO-4	Hwy 13	93.7	93.3	92.8	94.2	94.2
CO-5	Hwy 82	91.6	93.6	93.7	94.2	93.8
FL-1	Davis Hwy	91.8	94.2	94.8	94.3	95.2
GA-1	Hwy 13	95.0	95.7	95.8	96.0	96.5
IL-1	I-57	91.0	93.9	93.8	94.2	94.4
IL-2	I-64	91.8	94.2	94.1	94.4	95.2
IL-3	I-70	92.2	94.3	93.9	94.4	94.5
IN-1	Hwy 136	91.3	90.3	90.3	62.3	93.5
IN-2	I-69	91.4	90.7	91.7	94.7	94.1
KS-1	I-70	89.9	91.2	92.1	93.6	93.6
KY-1	CR1796	85.5	87.3	86.7	87.7	88.5
KY-2	I-64	92.2	93.2	93.3	93.9	94.1
KY-3	CR1779	92.6	93.1	93.7	94.3	94.2
MI-1	I-75	91.3	92.1	92.8	93.4	94.8
MI-2	Hwy 50	93.1	95.2	96.1	96.8	96.8
MI-3	Hwy 52	93.0	93.7	94.5	NA[1]	96.5
MO-1	I-70	93.4	96.4	95.6	95.8	96.5
MO-2	Hwy 65	92.6	94.2	92.7	94.4	95.1
MO-3	I-44	93.5	94.4	94.3	95.3	95.6
NC-1	I-85	90.1	92.8	91.7	93.0	93.4
NE-1	Hwy 8	92.6	95.4	95.5	95.3	95.7
NE-2	Hwy 77	93.0	95.2	95.0	95.3	95.7
NE-3	Hwy 8	91.0	94.8	95.1	95.0	95.4
NE-4	I-80	92.2	94.9	95.2	96.7	97.2
TN-1	Hwy 171	91.1	93.1	93.1	94.1	94.3
UT-1	Hwy 150	91.9	93.5	93.2	NA[2]	93.7
WI-1	US 45	92.4	93.8	93.8	94.4	94.3

[1] 1-Year cores not taken

[2] Section overlaid with plant-mix seal coat, NCAT Research Engineer elected not to take 1-Year cores.

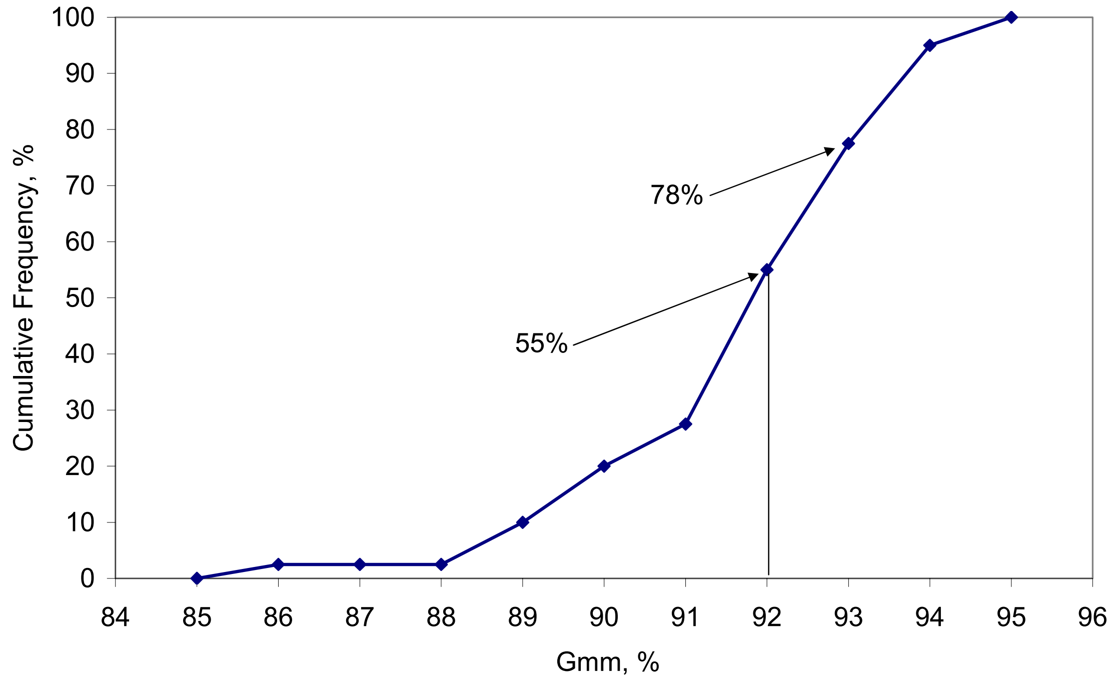

Figure 4.6. Cumulative Frequency Distribution of As-Constructed, In-place Density.

including:

1. State agency specifications,
2. The compactability of the mix,
3. The compaction effort or method of compaction used by the contractor, or
4. A combination of these factors.

An ANOVA was conducted using the General Linear Model (GLM) to examine factors which may have affected the as-constructed density. The two to three samples from each project were used as replicates, each sample represented by average of three cores. Agency, gradation (coarse or fine), high temperature PG, lift thickness to NMAS ratio, and 2000 Ndesign level were considered as factors. 2000 Ndesign level is the Ndesign rounded to the levels adopted in 2000 (50, 75, 100, and 125). The factor inputs are summarized in Table 4.1, presented previously. There were insufficient replicates to evaluate interactions, particularly considering the 16 levels for agency. Two factors were significant at the 95 percent confidence level: agency and Ndesign. The fitted means for the main effects indicated very low in-place density resulting from mixes with Ndesign of 50 gyrations. Only one project, KY-1, was designed at 50 gyrations. The average as-constructed density for KY-1 was 85.5 percent. There were no in-place density requirements in the specifications for KY-1. Therefore, this project was eliminated from the date set. The ANOVA was re-run resulting in agency being the only significant factor ($p = 0.000$). Examination of the main effects indicated that three agencies achieved particularly good as-constructed densities: Colorado, Missouri and Georgia. As noted previously, Colorado DOT uses 100 mm diameter SGC molds, which tends to result in lower sample densities and therefore higher asphalt contents which may aid in field compaction (*73*). All of the Colorado DOT projects used crushed gravel for the coarse aggregate, which may be easier to compact than crushed stone aggregate. Although many agencies have switched (or switched back) to density specifications based on cores sine the implementation of Superpave, Colorado DOT uses the nuclear gauge to determine in-place density. Gauges are calibrated to cores at the beginning of the project and density is monitored with additional cores throughout the project. Both the contractor and the agency conduct nuclear density tests. Georgia DOT will adjust the asphalt content of a mixture in the field to ensure in-place density requirements are met.

The main effects for lift thickness to NMAS ratio indicated some unexpected trends when agency was included as a factor. It was believed that this may have been due to interactions which could not be analyzed with the replicates available. Therefore, the

ANOVA was rerun as described previously without using agency as a factor. Only high temperature PG was significant (p = 0.000); however, this is driven by the PG 67-22 which was only used by two agencies, one of which consistently had low as-constructed densities. The fitted model is poor (R^2 = 0.37) without agency as a factor (R^2 = 0.67 with agency).

The main effects plot for the fitted means is shown in Figure 4.7. With the exception of the PG 67, the trends are as expected: increasing density with increasing lift thickness to NMAS, decreasing density with increasing Ndesign level, and increasing density with fine-graded as compared to coarse-graded mixes. As noted previously, coarse-graded mixes tend to require higher in-place density to be impermeable to water (*81*).

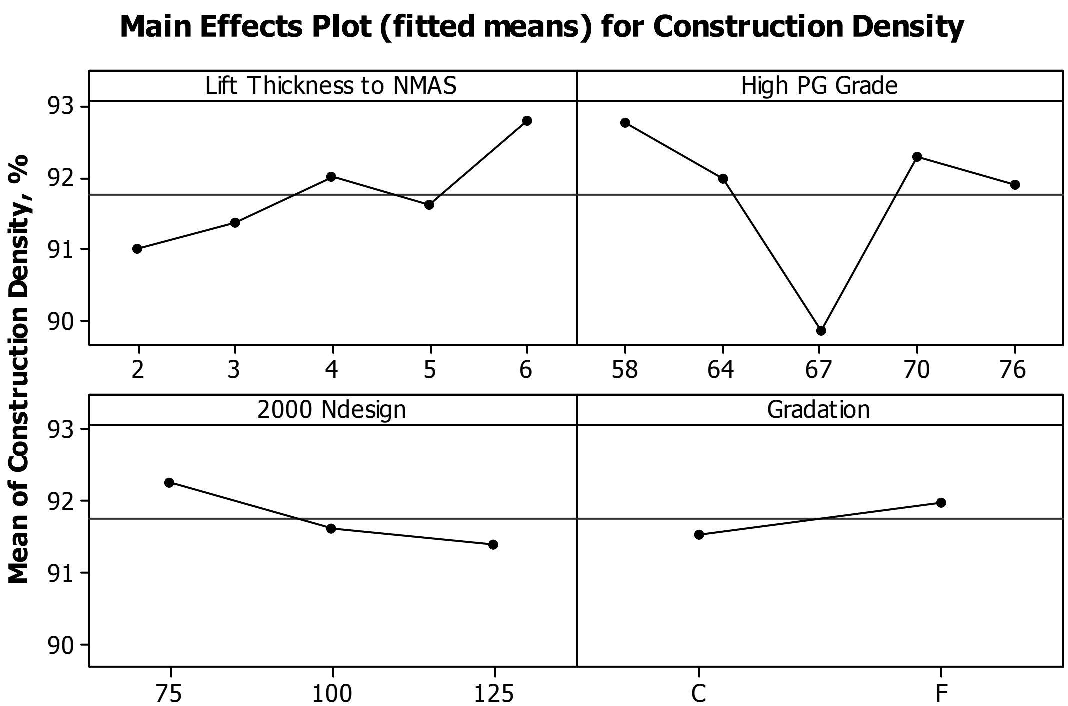

Figure 4.7. Main Effect Plot for Factors Affecting As-Constructed Density.

Figure 4.8 shows a cumulative frequency plot for in-place density for the sampling periods through 2-years. Individual plots, for each project, are shown in the Appendix. From Figure 4.8, it is apparent that the majority of the densification occurs in the first 3 months after construction (63 percent). There is little if any difference between the 3 and 6 month in-place densities. This is most likely due to the fact that projects constructed during the summer would be experiencing cooler weather between 3 and 6 months after construction. This matches the findings from the 2000 NCAT Test Track, which indicated that little densification occurred during the winter months (*78*). The in-place density representing the 50 percent frequency increased slightly from 93.0 to 93.2 percent between 6 months and 1 year, and then 1.4 percent to 94.6 percent between 1 and 2 years.

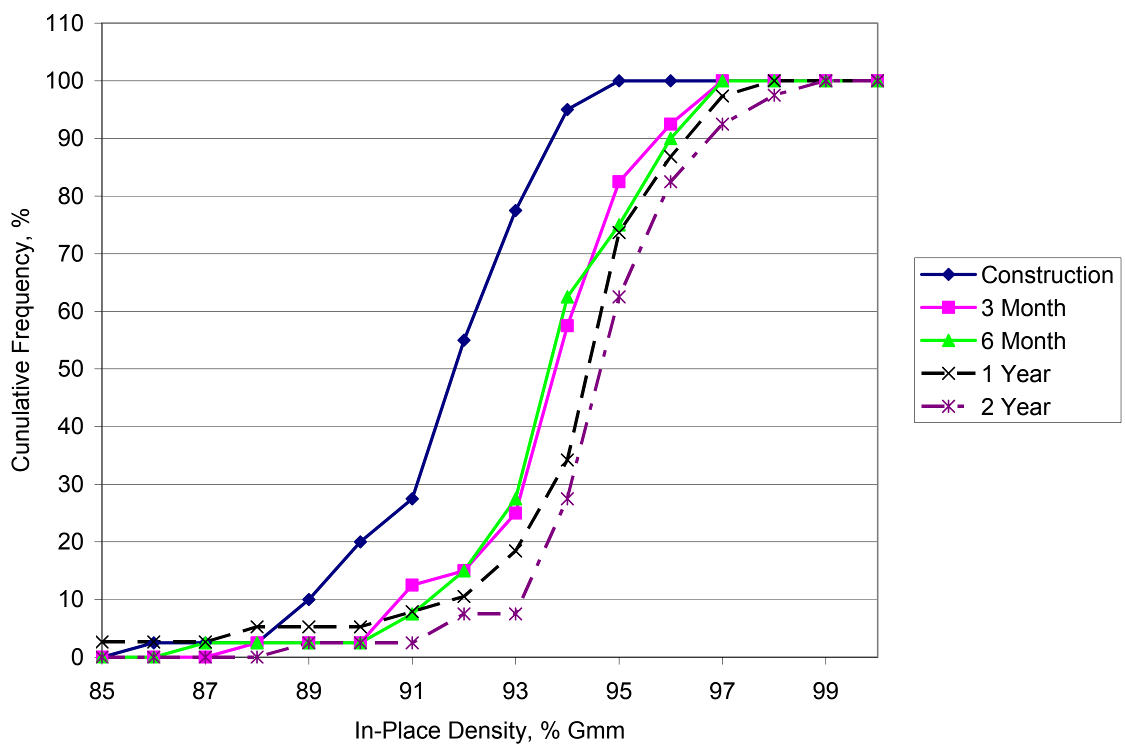

Figure 4.8. Cumulative Frequency Plot for In-Place Density by Sampling Period

Since there was a slight increase in density between the 1 and 2 year sampling intervals it was impossible to know if the pavements had reached their ultimate density after 2-years. The literature suggests that pavements reached their ultimate density after 2 to 3 years of traffic (*21, 51, 52, 58*), but could densify for a longer period of time (*57, 59*). Since the goal was to determine the Ndesign gyrations that produced samples with the same density as the ultimate density on the roadway, it was decided to extend the monitoring of the in-place density and take an additional set of cores after 4-years of traffic. The pavement condition survey conducted at the 4-year interval would also provide a better indication of the long-term performance of the pavement. Table 4.7 compares the 2-year and 4-year pavement densities for each project.

The average in-place density for all of the projects after both 2- and 4-years was 94.6 percent. Two tests were conducted to compare the 2-year and 4-year pavement densities, Student's *t*-test and a paired Student's *t*-test. In addition, an *F*-test was conducted to compare the sample variances prior to running the Student's *t*-test to determine whether the model with equal or unequal sample variances should be used. The *t*-test was used to compare the population means:

H_0: average 2-year density = average 4-year density,

H_1: average 2-year density $\neq$ average 4-year density.

Whereas the paired test examined the difference between the 2-year and 4-year density at each core site. In three cases, KY-1, NE-2, and NE-3 the *F*-test indicated that the sample variances were different between the 2-year and 4-year densities. The Student's *t*-test for unequal sample variances was used for these sites. The two-tail *p-value* is reported in all cases.

TABLE 4.7 Comparison of 2-Year and 4-Year Densities

Project	Roadway	% Gmm		Paired t-test		Population t-test	
		2-Year	4-Year	*p-value*	Significant $\alpha = 0.05$?	*p-value*	Significant $\alpha = 0.05$?
AL-1	Hwy 157	93.9	94.3	0.0886	No	0.2977	No
AL-2	Hwy 168	91.8	91.7	0.8968	No	0.9219	No
AL-3	Hwy 80	93.6					
AL-4	Hwy 84	94.3					
AL-5	Hwy 167	94.6					
AL-6	Andrews Rd	93.3	93.6	0.1202	No	0.4757	No
AR-1	I-40	94.2	94.2	0.2629	No	0.6918	No
AR-2	I-55	91.8	92.1	0.0941	No	0.4186	No
AR-3	I-40	94.7	94.6	0.7531	No	0.8442	No
AR-4	I-30	94.5	94.7	0.0894	No	0.3701	No
CO-1	Hwy 9	98.1	97.7	0.1063	No	0.3565	No
CO-2	Hwy 82	97.1	96.8	0.0196	Yes	0.4763	No
CO-3	I-70	95.7	95.7	0.6190	No	0.8492	No
CO-4	Hwy 13	94.2	94.4	0.4504	No	0.4613	No
CO-5	Hwy 82	93.8	93.3	0.0645	No	0.3068	No
FL-1	Davis Hwy	95.2					
GA-1	Hwy 13	96.5	96.3	0.3201	No	0.6385	No
IL-1	I-57	94.4	94.6	0.2052	No	0.5548	No
IL-2	I-64	95.2	95.3	0.0265	Yes	0.4559	No
IL-3	I-70	94.5	94.6	0.2154	No	0.5249	No
IN-1	Hwy 136	93.5	94.1	0.3286	No	0.3541	No
IN-2	I-69	94.1	94.8	0.0735	No	0.2087	No
KS-1	I-70	93.6	93.0	0.1085	No	0.2985	No
KY-1	CR1796	88.5	87.7	0.5281	No	0.4321	No
KY-2	I-64	94.1	94.4	0.0277	Yes	0.4279	No
KY-3	CR1779	94.2	94.4	0.4772	No	0.7774	No
MI-1	I-75	94.8	94.4	0.0944	No	0.1827	No
MI-2	Hwy 50	96.8	97.4	0.0091	Yes	0.3408	No
MI-3	Hwy 52	96.5	96.8	0.0279	Yes	0.1508	No
MO-1	I-70	96.5	NA[1]				
MO-2	Hwy 65	95.1	95.0	0.8276	No	0.8836	No
MO-3	I-44	95.6	95.5	0.6249	No	0.7958	No
NC-1	I-85	93.4	93.9	0.0062	Yes	0.0660	No
NE-1	Hwy 8	95.7	95.5	0.3002	No	0.6646	No
NE-2	Hwy 77	95.7	95.9	0.1870	No	0.3923	No
NE-3	Hwy 8	95.4	95.2	0.6303	No	0.6330	No
NE-4	I-80	97.2	97.4	0.0268	Yes	0.1964	No
TN-1	Hwy 171	94.3	93.6	0.0056	Yes	0.0427	Yes
UT-1	Hwy 150	93.7	93.6	0.7387	No	0.7850	No
WI-1	US 45	94.3	94.2	0.6521	No	0.8412	No

[1]Incorrect layer tested on four-year cores (Novachip added between 2- and 4-years).

The 4-year density was less than the 2-year density in 15 of 35 cases. If the 2-year and 4-year densities are not different, e.g. the 2-year density is the "ultimate" density, then lower values would be expected due to testing variability. The analyses indicate that the paired *t*-tests were significantly different ($\alpha = 0.05$) in 8 cases, and the average 4-year density was higher in 6 of those 8 cases. However, the paired *t*-test could be subject to differences due to variances in the longitudinal density of the pavement; although, generally pavement density is believed to be less variable in the longitudinal direction than in the transverse direction over short distances. The *t*-test to compare population means was only significantly different ($\alpha = 0.05$) in one case, TN-1. The average 4-year in-place density (93.6 percent) for TN-1 was less than the average 2-year density (94.3 percent). One possible explanation for this could be the onset of moisture damage. Based on these analyses, it is concluded that the ultimate density was achieved after 2-years of traffic.

Factors affecting pavement densification are of interest in this study. Figure 4.9 through Figure 4.11 show typical examples of the observed pavement densification with time. A figure for each project is shown in the Appendix. Figure 4.9 shows the densification of project CO-4. Project CO-4 is a relatively low volume pavement with 20-year design traffic less than 1 million ESALs and a posted speed limit of 55 mph. Figure 4.9 indicates that project CO-4 shows little densification with time or traffic. It should be noted that CO-4 was compacted to a relatively high as-constructed density (93.7 percent). Figure 4.10 shows the densification of project AL-1. AL-1 indicates a significant increase in density in the first 3 months after construction, after which time

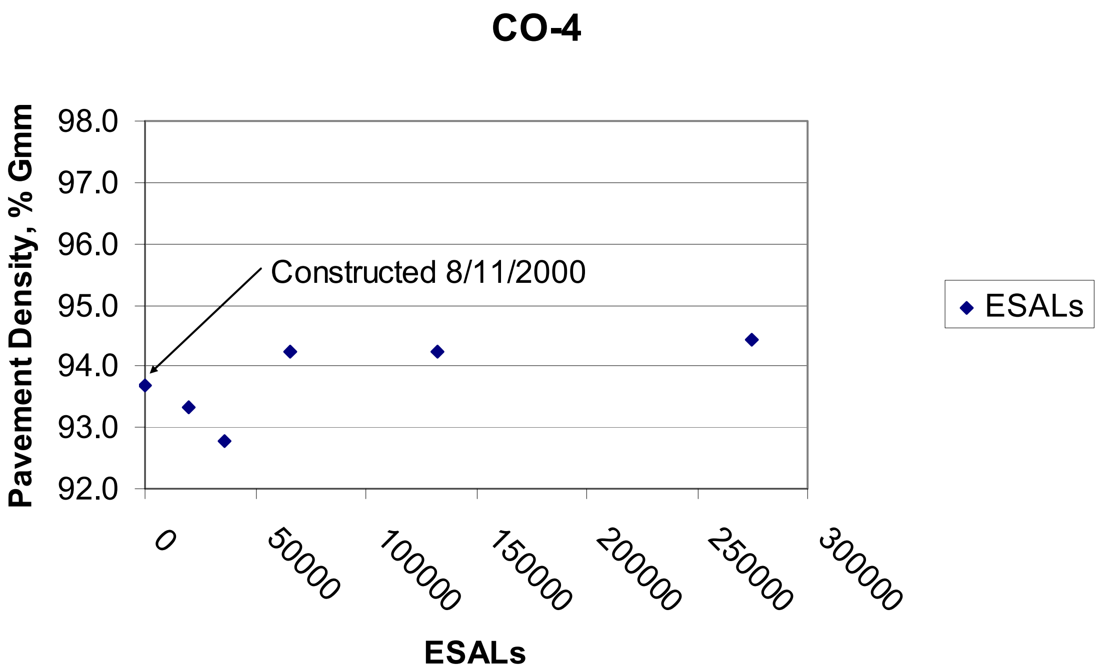

Figure 4.9. Densification of Project CO-4 with Time and Traffic.

the rate of densification levels off. The 20-year design traffic for AL-1 is 6.7 million ESALs. Project AL-1 was compacted to a low as-constructed density. AL-1 rapidly densified to an acceptable level in the first three months. Relatively little densification is observed after the first three months. This may be due to an increased rate of binder oxidization due to the low initial density. Figure 4.11 shows the densification of project MI-1. Project MI-1 is a high volume interstate with a 20-year design traffic of 16.0 million ESALs. The higher traffic volume appears to cause a steady rate of densification up until the 2-year sampling interval. The as-constructed density of project MI-1 was close to typical specifications. These examples demonstrate some of the apparent effects initial density and traffic can have on densification. These will be investigated in greater detail later in the report.

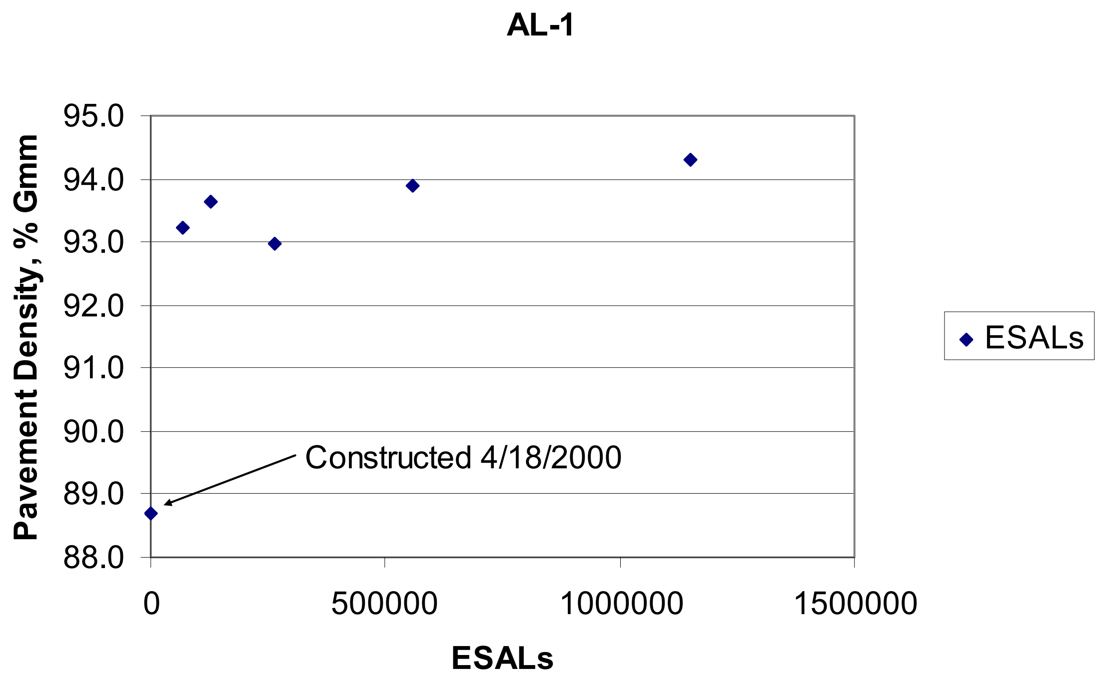

Figure 4.10. Densification of Project AL-1 with Time and Traffic.

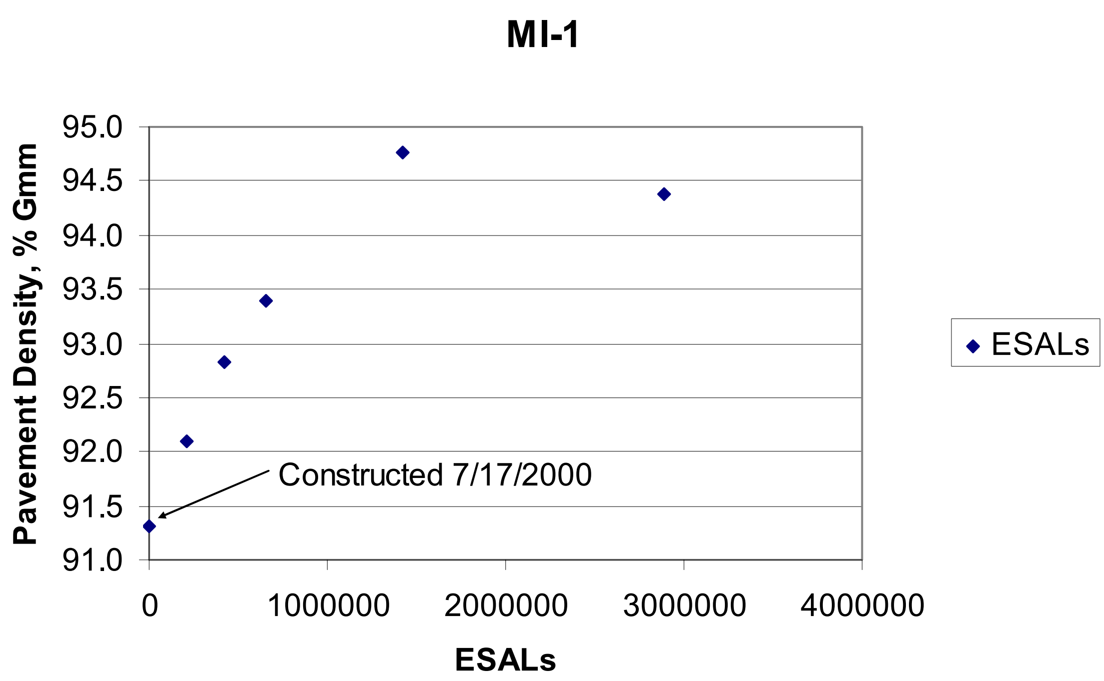

Figure 4.11. Densification of Project MI-1 with Time and Traffic.

Since the largest percent of pavement densification occurred in the first three months, the factors affecting the 3-month densification were investigated. The 3-month densification was calculated as the difference between the 3-month and as-constructed in-place density. An ANOVA was conducted using the GLM to examine factors which may have affected the densification after 3 months. The two to three samples from each project were used as replicates, each sample represented by average of three cores. Gradation, high temperature PG or bump in high PG, lift thickness to NMAS ratio, 2000 Ndesign level, and month of construction were considered as factors. 2000 Ndesign level is the Ndesign rounded to the levels adopted in 2000 (50, 75, 100, and 125). High temperature PG bump was considered as an alternate to High PG to better account for climatic differences between the sites. Month of construction was added based on speculation that pavements constructed in the fall would densify less than pavements constructed in the summer would.

The factor inputs are summarized in Table 4.1, presented previously. The results of the analysis using high temperature PG bump are shown in Table 4.8. High temperature PG bump (p = 0.016) and month of construction (p = 0.000) were identified as significant factors at $\alpha = 0.05$. A plot of the main effects is shown in Figure 4.12. The trends are generally as expected. There is a slight trend for increasing densification with increasing lift thickness to NMAS, except for the 6:1 ratio. Recall that there is only one project, MO-2, constructed at the 6:1 ratio. Densification decreases with high PG bump (1 grade bump would correspond to a 6 °C increase in high temperature PG), except for the half-grade bump resulting from the use of PG 67-22. As discussed previously, PG 67-22 was used by only two agencies, one of which tended to have low

TABLE 4.8 ANOVA (GLM) Results for 3-Month Densification

Source	Degrees of Freedom	Adjusted Sum of Squares	Adjusted Mean Squares	F-statistic	*p-value*	Sign.? α=0.05
Lift Thickness to NMAS	4	4.910	1.227	0.86	0.490	No
High Temperature PG Bump	3	16.491	5.497	3.86	0.012	Yes
2000 Ndesign	3	1.257	0.419	0.29	0.830	No
Month of Construction	6	59.405	9.901	6.95	0.000	Yes
Gradation	1	0.437	0.437	0.31	0.581	No
Error	92	131.141	1.425			
Total	109					

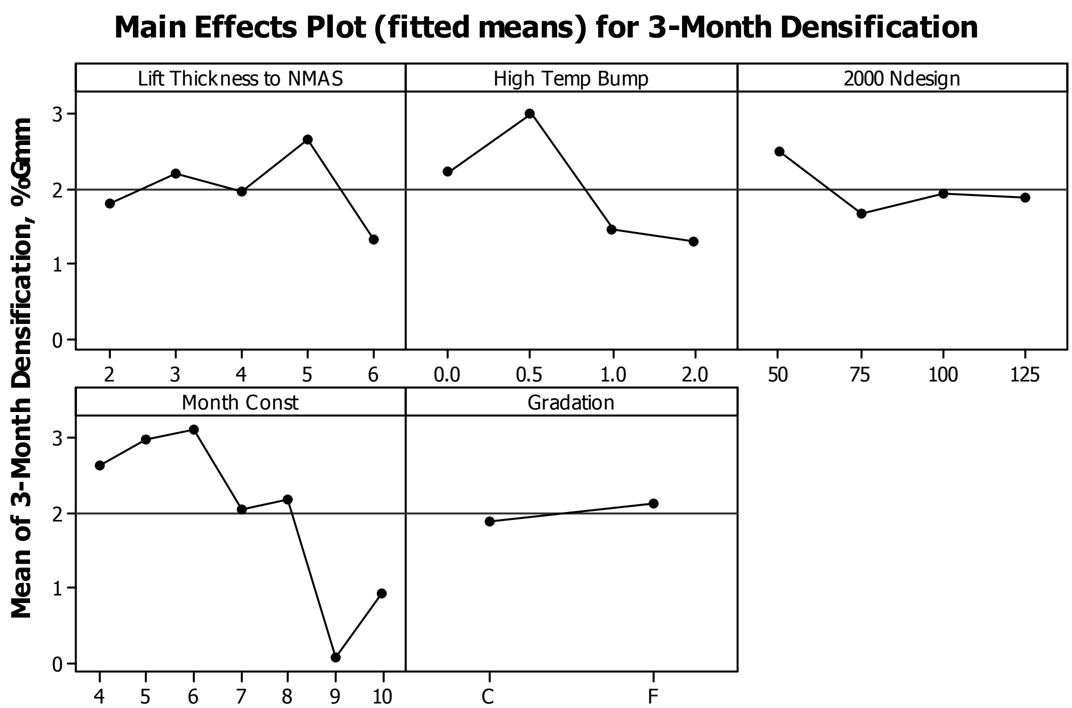

Figure 4.12. Main Effects Plot for Factors Effecting 3 Month Densification.

as-constructed density. Projects with low as-constructed density would be expected to densify more under traffic. Ndesign is neutral except for 50 gyrations. As noted previously, only one 50 gyration project was sampled, KY-1, with no in-place density specifications and a very low as-constructed density. This suggests that the current tiered

Superpave design system, with differing binder grades, aggregate properties and Ndesign levels generally accounts for the effect of varying traffic. Fine mixes appear to densify slightly more than coarse mixes. The most interesting effect may be that of month of construction. The numerical month is shown on the x-axis, e.g. April = 4. It appears that projects constructed between April (4) and June (6) densified the most, approximately 1 percent more than projects constructed in July (7) and August (8). The fact that projects constructed in April (4) densified slightly less than the projects constructed in June (6) again most likely illustrates the effect of binder aging since the projects constructed in April (4) would have aged slightly before the hottest summer weather. As expected, the projects constructed in September (9) and October (10) appear to have densified approximately 1 to 2 percent less than the projects constructed in mid-summer.

The ANOVA was re-run using the amount of densification after 2 years of traffic as the response variable. High PG bump (p = 0.007) was still significant at α = 0.05. Month of construction (p = 0.068) was not significant at α = 0.05, but was significant at α = 0.10. Figure 4.13 illustrates the fitted means of the effect. This indicates that month of construction has a strong influence on the long-term densification of a project with approximately a 2 percent change in densification between pavements constructed in May (5) as compared to pavements constructed in October (10). This emphasizes the need to obtain good compaction during late season paving. Compaction requirements cannot be waived with the assumption that the pavement will densify to an acceptable level with the onset of hot weather the following year.

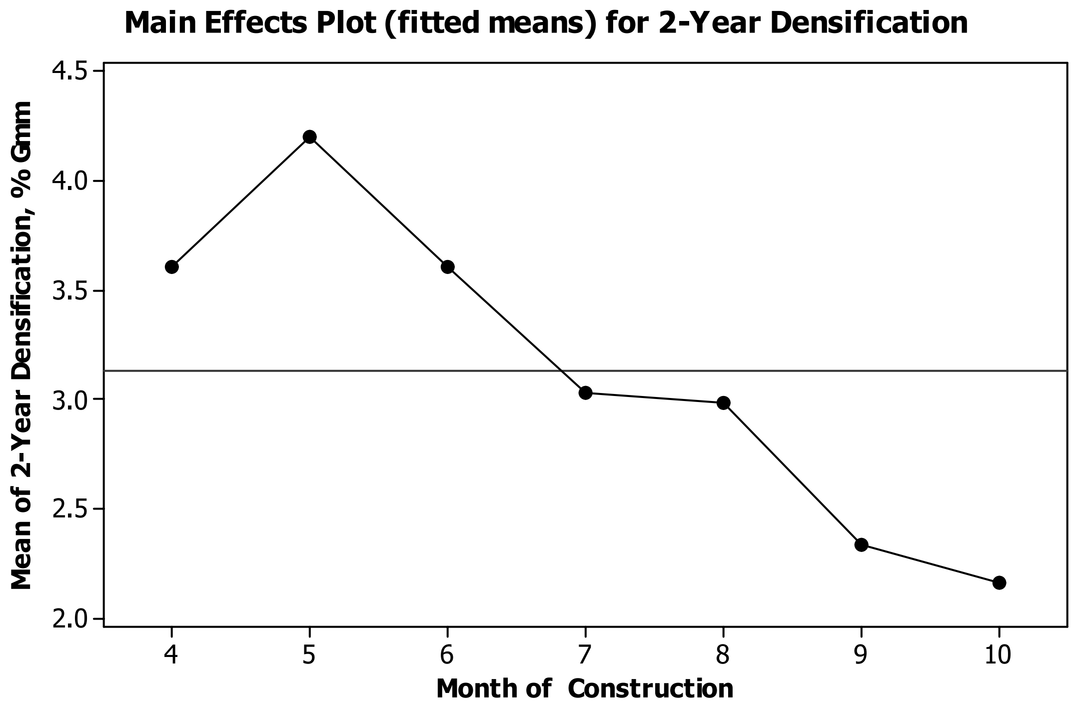

Figure 4.13. Main Effect Plot for Month of Construction on 2-Year Densification.

Data from the 2000 NCAT Test Track was analyzed in addition to the data from the field projects. The NCAT Test Track offered a unique opportunity to study pavement densification and it's relationship to the number of design gyrations, since all of the sections receive the same traffic, have the same base and subgrade support and are exposed to the same climatic conditions. Thirty-two of the test track sections were designed using Superpave and are included in the following analysis. The 32 sections represent a range of aggregate types, NMAS, and gradations.

One of the objectives of the work at the track was to evaluate densification of HMA. Cores, for evaluating densification, were taken at various traffic levels from the left wheel path of the last 25 feet of each section. When the test track was constructed, paving was carried past the end of the section, and the pavement cut back prior to constructing the next section. In this manner, the last 25 feet of the section should be representative mix. Initially, traffic began in September 2000 with only one truck in operation, three trucks were operational in November 2000 and traffic was fully implemented (four trucks) in February of 2001. For the first three months, cores were taken on a monthly basis and later quarterly.

The cores are sawed into their respective layers and the bulk specific gravity of each layer determined using AASHTO T-166. Density of samples having greater than 2 percent water absorption was determined using the Corelok device. In-place air voids were calculated using the construction maximum specific gravity values. Figure 4.14 shows the average test track pavement density as a function of ESALs for the Superpave sections through the completion of 10 million ESALs in December 2002. The figure indicates that the initial construction densities were slightly lower for the PG 76-22 surface layers as opposed to the other layers. For both the PG 67-22 and PG 76-22 sections, the construction densities were less for the upper lift. A second-order polynomial was fit to the data for each binder grade/lift combination.

The data seems to indicate distinct rates of densification for each binder grade/lift combination related to time after construction and temperature (season). There appears to be an initial seating of the mix between the first and third data points taken in September and December of 2000, respectively. The average pavement density appears to continue to increase from December 2000 (third data point) through October 2001 (data point at approximately 4.5 million ESALs). There is little increase in pavement density between October 2001 and June 2002 (data point at approximately 7.5 million

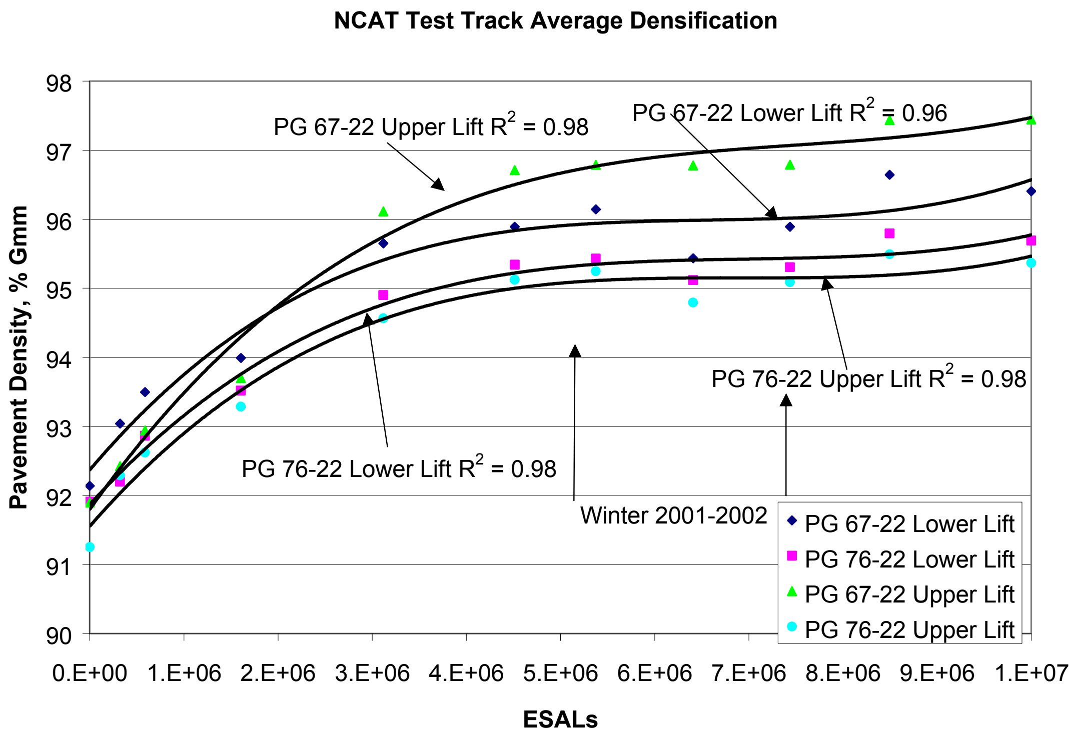

Figure 4.14. Average Test Track Pavement Densification (*78*).

ESALs). In fact, the average density for all but the PG 67-22 upper lift sections appears to decrease in March 2002 (data point at approximately 6.5 million ESALs). The change in density during the summer of 2002 (7.5 to 8.5 million ESALs) is similar to that which occurred during the summer of 2001 (3.0 to 4.5 million ESALs). A slight decrease in density was observed between September and December 2002 with the exception of the PG 67-22 upper lifts, which increased slightly.

There appears to be a significant difference in the rate of densification based on binder grade. As expected, the sections with the softer binder, PG 67-22, densified faster. This was true for both the upper and lower lifts. Further, it appears that for the PG 67-22 sections, the lower lift, which was 50 mm below the surface of the pavement, did not densify as fast as the PG 67-22 surface lift. The difference in density was approximately one percent from approximately 4 through 10 million ESALs. The difference was not apparent prior to 4 million ESALs because the lower lifts were constructed at a higher initial density. Recall that Blankenship (*64*) did not find a relationship between traffic and pavement densification for layers deeper than 100 mm from the pavement surface. Based on the reduced vertical pressure calculated using Boussinesq theory, Brown and Buchanan (*2*) recommended Ndesign be reduced by 28 percent or approximately one gyration level for layers deeper than 100 mm from the pavement surface. Brown et al. (*78*) also note that permanent deformation (and densification) essentially stopped when the air temperature was less than 28 C. Important findings from the densification of the 2000 NCAT Test Track related to this study include (*78*):

1. Modified binders (2 High PG bump) rutted approximately 60 percent less than unmodified (0.5 High PG Bump) based on an average rut depth after 10 million ESALs of 1.7 mm for the modified mixes and 4.1 mm for the unmodified mixes. Densification was reduced by 25 percent for the surface mixes containing modified binders with an average reduction in air voids of 4.1 percent for the modified mixes and 5.6 percent for the unmodified mixes.
2. The densification of pavement layers 50 mm from the pavement surface was approximately 1 percent less than for surface layers.

4.2.4 Determination of Ndesign to Match Ultimate In-Place Density

Three different analyses were performed to relate Ndesign to the ultimate in-place density. Each of these analyses will be described in the following section. First, regressions were performed between the accumulated traffic after two years and the predicted Ndesign values. The data were subdivided and potential outliers examined in an attempt to improve the relationship. Second, regressions were performed between the accumulated ESALs at each of the sampling intervals (3-months, 6-months, 1-year, 2-years and 4-years) and the predicted gyrations to match the in-place density at each of those intervals. Third, models were developed to predict Ndesign, which accounted for as-constructed density, high temperature PG grade and traffic. In addition, the ultimate in-place density was compared to the density at the agency specified Ndesign.

The number of gyrations necessary to obtain the in-place density after two years of traffic or ultimate density was determined by performing a linear regression between the estimated sample density at a given number of gyrations and the Log gyrations. This was done both for the average densities and pill heights for a project as well as the average density and sample height for each sample within a project (average of 3 SGC pills). The pill height and density at 8, 25, 50, 75, 100, 125 and 160 gyrations were used for the regression to determine the slope and offset. The heights and pill densities from 8, 25, 50, 75, and 100 gyrations were from the SGC pills compacted to 100 gyrations; the pill heights and densities for 125 and 160 gyrations were from the SGC pills compacted to 160 gyrations. It should be noted that the SGC pill densities at 100 and 160 gyrations were measured, but the other pill densities were estimated using Equation 6. References (*69, 71, 76*) discuss the errors in back calculation of sample density. Due to the scope of the project, back calculation was unavoidable. Once the slope and offset were determined, the number of gyrations to match the ultimate density could be calculated. This was done for both the Pine and Troxler SGCs. Figure 4.15 shows a plot of the average (for each project) Ndesign to match the 2-year in-place density for each SGC versus estimates of the accumulated traffic after 2-years. The figure is shown with an arithmetic scale, to better show the difference in predicted gyrations between the Pine and Troxler SGCs. The best fit line in the figure is a power model which would produce a straight line on a Log-Log plot. The R^2 values indicate a weak correlation between Log 2- year ESALs and Log predicted gyrations. There appear to be a number of potential outliers. All of the potential outliers are 9.5 mm NMAS mixes and occurred with the Troxler compactor. It also appears that the predicted gyrations for the Troxler compactor are approximately 20 gyrations higher than the predicted gyrations for the Pine compactor.

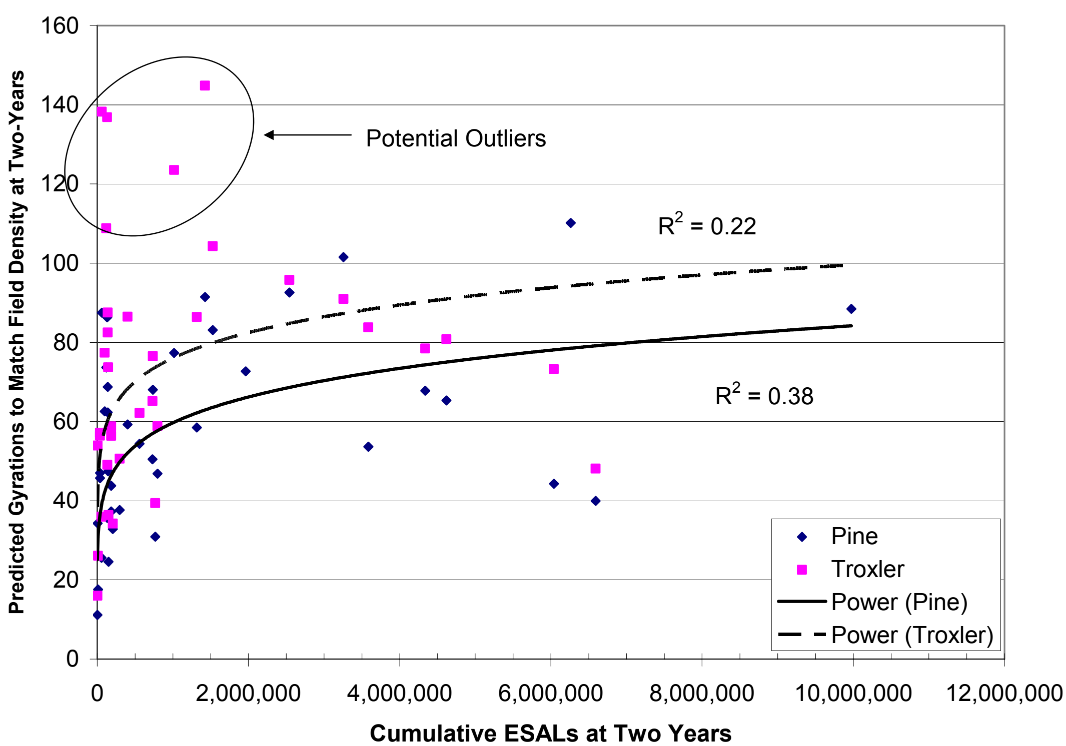

Figure 4.15. Predicted Gyrations to Match Two-Year Density.

Significant efforts have been made to study the differences in sample density produced by different models and units of gyratory compactors. One influencing factor that has been identified is the dynamic internal angle (DIA) of gyration. The internal angle of gyration can be measured using a device called the dynamic angle verification kit (DAVK). FHWA proposed a DIA of 1.16 ± 0.03 degrees (*48*). In a study conducted for Alabama DOT, Prowell et al. (*49*) determined that a change of DIA of 0.1 degrees will result in a change of 0.01 G_{mb} units as shown in Figure 2.13. Dalton (*44*) found a similar relationship with a change of DIA of 0.1 degrees resulting in a change of 0.014 G_{mb} units. After the completion of this Alabama DOT study, the DIA of the Pine compactor was measured as 1.23 degrees as part of the Alabama DOT study. The DIA of the Troxler compactor was not measured at that time due to a problem with the electronics but was later measured as 1.02 degrees. Using the first relationship, the compacted sample densities from both compactors were adjusted to that which would have been produced if both compactors had been set to a DIA of 1.16 degrees. The predicted gyrations to match the in-place density after two-years of traffic were then recalculated and are summarized in Figure 4.16. As shown in Figure 4.16, the best fit line for the predicted gyrations to match the in-place density from both compactors adjusted to an internal angle of 1.16 degrees falls along the line of equality. The best fit line for the original data is shown for comparison.

The data in Table 4.9 are sorted by the 20 year design traffic. Lines have been added to the table to separate between the current design traffic levels. In Figure 4.16 and Table 4.9, there appear to be a few potential outliers in the adjusted data, specifically the Pine results for IL-3 and the Troxler results KY-2 and MI-1. The two Troxler points also appeared to be potential outliers in Figure 4.15. One tool for evaluating potential outliers in a relationship is to look at the standardized residual. The standardized residual

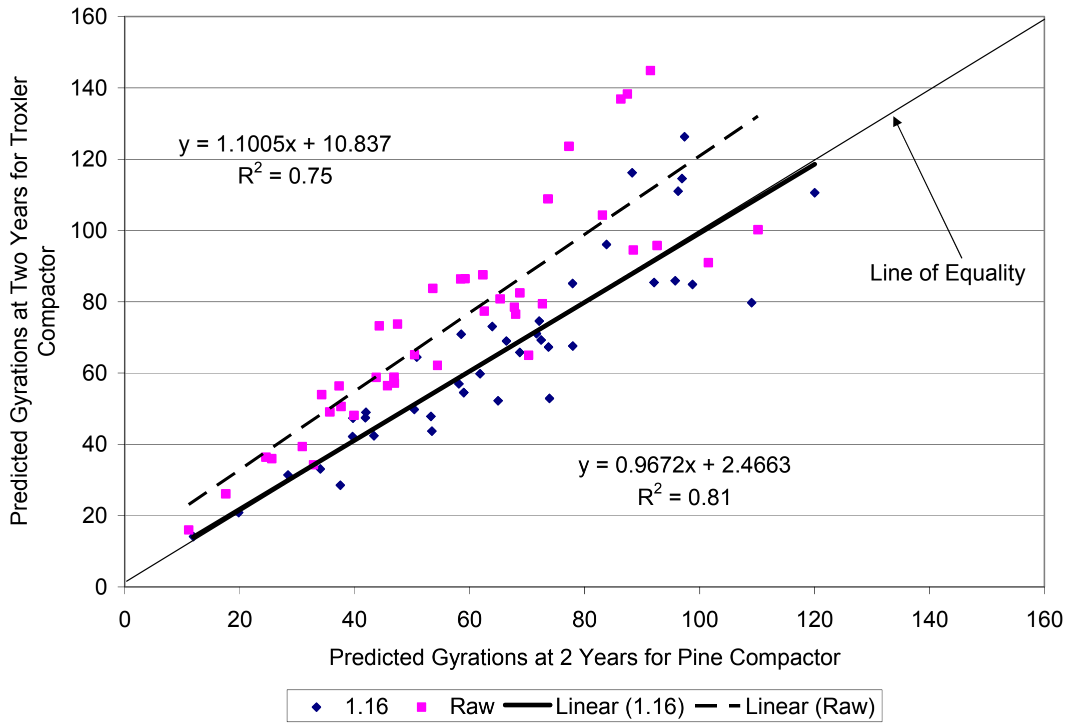

Figure 4.16. Comparison of Predicted Gyrations to match In-Place Density after Two-Years with and without Correction for DIA.

is the difference between the observed and the fit values divided by the square root of the mean square error (MSE). Montgomery (*85*) states that standardized residuals which exceed ± 3.0 may be considered outliers. The standardized residuals for IL-3, KY-2, and MI-1 were -2.44, 2.45, and 2.57, respectively; this indicates that they should not be removed as outliers. The other three potential outliers in Figure 4.15, FL-1, MI-2 and MI-3, have standardized residuals of 1.58, 1.09, and 1.33, respectively, when corrected to a DIA of 1.16 degrees in Figure 4.16. Research by Moseley et al. (*86*) indicated that the measured DIA is affected by the HMA mixture. Nova Scotia granite, the same as that used in project FL-1, produced the largest differences between compactors; 9.5 mm NMAS mixes also showed larger differences.

Observation of Table 4.9 indicates that very few of the predicted gyrations to match the in-place density after 2 years exceed the currently specified Ndesign values.

TABLE 4.9 Original and Adjusted Gyrations to Match In-Place Density at 2 Years

Project	20 Tear Design Traffic, ESALs	Average Predicted Gyrations to Match 2 Year Density					
		Pine 1.23 Degrees	Pine 1.16 Degrees	Pine Std.	Troxler 1.02 Degrees	Troxler 1.16 Degrees	Troxler Std.
KY-1	53,706	11	12	1.3	16	14	1.2
KY-3	84,028	34	40	17.4	54	47	22.9
AL-6	143,958	18	20	2.1	26	21	1.3
NE-3	365,719	46	53	10.6	56	44	13.6
NE-1	383,385	47	65	53.3	57	52	39.6
CO-3	523,624	63	69	11.0	77	66	7.4
CO-4	720,911	36	40	11.7	49	42	8.7
CO-1	756,789	62	72	21.7	88	75	18.9
UT-1	771,982	26	28	7.5	36	31	8.8
FL-1	811,658	87	97	14.7	138	115	19.4
CO-2	1,017,593	44	50	13.9	59	50	13.7
CO-5	1,017,593	37	42	13.5	56	49	15.4
MI-2	1,250,146	74	84	27.4	109	96	32.8
NE-2	1,450,960	69	78	15.1	82	68	13.2
MI-3	1,515,200	86	96	2.5	137	111	5.2
AL-5	1,809,675	25	59	9.2	36	55	9.1
IN-1	1,850,992	47	51	5.0	74	64	5.4
TN-1	3,490,393	33	37	10.1	34	29	10.2
AL-2	3,610,001	38	42	16.5	51	47	20.9
AL-4	4,899,406	59	66	3.0	86	69	4.9
AL-1	6,748,142	54	59	9.2	62	55	9.1
GA-1	8,803,521	47	53	10.7	59	48	5.4
AL-3	8,861,352	31	34	0.5	39	33	1.1
KS-1	10,075,962	50	58	21.6	65	57	20.4
KY-2	12,438,605	77	88	43.9	124	116	57.5
MO-2	12,517,675	68	74	3.6	77	67	6.9
WI-1	14,614,748	58	64	4.9	86	73	9.1
MI-1	15,966,398	91	97	8.9	145	126	15.6
NE-4	20,084,248	83	92	3.0	104	85	5.4
IL-1	26,285,917	73	78	7.2	79	85	10.6
MO-1	27,546,007	93	99	13.0	96	85	4.9
IL-3	44,466,336	102	109	10.6	91	80	6.6
IN-2	45,150,555	54	59	9.1	84	71	9.1
IL-2	46,344,297	70	74	17.6	65	53	15.0
AR-1	48,726,562	65	72	16.4	81	71	13.9
MO-3	53,683,941	68	72	4.8	78	69	4.4
NC-1	73,918,507	44	62	16.0	73	60	14.7
AR-2	91,370,805	40	43	5.6	48	42	7.3
AR-4	97,890,077	110	120	3.1	100	111	9.4
AR-3	170,842,507	88	96	14.3	94	86	11.0

The Pine and Troxler results for FL-1 (97 and 115, respectively) exceed 75 gyrations in the > 0.3 to < 3 million ESALs category. The Troxler results for MI-3 (111), KY-2 (116), and MI-1 (126) all exceed 100 gyrations in the >3 to < 30 million ESALs category. The higher numbers for the Troxler compactor may be partially attributed to error in the correction to a DIA of 1.16 degrees. It is expected that if DIA of the Troxler compactor used in this study were measured with the DAVK using these mixes, the measured DIA would be less than the DIA of 1.02 degrees measured in the Alabama DOT study.

Figure 4.17 shows the predicted gyrations to match the two-year density, corrected to a DIA of 1.16 degrees, versus the two-year ESALs. Comparison of Figure 4.17 to Figure 4.15 (showing the uncorrected gyration data) indicates that correction of the gyratory data to a common DIA produces similar relationships between two-year

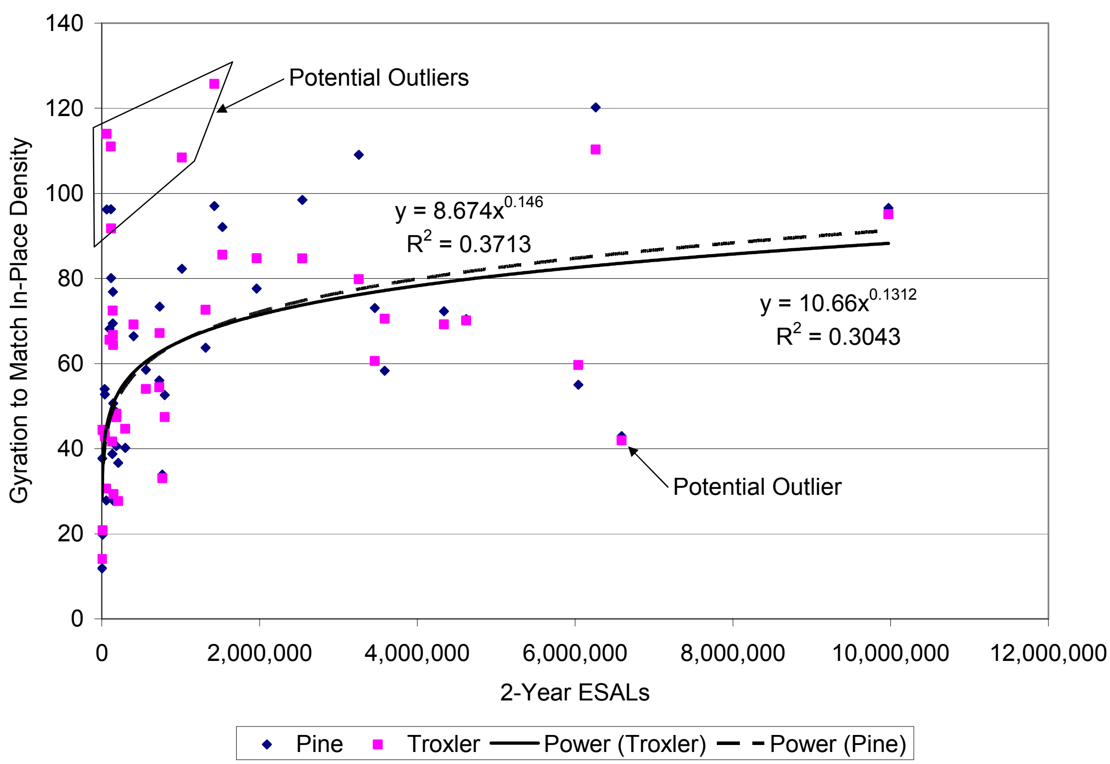

Figure 4.17. Predicted Gyrations to Match Two-Year Density Corrected to a DIA of 1.16 Degrees.

ESALs and predicted gyrations for the two SGCs, but does not significantly improve the R^2. The same five points discussed previously appear to be potential outliers. An additional point, AR-2, appears to be a potential outlier having a low number of predicted gyrations (43) for a high 2-year traffic level (6.6 million ESALs).

Figure 4.18 shows the predicted gyrations to match both the 2-year and 4-year in-place densities versus the 20-year design ESALs. Previously, it was shown that there was no statistical difference between the 2-year and 4-year in-place density. Figure 4.18 shows a slight increase in predicted gyrations to match the 2-year and 4-year in-place densities for both the Pine and Troxler compactors. However, this appears to be somewhat driven by project AR-4. The in-place density for project AR-4 increased by 0.2 percent between 2-years and 4-years. This resulted in an approximately 9 gyration

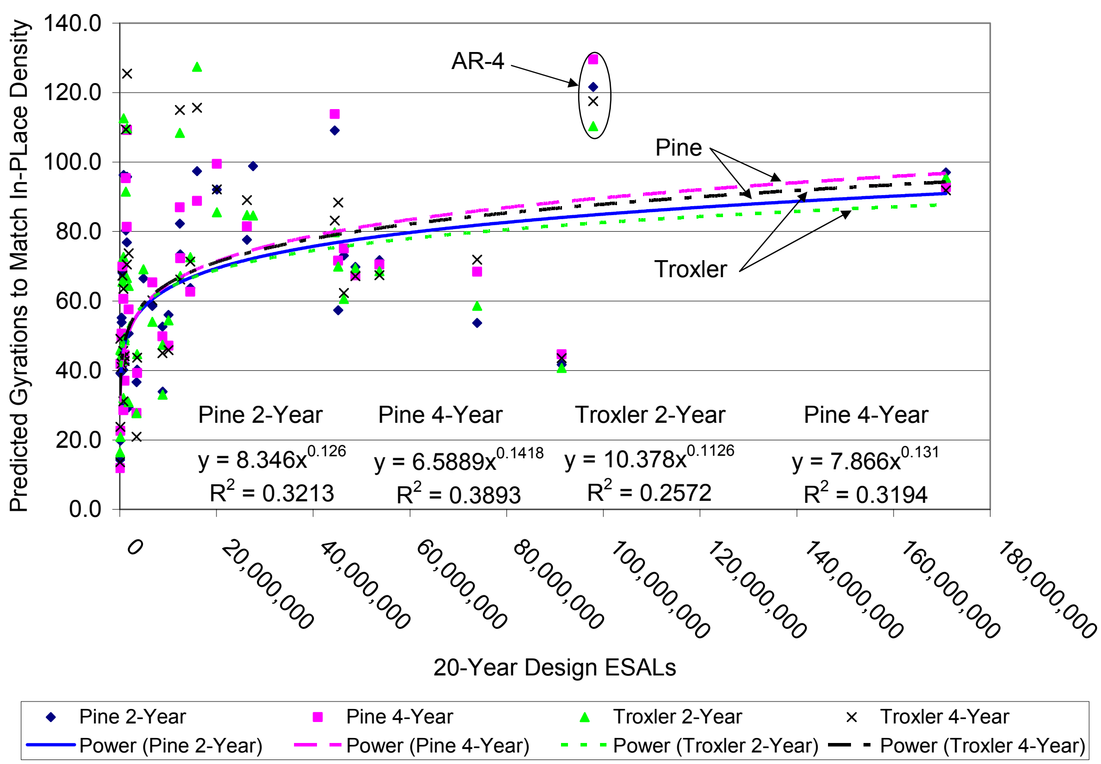

Figure 4.18. Predicted Gyrations to Match In-Place Density Corrected to a DIA of 1.16 Degrees.

increase between 2- and 4-years. The slight increase in R^2 for the 2- and 4-year relationships is most likely due to missing 4-year data, particularly FL-1.

Another way to evaluate whether or not the current Ndesign values are correct is to compare the laboratory air voids at the Ndesign specified by the agency with the in-place density after 2 years of traffic or ultimate density similar to Figure 2.19 (*73*). Figure 4.19 shows the air voids at Ndesign (1.16 degrees) versus the 2 year in-place air-voids for each of the samples within a project. As expected based on the data presented so far, there is a great deal of scatter in the data. However, the relationship is significant at $\alpha = 0.05$. Based on the regression line, at a void level at Ndesign of 4-percent the average in-place air voids are 5.5 percent, or 1.5 percent higher than design. Only a few points fall below the line of equality. This indicates that the pavements have not

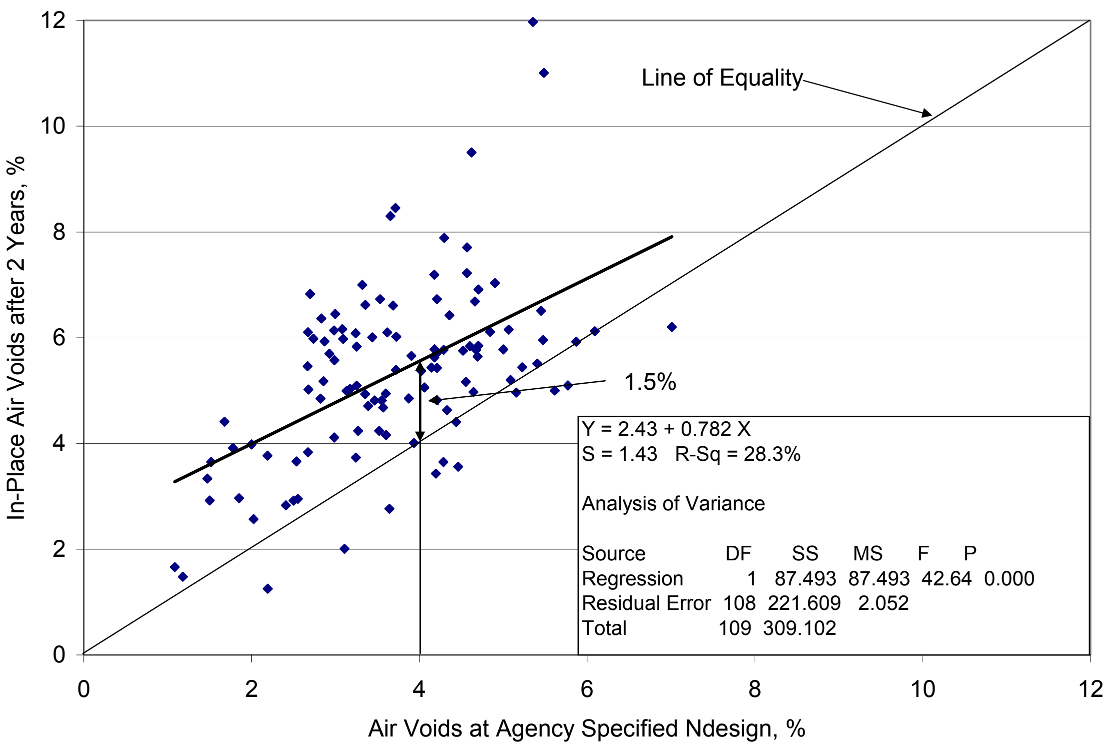

Figure 4.19. In-Place 2 Year versus Agency Specified Ndesign Air Voids.

densified to their design levels. It further suggests that the Ndesign levels may be too high. By comparison, Harmelink and Aschenbrener (*73*) found a difference of 1.2 percent after 5 to 6 years based on 22 projects, again indicating that the design levels were too high.

Similar to the 40 field projects, the numbers of gyrations to match field density were back-calculated for the 28 Superpave sections at the 2000 NCAT Test Track. Two Troxler Model 4141 SGC, the same Troxler model used in the field study, were used to compact the SGC samples at the 2000 NCAT Test Track. Three replicate samples were compacted for each sublot. The samples were compacted to the same N_{design} level used in the mix design, generally 100 gyrations. The bulk specific gravities of the samples were determined with AASHTO T166. All of the heights were digitally recorded and used in the back-calculation. The data have been adjusted to an internal angle of 1.16 degrees. The internal angles of gyration for the two compactors used during the construction of the 2000 NCAT Test Track were not known and could not be measured since these compactors were no longer operational. Therefore the average angle, 1.02 degrees, determined for that Troxler model in a previous study was used when adjusting the data to a DIA of 1.16(*49*). Figure 4.20 shows the average number of gyrations to match the in-place density versus ESALs for a given group of Test Track sections. The data are subdivided by binder grade (PG 67-22 or PG 76-22) and lift (upper surface lift or lower lift 50 mm deep). Second order polynomials provided good fits to the data. On average, there was a 25 gyration difference between predicted gyrations to match the upper (97 gyrations) and lower (72 gyrations) PG 67-22 lifts at 10 million ESALs and there is a 37 gyration difference between the predicted gyrations to match the upper lifts of PG 67-22

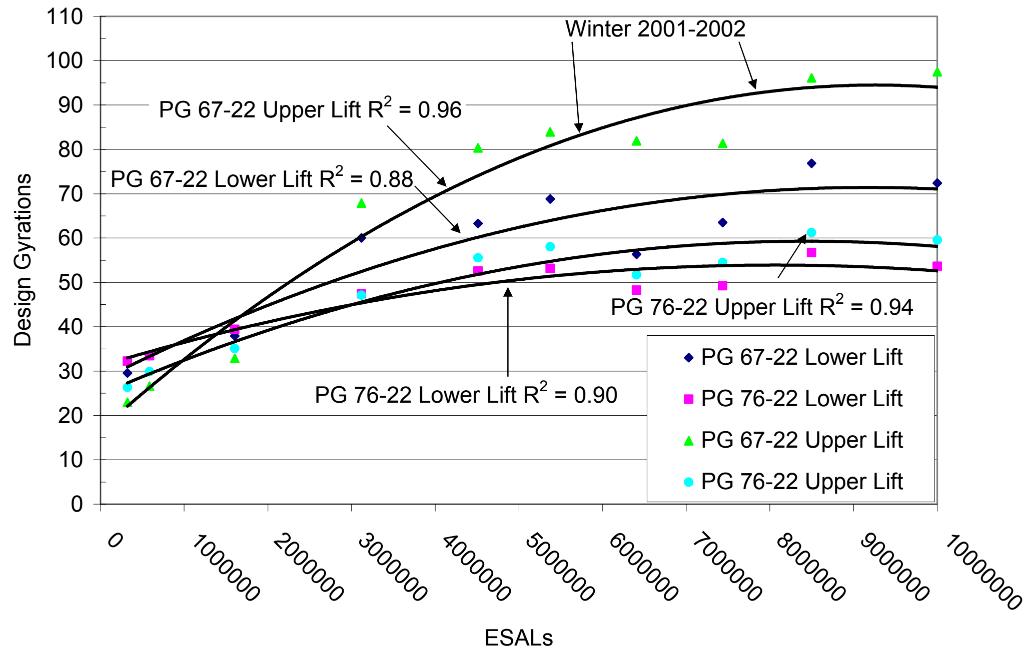

Figure 4.20. Average Gyrations to Match 2000 NCAT Test Track Density.

(97 gyrations) and PG 76-22 (60 gyrations) at 10 million ESALs. As noted previously, no densification occurred during the winter of 2001-2002. Although the relationship between the average predicted gyrations and applied traffic is strong, there is a great deal of scatter in the data. Figure 4.21 presents the actual data for the PG 67-22 and PG 76-22 upper lifts where each point represents the number of gyrations to match the in-place density for a given section at a given number of ESALs. It is apparent from Figure 4.21 that the scatter in the data is much larger for the PG 76-22 sections than for the PG 67-22 sections. This is evidenced by the R^2 = 0.63 for the PG 67-22 mixes and R^2 = 0.18 for the PG 76-22 mixes. It is possible that if the field data were similarly subdivided, a better relationship could be found from which to predict the appropriate Ndesign levels to match ultimate density.

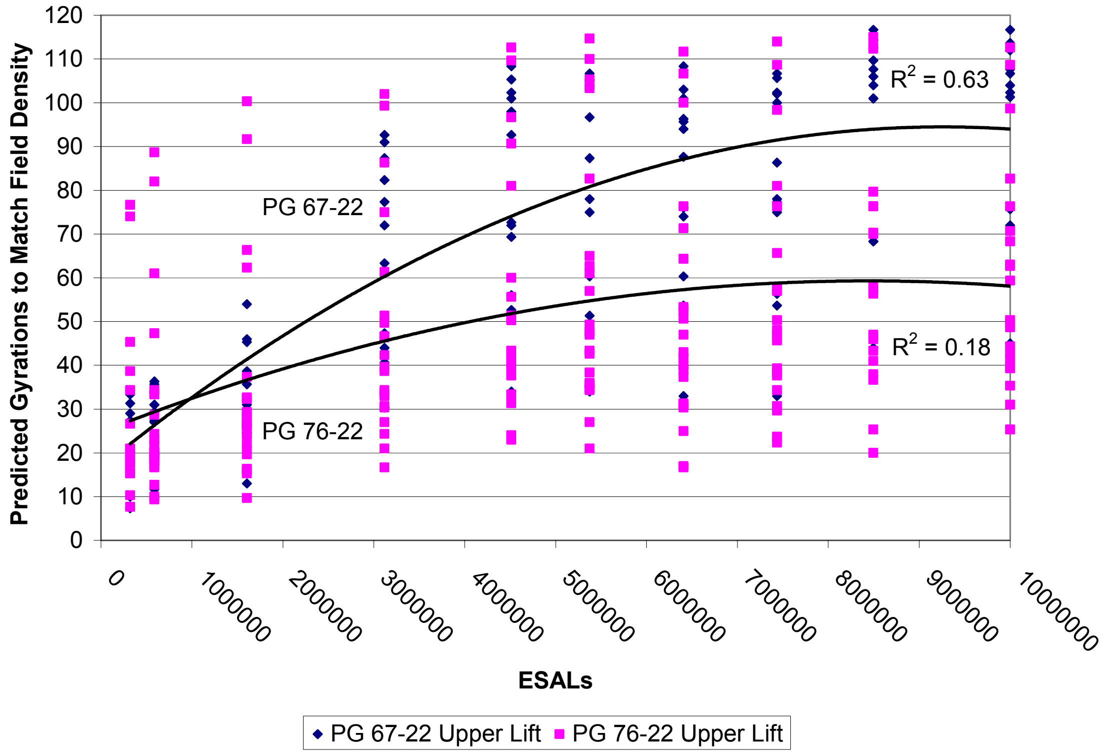

Figure 4.21. Predicted Gyrations to Match 2000 NCAT Test Track Density.

The predicted gyrations, corrected to an internal angle of gyration of 1.16 degrees, to match the two-year in-place density from the NCHRP 9-9 (1) field projects, excluding the nine projects which used PG 76-22, are shown in Figure 4.22. It is evident from the figure that there is still a great deal of scatter in the data. Three projects with a high number of predicted gyrations for a low design traffic level are CO-1, MI-2, and MI-3. All three of the projects were constructed with PG 58-28 binder and were constructed with crushed gravel aggregate. Project FL-1 was constructed to 91.8 percent Gmm and densified to 95.2 percent Gmm after two years. Nothing appears to be unusual about the densification; however, a high number of gyrations were predicted to match the two-year density for a relatively low traffic volume. The laboratory voids for FL-1 were high with

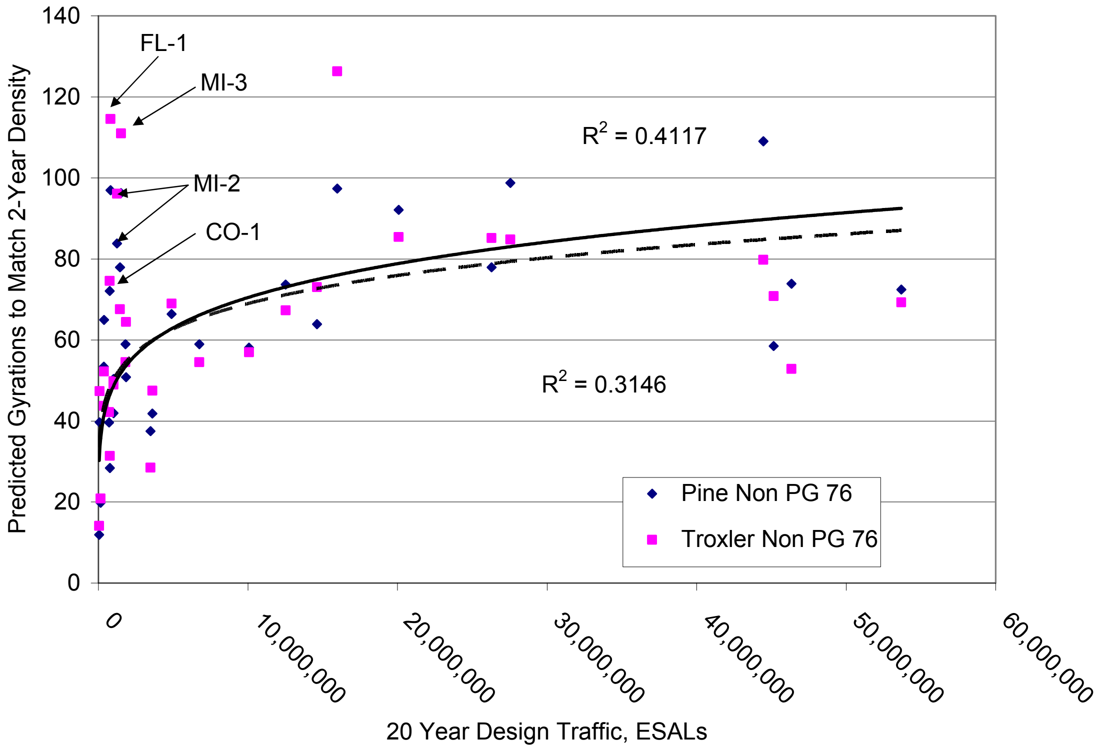

Figure 4.22. Predicted Gyrations Excluding Projects Using PG 76-22.

air voids at the agency specified Ndesign of 5.1 and 5.6 percent, respectively for the Pine and Troxler compactors.

A regression was performed using Log 20-year ESALs as a predictor for Log gyrations. The average Pine and Troxler results at 1.16 degrees were combined resulting in two data points for each project. The data for CO-1, MI-2, MI-3 and FL-1 were eliminated from the data set. The R^2 = 0.52 indicates a weak correlation between Log 20-year ESALs and Log predicted gyrations. However, the Troxler results for MI-1 were indicated as a possible outlier with a standardized residual of 3.41. The Troxler results for MI-1 were removed from the data set and the regression re-run. The resulting R^2 (0.57) still indicates a weak correlation, but improved. Figure 4.23 shows the standardized residuals versus the fitted value for the regression. The residuals appear to be well distributed. Figure 4.24 shows a plot of the regression with the 80 percent confidence interval. The regression was used to predict fitted values for the currently specified traffic levels. The 80 percent prediction interval for the regression is also shown.

Using the regression shown in Figure 4.24, the number of gyrations for each of the currently specified Superpave traffic levels was calculated along with the 80 percent confidence prediction interval (Table 4.10). The 80th percentile, calculated using the data in Table 4.9 is shown for comparison. The data for the 80th percentile includes the projects constructed with PG 76-22, while the predicted values from the regression does not include the projects constructed with PG 76-22. From Table 4.10 it can be seen that the high side of the interval for the 80 percent prediction interval approximately matches the currently specified gyration levels (*4*). However, the original Ndesign experiment used the predicted value with 50 percent confidence (*64*). The data from the 2000 NCAT Test Track and the 80th percentile data support the fact that an Ndesign of 100 gyrations should be adequate for very high traffic levels (Figure 4.20). The 20-year design traffic for the 2000 NCAT Test Track would be in excess of 100 million ESALs. From Figure 4.24 it can be seen that the predicted gyrations change very rapidly at design traffic levels less than approximately 3 million ESALs. Caution is required when recommending Ndesign for between 0.3 and 3 million design ESALs.

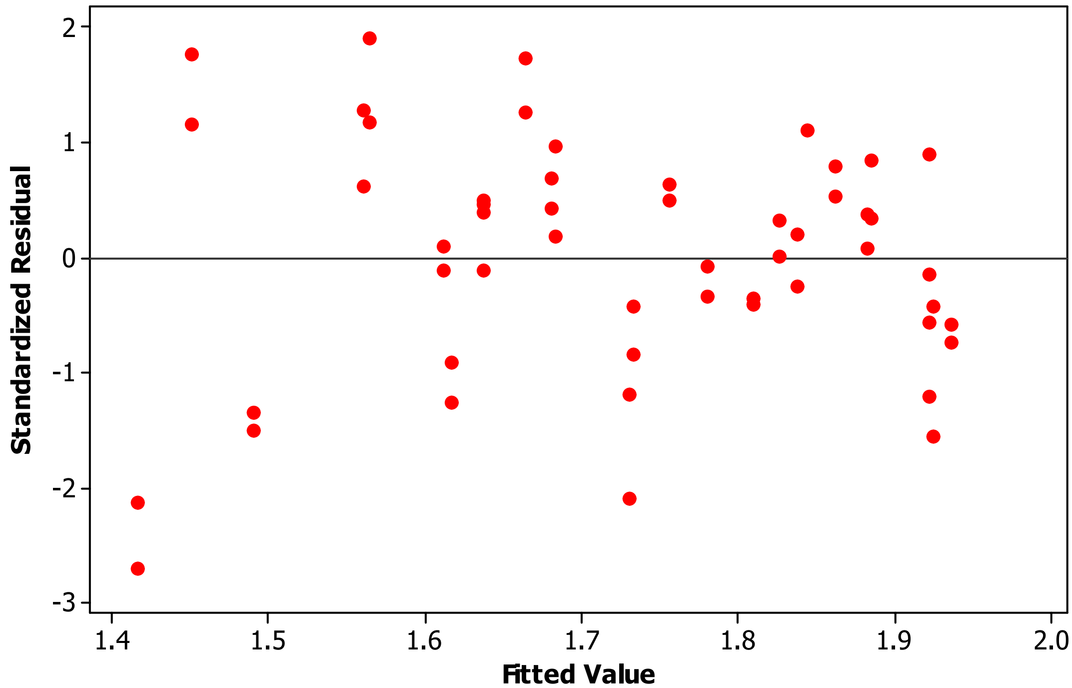

Figure 4.23. Standardized Residuals versus Fitted Mean for Log Predicted Gyrations versus Log 20-Year ESALs.

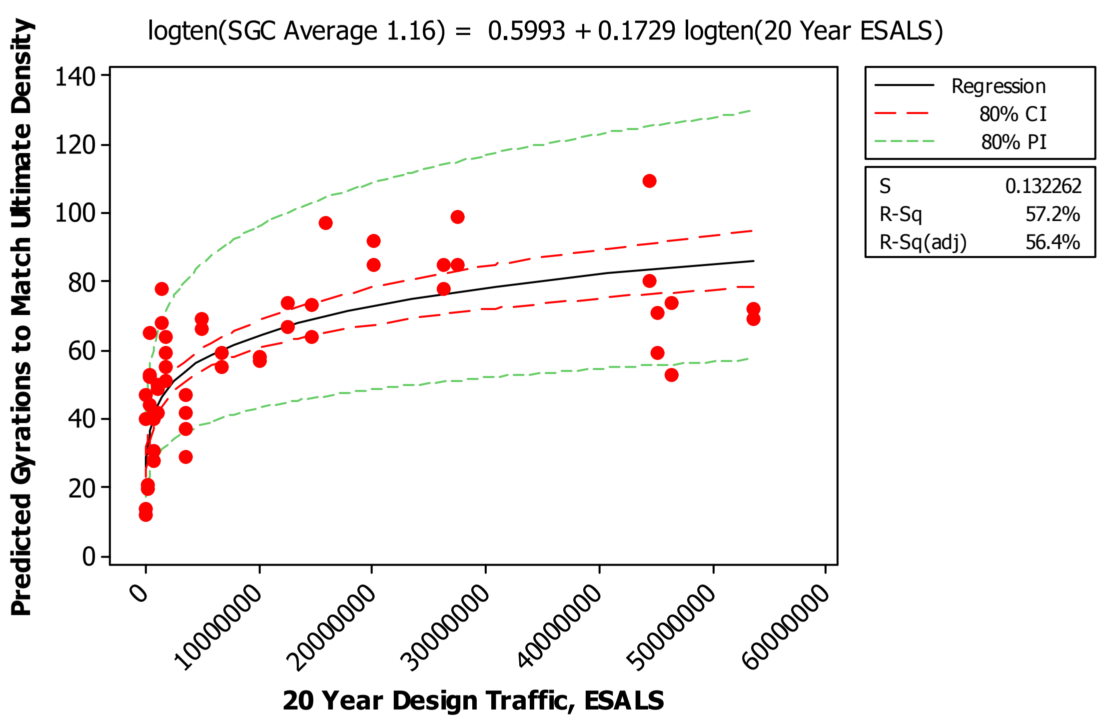

Figure 4.24. Predicted Gyrations versus 20 Year Design Traffic without PG 76-22 Data.

TABLE 4.10 Predicted Gyrations to Match Ultimate Density

20 Year Design ESAL	Current Ndesign	Predicted Ndesign	80 % Prediction Interval		80th Percentile		
			Low	High	Pine	Troxler	Avg.
300,000	50	35	23	53	32	43	37
1,000,000	75	43	29	65	71	73	72
3,000,000	100	52	35	78	83	90	87
10,000,000	100	65	43	96	59	55	57
30,000,000	125	78	52	117	95	104	100
100,000,000	125	96	64	145	101	82	92

Figure 4.25 shows the relationship between 20-year design ESALs and the predicted gyrations to match the 2-year density for the projects constructed with PG 76-22. Although a best fit line is shown in the figure, there is no relationship between the 20-year design ESALs and the predicted gyrations for the projects constructed with PG 76-22. A poor relationship (R^2 = 0.18) was also observed for the data from the 2000 NCAT Test Track (Figure 4.21). This indicates that for the modified binders there was

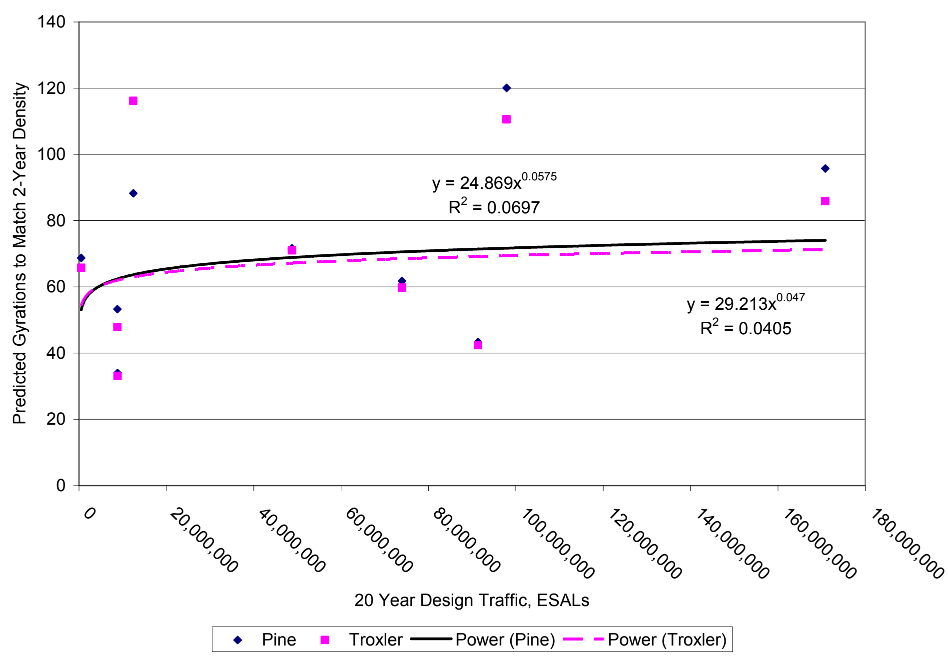

Figure 4.25. Predicted Gyrations for Projects with PG76-22.

no correlation between change in density and traffic.

When the Ndesign table was originally developed, regression analysis was performed between gyrations determined to match the as-constructed density and the gyrations required to match the in-place pavement density after more than 12 years of traffic (*64*). The analyses for the NCHRP 9-9 (1) field section presented thus far have been based solely on the number of gyrations to match the ultimate pavement density (2-year or 4-year). Figures 4.20 and 4.21 presented the predicted gyrations to match in-place density for the 2000 NCAT Test Track as traffic accumulated. Figure 4.26 presents a log-log plot of predicted gyrations versus accumulated traffic for all of the NCHRP 9-9 (1) field sections. The gyratory data corrected to a DIA of 1.16 degrees were used for the predictions. As expected, there is considerable scatter in the data as evidenced by the low

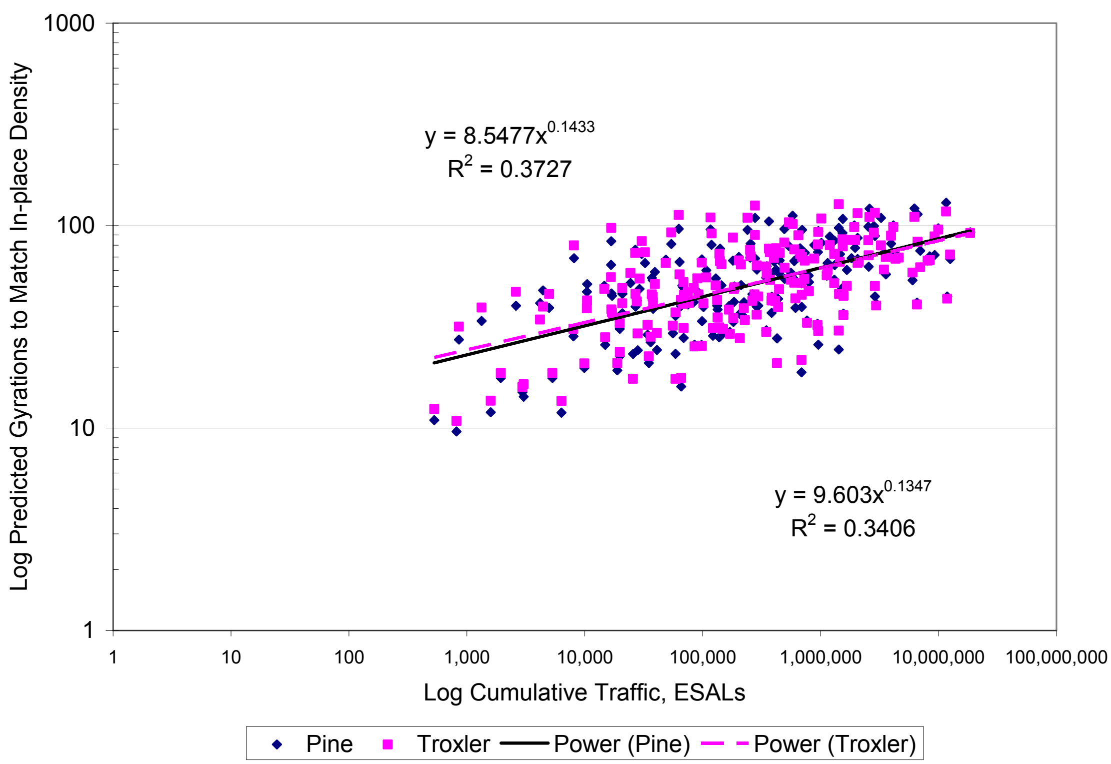

Figure 4.26. Predicted Gyrations to Match In-Place Density for all Post-Construction Sampling Periods.

R^2 values. The regression line for the Pine and Troxler data are approximately identical. The equations for the best fit line were used to predict gyration levels similar to Table 4.10 and are presented in Table 4.11. This method of analysis produces slightly higher predicted gyration levels, close to those currently specified.

TABLE 4.11 Predicted Gyrations to Match In-Place Density

20-Year Design ESAL	Troxler	Pine
300,000	52	52
1,000,000	62	62
3,000,000	73	74
10,000,000	86	89
30,000,000	101	105
100,000,000	120	126

One concern about the predictions is the high percentage of projects with low as-constructed density. Figure 4.6 indicates that 55 percent of the projects had as-constructed in-place densities less than 92 percent. Examination of the data suggested that there was a strong trend between as-constructed density and ultimate (2-year) density as shown in Figure 4.27 for both the field projects and the 2000 NCAT Test Track. Regression analyses between the as-constructed and two-year in-place densities indicated $R^2 = 0.65$ and $R^2 = 0.54$ for the field projects and 2000 NCAT Test Track, respectively. The shift in the regression lines between the field projects and the 2000 NCAT Test Track is somewhat expected based on the accelerated traffic loading at the 2000 NCAT Test Track. Since a higher as-constructed density would result in a higher ultimate density, this could affect the predicted Ndesign levels. Therefore, an attempt was made to model pavement densification to predict in-place density. It was felt that this model could possibly be used to predict Ndesign with ideal field conditions, e.g. 92 or 93 percent as-constructed density and 96 percent ultimate density.

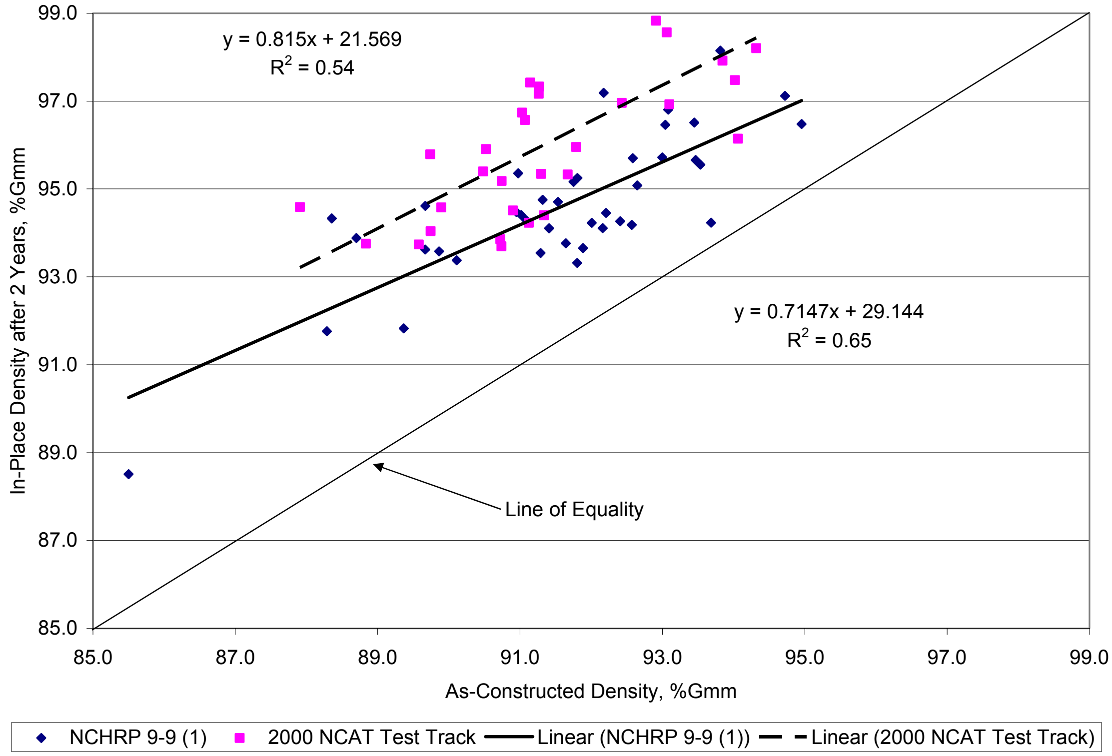

Figure 4.27. Relationship between As-Constructed and 2-Year (Ultimate) Density.

Epps et al. (*58*) described factors expected to affect pavement densification. Previously, high temperature PG bump and month of construction were shown to be significant factors which affect pavement densification. Brown and Cross (*60*) suggested that the Log of accumulated ESALs divided by the Log of the design compaction effort (in this case gyrations) was a good predictor for in-place density. Based on the literature, it was suggested that pavements constructed to a low initial density would tend to densify more and eventually obtain the same ultimate density as pavements constructed to higher initial densities. Figure 4.28 indicates that there is a weak trend of increased densification for projects with lower as-constructed densities, but no trend for projects with acceptable construction densities. Therefore, the difference between the laboratory density at Ndesign and the as-constructed density was considered as an alternative.

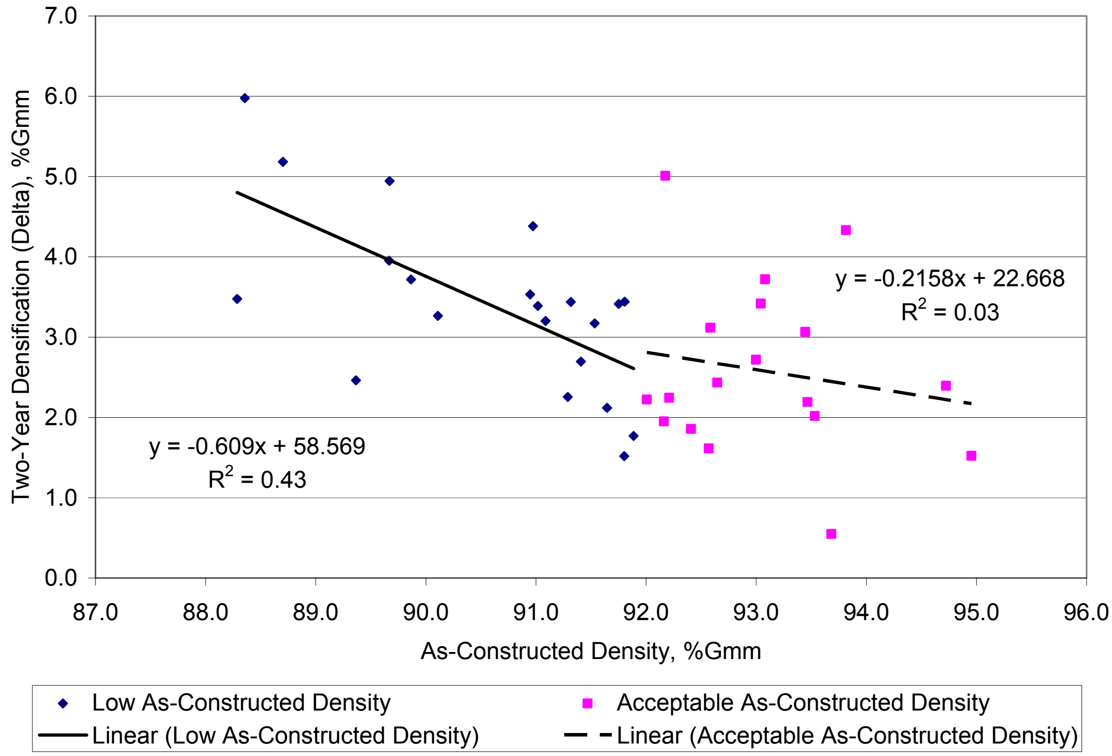

Figure 4.28. Two-Year Densification versus As-Constructed Density.

A number of techniques, such as best subsets and step-wise regression, and a number of iterations were attempted to develop a model to predict the 2-year pavement density. Variables used to predict 2-year density included: Degree days over 30°C, mean average annual air temperature, NMAS, high PG grade, agency specified design gyrations, month of construction, 2-year ESALs, and as-constructed density. Initially, an attempt was made to model pavement densification, but not even a fair model could be found. Better results were obtained when predicting pavement density. One of the best models developed is Equation 9:

$$2\ Year\ Den. = 0.771 \times Const.\ Den. - 0.325 \times Month\ of\ Const. - 0.078 \times High\ PG \qquad (9)$$

Month of construction was entered as the numerical month of construction, e.g. July = 7. High PG is the high PG binder grade, e.g. 64, 67, 70, or 76. The model has an $R^2 = 0.71$

with a *standard error* = 0.91 and a Mallow's C-p statistic of 5.6. All of the variables in the model are significant ($\alpha = 0.05$). It is generally desirable to have a Mallow's C-p statistic less than the number of variables in the model. This model only represents a slight improvement over the prediction made with just as-constructed density (Figure 4.27, $R^2 = 0.65$).

Minitab's best subsets analysis identified a five variable model with a Mallow's C-p statistic of 4.5. In addition to as-constructed density, month of construction and high temperature PG grade, this model included degree days over 30° C and Log of 2-Year ESALs. Degree days over 30° C was determined for each project from LTPPBind version 2.1 (*77*). If on a given day the temperature were 35° C, that day would account for five degree days. The reported value is the average yearly cumulative degree days. The data set contained projects with from 0 to 444 degree days over 30° C. Regions in the southwestern U.S. have much higher values for degree days over 30° C. For example, Phoenix, AZ has approximately 1400 degree days over 30° C. Equation 10 presents the second model developed for predicting 2-year (ultimate) density:

$$\begin{aligned} 2Y\ Density &= 30.61 + 0.786 \times ACD - 0.132 \times High\ PG - 0.204 \times MC \\ &+ 0.0041 \times 30CDD + 0.321 \times Log 2Y\ ESALS \end{aligned} \tag{10}$$

where,

2Y Density = in-place density after 2-years of traffic,

ACD = as-constructed density,

High PG = high temperature PG grade,

MC = month of construction (July = 7),

30CDD = degree days over 30° C, and

2Y ESALs = accumulated ESALs at 2 years.

Equation 10 has an R^2 0.76 and a standard error of 0.88. Degree days is not significant at the 5 percent level but is significant at the 10 percent level. The p-value for Log 2-year ESALs is 0.182, indicating that it is not significant. The fact that accumulated traffic is not strongly related to densification is not completely surprising since the projects were designed with a tiered system where projects with higher traffic levels tended to have more angular aggregates, stiffer binders and higher design gyration levels.

The models were then used to recalculate the 2-year density for each project assuming that the as-constructed density was 92 percent (the actual values were used for all of the other variables). The number of gyrations to match the new 2-year density (based on a 92 percent as-constructed density) was calculated for each project. Unfortunately, the resulting predicted gyrations produced even poorer relationships with design traffic than those presented previously. This tends to indicate that the scatter in the predicted gyration versus ESAL data was not due to the range of as-constructed densities.

Another source for the scatter in the predicted gyration versus ESAL data might be the fact that the HMA for the different projects were not all produced at 4 percent air voids. A project constructed with higher laboratory air voids would be less likely to densify in the field and a project constructed with low laboratory air voids would be more likely to densify in the field. One way to address this issue would be to look at the field densities as a percent of laboratory density. A model was developed to predict Ndesign as a function of high temperature PG grade and Design ESALs. As-constructed density was normalized to 92 percent Gmm in the model development. The following steps summarize the model development:

1. Express the 2-year in-place density for each project as a percent of Gmb (laboratory density) determined at 100 gyrations for both the Pine and Troxler SGCs normalized to a DIA of 1.16 degrees.
2. Develop a model to predict the 2-year percent of laboratory density similar to Equations 9 and 10. Models were developed to predict laboratory density (% Gmb) as a function of as-constructed density, high temperature PG and ESALs.
3. Develop a matrix of twelve 2-year in-place densities based on as-constructed densities of 92 percent, two high temperature PG grades (64 and 76) and a range of design traffic (Table 4.12).
4. Determine the in-place density (%Gmm) corresponding to each of the predicted laboratory densities (%Gmb) in Table 4.12 for each project.
5. Determine the number of gyrations needed to match each of the in-place densities determined in Step 4. The range of gyrations for each percent of laboratory density determined in Step 3 is relatively small. Essentially this says that the SGC compacted all of the mixes in this study at approximately the same rate. This makes sense sine the SGC is a constant strain compaction device. The average number of gyrations to match each of the percent of laboratory density in Table 4.12 was determined for both the Pine and Troxler SGCs.

6. Finally, a model was developed to relate Ndesign back to high temperature PG grade and Log ESALs. As-constructed density dropped out of the model since it was set to 92 percent Gmm in all cases. This was accomplished through the percent of laboratory (Gmb) density described in Steps 1-5.

The 2-year in-place density expressed as a percent of the laboratory density determined at 100 gyrations was regressed against the same sent of predictors used previously (Step 2). Equations 11 and 12 present the models developed for the Pine and Troxler compactors, respectively:

$$2\,Year\,\%\,Pine\,Lab\,Density = 53.95 + 0.452 \times ACD - 0.58 \times HPG + 1.19 \times Log\,2Y\,ESALs \quad (11)$$

$$2\,Year\,\%\,Troxler\,Lab\,Density = 62.34 + 0.381 \times ACD - 0.08 \times HPG + 1.06 \times Log\,2Y\,ESALs \quad (12)$$

where,

ACD = as-constructed density,

High PG = high temperature PG grade, and

2Y ESALs = accumulated ESALs at 2 years.

The $R^2 = 0.53$ for the Pine model and $R^2 = 0.45$ for the Troxler model with standard errors of 1.27 and 1.28, respectively. The high PG grade was not significant in either model, with p-values of 0.235 and 0.129 for the Pine and Troxler data, respectively. These variables were selected since the produced reasonable R^2 values for both compactors. Better models were identified for one or the other compactor, but they did not share the same variables.

A matrix of variables was developed to examine the effect of determining the predicted gyrations to match a given percentage of laboratory density (Step 3). Table

4.12 presents the matrix of variables and the resulting percentages of laboratory density. The in-place density corresponding to each of the percentages of laboratory density shown in Table 4.12 was calculated for each project (Step 4). Then the number of gyrations to match that in-place density was calculated for each project (Step 5). The predicted gyrations to match each of the percentages of laboratory density are shown in Tables 4.13 and 4.14 for the Pine and Troxler compactors, respectively. For the Pine compactor, the predicted gyrations for a given percentage of laboratory density had a low variability with standard deviations ranging from 3.44 to 8.99. The predicted gyrations to match a given percentage of laboratory density for the Troxler compactor also had low variability with standard deviations ranging from 4.83 to 8.98. Thus regardless of the mix, a given percentage of laboratory density (determined at an Ndesign of 100 gyrations) can be achieved with a similar number of gyrations.

TABLE 4.12 Matrix of Predicted Percentage of Laboratory Density

As-Constructed Density	2-Year ESALs	Log 2 Year ESALS	Approximate 20-Year ESALs	High PG	Pine Predicted 2-Year %Gmb (Lab Density)	Troxler Predicted 2-Year % Gmb (Lab Density)
92	30,000	4.48	300,000	64	97.2	97.3
92	90,000	4.95	1,000,000	64	97.8	97.8
92	230,501	5.36	3,000,000	64	98.3	98.2
92	920,577	5.96	10,000,000	64	99.0	98.9
92	2,583,607	6.41	30,000,000	64	99.5	99.3
92	6,773,140	6.83	100,000,000	64	100.0	99.8
92	30,000	4.48	300,000	76	96.5	96.4
92	90,000	4.95	1,000,000	76	97.1	96.9
92	230,501	5.36	3,000,000	76	97.6	97.3
92	920,577	5.96	10,000,000	76	98.3	98.0
92	2,583,607	6.41	30,000,000	76	98.8	98.4
92	6,773,140	6.83	100,000,000	76	99.3	98.9

TABLE 4.13 Pine Predicted Gyration to Match Percentage of Lab Density

Project	Percent of Lab Density, %Gmb											
	97.2	97.8	98.4	99.0	99.6	100.2	96.5	97.1	97.7	98.3	98.9	99.5
	Predicted Gyrations											
AL-1	50	58	68	79	91	107	42	49	57	66	77	90
AL-2	55	64	72	83	94	108	47	55	62	71	81	93
AL-3	42	51	62	75	91	111	33	41	49	60	72	89
AL-4	34	43	54	69	86	109	26	33	41	53	66	83
AL-5	33	42	53	67	84	108	25	32	40	51	64	82
FL-1	43	52	61	73	86	104	35	42	50	60	70	84
MI-1	52	60	69	80	91	105	44	51	59	68	77	89
MI-2	48	57	66	78	91	107	40	47	55	65	75	89
WI-1	44	53	63	76	90	108	36	43	51	62	73	88
CO-1	38	47	57	71	86	107	30	37	45	56	68	84
CO-2	38	47	57	71	86	106	30	37	45	56	68	84
CO-3	44	53	62	74	87	103	36	43	51	61	71	85
CO-4	47	55	65	77	90	107	38	46	53	63	74	88
CO-5	46	55	64	76	89	106	38	45	53	63	73	87
IN-1	54	62	71	81	92	106	47	53	61	70	79	91
IN-2	40	49	58	71	85	103	32	39	47	57	68	83
KY-1	58	66	75	85	96	109	50	57	64	73	83	94
KY-2	58	66	74	84	94	107	50	57	64	73	82	93
KY-3	42	51	61	74	89	108	34	41	49	60	71	87
AL-6	33	42	54	70	89	115	24	32	40	52	66	86
AR-1	52	61	71	83	96	113	43	51	59	69	80	94
AR-2	52	61	71	83	96	112	44	51	59	69	80	94
AR-3	47	56	67	80	95	114	38	46	54	65	77	93
AR-4	42	50	60	72	85	102	34	41	48	58	69	83
GA-1	34	44	55	71	89	115	26	33	41	53	67	87

TABLE 4.13 Pine Predicted Gyration to Match Percentage of Lab Density (Continued)

Project	Percent of Lab Density, %Gmb											
	97.2	97.8	98.4	99.0	99.6	100.2	96.5	97.1	97.7	98.3	98.9	99.5
	Predicted Gyrations											
IL-1	56	64	72	82	93	107	48	55	62	71	80	92
IL-2	56	64	73	84	96	111	47	55	62	72	82	94
IL-3	54	62	71	82	93	107	46	53	60	69	79	91
KS-1	43	52	62	75	89	107	35	42	50	60	72	87
MI-3	40	49	59	72	87	107	32	39	47	57	69	85
MO-1	58	67	76	87	98	113	50	57	65	74	84	97
MO-2	54	63	71	82	93	107	47	54	61	70	79	91
MO-3	55	64	73	84	96	110	47	55	62	72	81	94
NC-1	29	39	51	68	88	117	21	28	37	49	64	85
NE-1	30	39	50	65	84	110	22	29	37	48	62	81
NE-2	36	45	56	70	86	108	28	35	43	54	67	83
NE-3	27	36	46	61	78	103	20	26	34	45	57	76
NE-4	37	46	56	70	86	107	29	36	44	55	67	83
TN-1	36	46	57	71	88	111	28	36	44	55	68	86
UT-1	47	56	66	78	91	108	39	46	54	64	75	89
Minimum	27.1	35.7	46.0	60.6	78.0	102.4	19.9	26.2	33.8	44.5	57.3	75.5
Average	44.6	53.4	63.1	75.8	89.8	108.3	36.5	43.7	51.5	61.7	73.0	87.8
Maximum	58.0	66.5	75.5	86.7	98.4	116.6	50.2	57.2	64.7	74.3	84.3	96.8
Std. Dev.	8.97	8.61	7.82	6.30	4.33	3.44	8.95	8.99	8.72	7.95	6.67	4.60

TABLE 4.14 Troxler Predicted Gyration to Match Percentage of Lab Density

Project	Percent of Lab Density, %Gmb											
	97.2	97.8	98.4	99.0	99.6	100.2	96.5	97.1	97.7	98.3	98.9	99.5
	Predicted Gyrations											
AL-1	52	60	68	78	89	102	41	48	54	62	71	81
AL-2	58	66	73	83	92	104	48	54	60	68	76	85
AL-3	43	52	60	72	84	99	33	39	46	54	63	75
AL-4	36	44	54	67	81	101	25	31	38	47	57	71
AL-5	35	42	51	62	75	91	25	31	37	45	54	66
FL-1	46	54	63	75	87	103	35	41	48	57	66	78
MI-1	53	61	69	79	89	102	43	49	55	63	71	82
MI-2	49	57	65	76	88	103	38	44	51	59	68	80
WI-1	45	54	63	74	87	102	34	41	48	56	66	78
CO-1	42	51	60	72	85	101	32	38	45	53	63	75
CO-2	44	53	62	75	88	106	32	39	46	55	66	79
CO-3	46	54	63	73	84	99	36	42	49	57	65	77
CO-4	48	56	65	76	87	102	37	44	50	59	68	79
CO-5	47	55	63	74	86	100	36	42	49	57	66	78
IN-1	54	62	69	79	89	101	44	50	56	64	72	82
IN-2	41	50	59	71	83	100	31	37	44	52	62	74
KY-1	58	66	73	83	92	104	48	54	60	68	76	85
KY-2	59	66	74	84	93	105	48	55	61	69	77	87
KY-3	42	51	60	71	84	101	32	38	45	53	63	75
AL-6	34	42	52	65	81	101	23	29	36	45	56	70
AR-1	54	63	71	82	93	106	43	50	57	65	74	85
AR-2	56	64	72	83	94	108	44	51	58	66	75	86
AR-3	56	65	74	86	98	113	45	51	59	68	77	89
AR-4	50	59	69	81	95	112	38	45	52	62	72	85
GA-1	36	45	55	68	83	103	26	32	39	48	58	73

TABLE 4.14 Troxler Predicted Gyration to Match Percentage of Lab Density (Continued)

Project	Percent of Lab Density, %Gmb											
	97.2	97.8	98.4	99.0	99.6	100.2	96.5	97.1	97.7	98.3	98.9	99.5
	Predicted Gyrations											
IL-1	56	63	71	80	89	101	46	52	58	65	73	83
IL-2	59	66	74	84	94	107	48	54	61	69	77	87
IL-3	57	64	72	82	92	104	46	52	59	67	75	85
KS-1	46	55	64	76	88	104	35	42	49	57	67	79
MI-3	41	49	58	70	83	100	30	36	43	51	61	74
MO-1	60	68	76	86	96	109	49	56	62	71	79	89
MO-2	57	65	72	81	91	103	47	53	59	67	75	84
MO-3	58	65	73	83	93	105	47	53	60	67	76	86
NC-1	30	38	46	57	69	85	22	27	32	40	49	60
NE-1	33	41	51	64	79	99	22	28	35	44	54	68
NE-2	39	47	57	70	84	102	28	34	41	50	60	74
NE-3	30	38	47	60	75	95	20	25	32	40	50	64
NE-4	40	48	57	68	81	98	29	35	42	50	60	72
TN-1	37	46	56	68	83	102	27	33	40	49	59	73
UT-1	48	56	65	75	86	100	38	44	50	59	67	78
Minimum	29.5	37.6	45.7	56.6	68.7	85.0	19.9	25.3	31.6	40.0	48.6	60.1
Average	46.9	55.0	63.6	74.7	86.7	102.1	36.3	42.4	49.0	57.5	66.6	78.3
Maximum	60.3	68.2	76.2	86.2	97.7	112.9	49.3	55.8	62.4	70.6	79.0	89.3
Std. Dev.	8.98	8.78	8.30	7.39	6.19	4.83	8.81	8.98	8.96	8.66	8.08	7.05

Since the gyrations were related to the percentage of laboratory density at 100 gyrations, and since the percentage of laboratory density was related to as-constructed density, high PG grade and ESALs, the data were analyzed to see if a relationship existed between the average predicted gyration and high PG grade and ESALs (Step 6). Since a single target as-constructed density was desired (92 percent), this variable should drop out of the relationship. Higher as-constructed densities would (using Equations 11 or 12) result in higher predicted gyrations. Although this seems counter intuitive from a field compaction standpoint, if a mix was constructed to a higher level of density initially, one would want it to be more resistant to additional densification. Likewise a pavement constructed to a lower as-constructed density would tend to age faster, producing a stiffer mix. Therefore, one would need a mix that would densify more readily to achieve the same ultimate density.

Table 4.15 shows the data used to develop the models to predict Ndesign gyration levels from high PG grade and 2-year ESALs. The average gyrations to match a percentage of laboratory density are those shown in Tables 4.13 and 4.14 to meet the percentage of lab density determined for the matrix in Table 4.12. Since the Pine and Troxler number of gyrations to match a percentage of laboratory density at a DIA of 1.16 degrees were so close to each other, they were averaged. Two models were then developed between ESALs, High PG grade and gyrations, one using the 2-year ESALs (Equation 13) and one using the 20-year ESALs (Equation 14). Equation 14 was determined following the same steps as Equation 13 using the 20-year ESALs.

TABLE 4.15 Matrix of Gyrations

2-Year ESALs	20-Year ESALs	High PG	Avg. Pine Gyrations to a Percentage of Lab Density	Avg. Troxler Gyrations to a Percentage of Lab Density	Average Gyrations to a Percentage of Lab Density	Predicted Gyrations to a Percentage of Lab Density	
						Eq.13	Eq.14
30,000	300,000	64	45	47	46	46	46
90,000	1,000,000	64	53	55	54	56	56
230,501	3,000,000	64	63	64	63	64	66
920,577	10,000,000	64	76	75	75	77	76
2,583,607	30,000,000	64	90	87	88	86	86
6,773,140	100,000,000	64	108	102	105	95	96
30,000	300,000	76	37	36	36	31	30
90,000	1,000,000	76	44	42	43	41	41
230,501	3,000,000	76	51	49	50	49	50
920,577	10,000,000	76	62	58	60	62	61
2,583,607	30,000,000	76	73	67	70	71	71
6,773,140	100,000,000	76	88	78	83	80	81

$$Ndesign = 33.0 - 1.25 \times HPG + 20.9 \times Log\ 2\ Year\ ESALs \qquad (13)$$

$$Ndesign = 16.8 - 1.27 \times HPG + 20.1 \times Log\ 20\ Year\ ESALs \qquad (14)$$

where,

Ndesign = the number of design gyrations,

HPG = high PG grade, and

2-Year or 20-Year ESALs = the 2-year or 20-year design ESALs for the project.

The R^2 for both Equation 13 and Equation 14 is 0.97 with *standard errors* of 3.66 and 3.54, respectively. Note that the model reduces Ndesign by approximately 15 gyrations for a two grade bump in high PG grade (e.g. 64 to 72). The lowest traffic level is equivalent to the 50 gyrations currently specified in AASHTO R 35 for less than 300,000 ESALs. The predicted Ndesign for unmodified binders for the highest traffic level is approximately 25 gyrations less than currently specified in AASHTO R 35 (125 gyrations). Further the predicted gyrations for the unmodified binder (PG 64) approximately match those determined in Table 4.10 (presented previously), but are slightly higher in the 10 to 30 million 20-year ESAL range.

4.2.5 Evaluation of Locking Point

The locking point concept was developed by Illinois DOT (*75, 76*). Since its development, other agencies have altered the definition of the locking point. The original definition is the first instance of three consecutive gyrations having the same sample height immediately preceded by two instances of two consecutive gyrations resulting in the same sample height (locking point 3-2-2). Other values used include: first instance of two consecutive gyrations resulting in the same sample height (locking point 2-1), second instance of two consecutive gyrations resulting in the same sample height (locking point 2-2), the third instance of two consecutive gyrations resulting in the same sample height (locking point 2-3) and One criticism of the locking point was that there was little research to tie the results to a physical quantity in the field.

The locking point was determined manually for each of the cases described above. One encouraging aspect of the locking point calculations was that the locking point was approximately the same number of gyrations for the Pine and Troxler SGCs without any adjustments (Figure 4.29). However, the density at a given definition of the locking point was higher for the Pine compactor (Figure 4.30), if the data are not corrected to a DIA of 1.16 degrees. Comparisons were made between the calculated density at the four different definitions of the locking point and as-constructed and two-year in-place density. The 2-1 locking point overestimated the as-constructed density as seen in Figure 4.31. The 3-2-2 locking point appears to provide the best relationship with ultimate density (Figure 4.32). However, the relationship is poor, weaker than that determined using design traffic. Various subdivisions of unmodified and modified binder were attempted, since binder stiffness should not affect the results during compaction. The best relationship ($R^2 = 0.47$) was determined for the projects with modified binders based on the Troxler densities for the 3-2-2 locking point. However, 3 of the 20 projects, AR-3, AR-4 and IL-2, had missing data which prevented their inclusion.

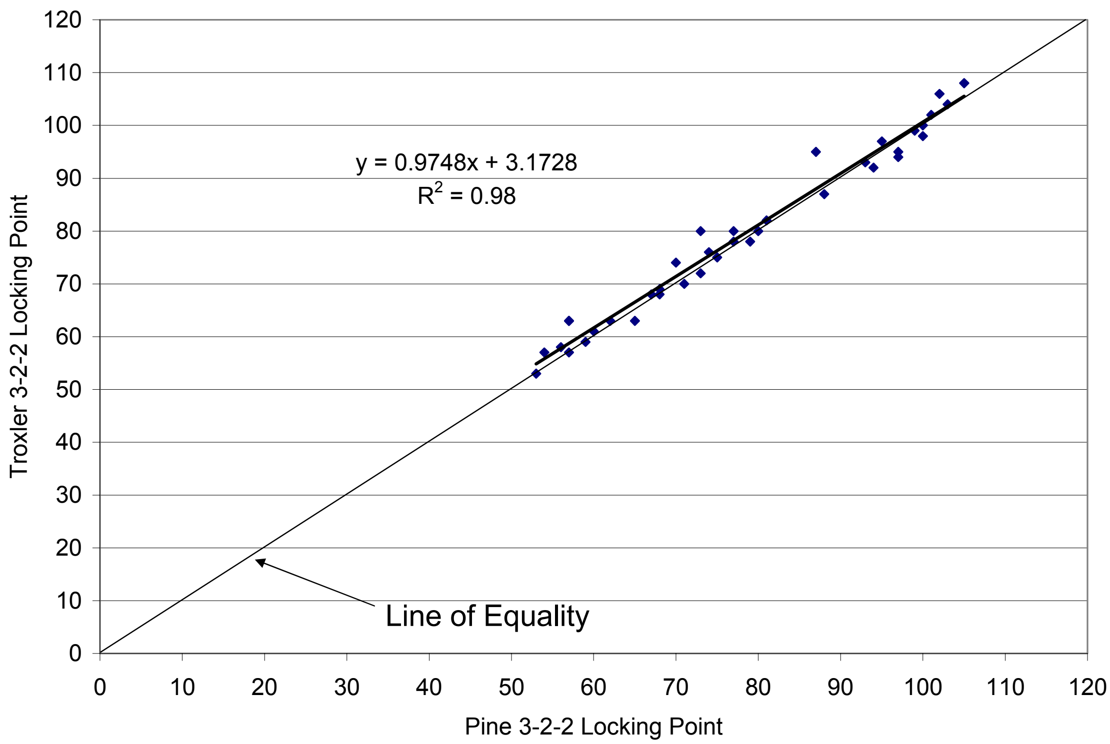

Figure 4.29. Comparison between 3-2-2 Pine and Troxler Locking Point.

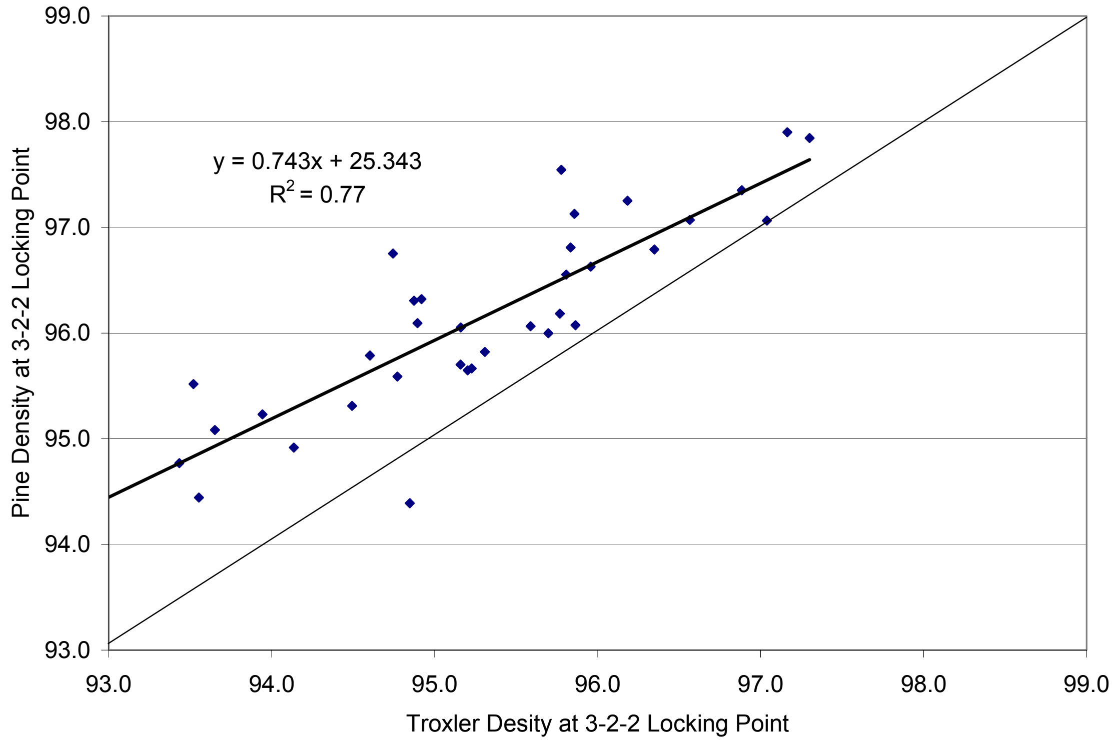

Figure 4.30. Comparison of Average Pine and Troxler Density at 3-2-2 Locking Point.

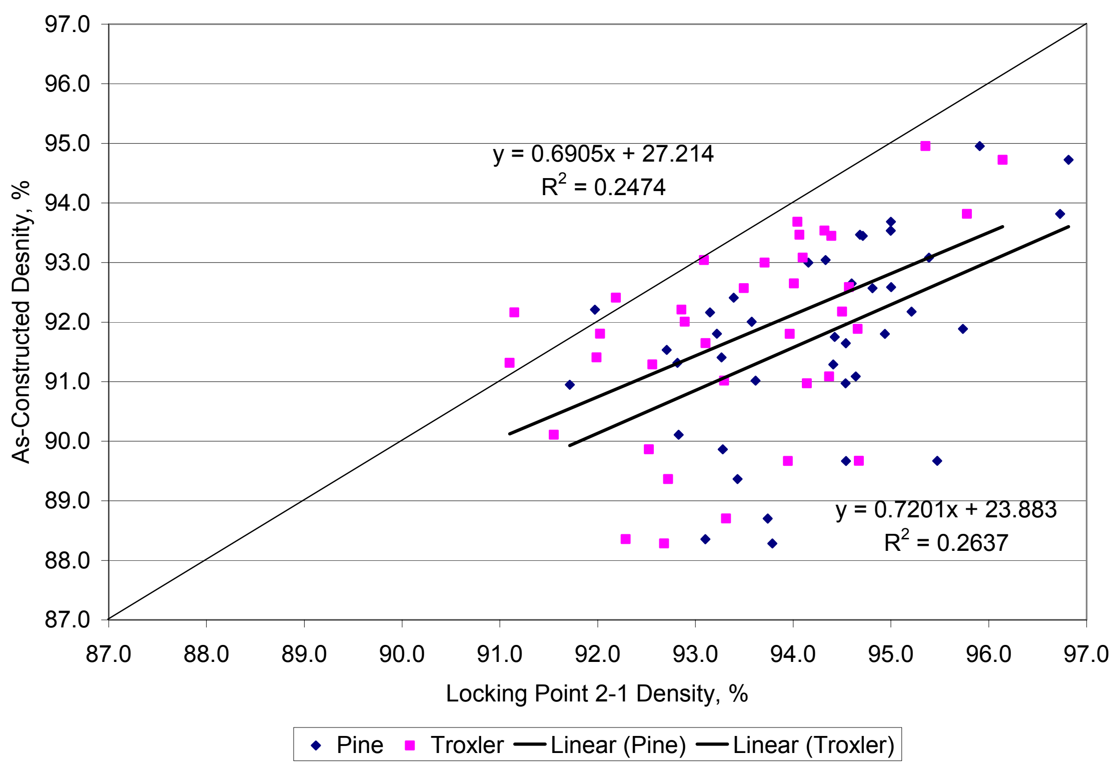

Figure 4.31. 2-1 Locking Point Density versus As-Constructed Density.

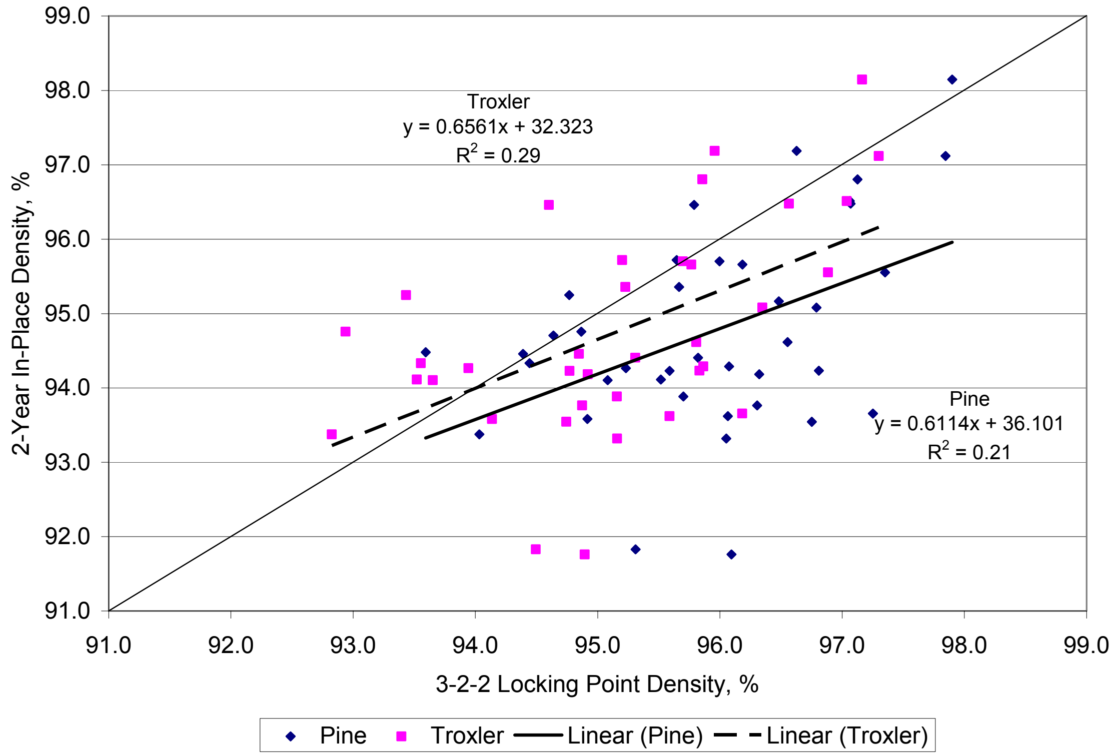

Figure 4.32. 3-2-2 Locking Point Density versus 2-Year Density.

The use of the 3-2-2 locking point would appear to be a conservative way to estimate the ultimate density of the pavement. One potential concern about the use of the locking point is the lubricating effect of binder content on the number of gyrations determined for the locking point. If the asphalt content selected for the locking point determination is on the dry portion of the VMA curve, then the locking point may be higher, whereas if it is on the wet side it may be lower than or close to the locking point at the optimum asphalt content. An evaluation of the locking point over a range of binder contents is beyond the scope of this study. Also, the locking point appears to be a function of the aggregate type, angularity and gradation and is not related to the design traffic.

4.2.6 Pavement Condition after Four Years

Visual assessments were conducted along with the pavement coring at each coring interval. Rut depths were measured with a six-foot string line. Table 4.16 presents the 4-year rut depth measurements. The maximum observed rutting averaged 6.4 mm. The average rutting observed for all of the projects was 1.7 mm. The Superpave mixes are all very rut resistant. Noticeable raveling was observed on 14 of the projects; 13 projects exhibited cracking; 13 projects had popouts; and 7 projects exhibited moisture damage in either the test layer or the underlying layer.

The rut depths from the field projects match the findings of the 2000 NCAT Test Track. Brown et al. (2004) reported an average rut depth after 10 million ESALs in two years of 2.7 mm with a maximum rut depth of 7.4 mm. The two sections with the most rutting, N3 (7.4 mm) and N5 (7.1 mm) were both placed with asphalt contents approximately 0.5 percent above optimum. Brown et al. also noted that sections containing PG 76-22 rutted 60 percent less than sections constructed with unmodified PG 67-22. It should be noted that the majority of the observed "rutting" was attributed to pavement densification under traffic.

TABLE 4.16 Four-Year Rut Depth Measurements

Project	Sublot 1			Sublot 2			Sublot 3			Avg., mm	Std.Dev., mm
	Core Location										
	1	2	3	1	2	3	1	2	3		
AL-1	2	2	2	2	1	3	-	-	-	2.0	0.83
AL-2	3	2	2	0	0	2	5	5	6	2.7	2.03
AL-3											
AL-4											
AL-5											
FL-1											
MI-1	10	9	9	6	7	7	3	2	4	6.4	2.60
MI-2	2	2	2	1	0	2	-	-	-	1.3	0.82
WI-1	0	0	0	0	0	0	0	0	0	0.0	0.00
CO-1	3	2	4	5	5	3	7	6	7	4.8	1.77
CO-2	1	2	2	2	3	3	3	3	3	2.6	0.79
CO-3	0	0	0	0	0	0	-	-	-	0.0	0.00
CO-4	2	1	1	2	2	1	2	2	2	1.5	0.62
CO-5	3	2	2	5	5	5	4	3	3	3.6	0.98
IN-1	2	3	2	0	0	2	3	5	2	2.2	1.53
IN-2	3	2	2	5	3	4	3	2	2	3.0	0.95
KY-1	0	0	0	0	0	0	-	-	-	0.0	0.00
KY-2	1	2	0	0	0	0	-	-	-	0.4	0.66
KY-3	1	0	0	0	0	1	0	0	0	0.2	0.37
AL-6	0	0	0	0	0	0	-	-	-	0.0	0.00
AR-1	2	2	2	2	2	2	2	2	2	1.9	0.40
AR-2	3	2	3	3	3	2	-	-	-	2.8	0.66
AR-3	3	3	2	2	1	3	-	-	-	2.2	1.06
AR-4	2	2	2	2	2	2	2	2	2	2.1	0.40
GA-1	1	1	1	1	1	0	2	1	0	0.7	0.48
IL-1	0	1	0	1	1	1	0	0	1	0.4	0.42
IL-2	1	3	2	3	3	3	3	2	2	2.6	0.79
IL-3	0	0	0	0	0	1	1	2	2	0.5	0.69
KS-1	1	0	1	1	2	2	2	1	1	1.0	0.53
MI-3	0	0	0	0	0	0	0	0	0	0.0	0.00
MO-1	1	2	1	2	1	3	4	1	2	1.9	1.19
MO-2	2	2	2	1	1	2	3	2	2	1.7	0.74
MO-3	0	0	0	1	0	0	0	0	0	0.1	0.26
NC-1	6	2	2	1	2	2	1	2	1	2.0	1.73
NE-1	2	5	4	2	2	2	-	-	-	2.5	1.46
NE-2	1	1	1	2	2	2	2	2	2	1.5	0.62
NE-3	2	2	2	2	2	2	2	2	2	2.2	0.35
NE-4	1	1	2	2	2	4	2	2		1.8	1.02
TN-1	2	2	2	3	2	3	2	2	1	2.0	0.80
UT-1	0	0	0	0	0	0	0	0	0	0.0	0.00

4.2.7 Evaluation of Ninitial

The densities at Ninitial, corrected to a DIA of 1.16 degrees, are shown in Table 4.17. Table 4.17 is sorted by 20-year traffic. AASHTO M 323-04 specifies that the density at Ninitial shall be less than 91.5 percent for 20-year traffic levels less than 300,000 ESALs, less than 90.5 percent for traffic levels between 300,000 and 3,000,000 ESALs, and less than 89.0 percent for traffic levels greater than 3,000,000 ESALs. Based on Table 4.12, none of the samples from projects with design traffic less than 300,000 ESALs fail Ninitial, 36 percent of the samples with design traffic levels between 300,000 and 3,000,000 ESALs fail Ninitial, and 26 percent of the samples with design traffic levels greater than 3,000,000 ESALs fail Ninitial. Failures occur in 11 of the 40 projects. The mixes are fine-graded for 9 of the 11 projects that fail Ninitial. Both of the coarse-grade projects, AL-3 and AL-5 had lower laboratory air voids at the agency specified Ndesign level. Both projects averaged 3.0 percent air voids. Project GA-1 also had low air voids at the agency specified Ndesign gyrations (1.9 percent).

The field notes taken at the time of construction only indicate tender mix problems for one project, NE-4. NE-4 does fail the Ninital requirements. However, construction issues were not commented on at all for many of the projects, so it is possible that there were tender mix problems on other projects. Historically, contractors have found ways to deal with tender mixes in the field.

When the Superpave system was first introduced, the Ninitial requirements worked in conjunction with the restricted zone requirements and the fine aggregate angularity requirements to limit the amount of natural sand, or rounded fine aggregate particles in HMA. The restricted zone requirement has been eliminated since it was

TABLE 4.17 Summary of Densities, %Gmm, at Ninitial

Project	20-Year ESALs	Gradation	Ninitial	Pine			Troxler		
				1	2	3	1	2	3
KY-1	53,706	C	6.0	85.3	84.9	-	85.1	84.5	-
KY-3	84,028	F	7.0	88.8	89.1	88.9	88.8	88.6	88.6
AL-6	143,958	F	8.0	90.8	91.0	-	90.6	91.0	-
NE-3	365,719	F	7.0	90.5	91.7	90.8	90.5	92.1	91.3
NE-1	383,385	F	7.0	90.4	91.9	-	90.7	91.9	-
CO-3	523,624	C	8.0	87.8	88.3	-	88.1	88.3	-
CO-4	720,911	F	7.0	88.4	88.9	87.3	88.3	88.4	87.6
CO-1	756,789	F	7.0	90.6	92.3	91.5	90.4	91.9	91.2
UT-1	771,982	F	7.0	87.9	88.8	88.9	87.7	88.5	88.8
FL-1	811,658	C	7.0	85.8	87.9	-	86.1	87.1	-
CO-2	1,017,593	F	7.0	91.9	90.8	90.9	91.8	91.0	90.8
CO-5	1,017,593	F	7.0	87.5	87.9	87.6	87.3	87.5	87.4
MI-2	1,250,146	F	7.0	87.8	88.4	87.9	87.9	88.1	87.9
NE-2	1,450,960	F	8.0	89.4	89.6	89.7	89.6	90.1	89.8
MI-3	1,515,200	F	7.0	88.8	89.0	-	88.5	88.8	-
AL-5	1,809,675	C	7.0	91.2	91.1	90.9	90.9	90.4	91.0
IN-1	1,850,992	C	8.0	84.3	85.8	85.8	84.3	85.2	85.2
TN-1	3,490,393	F	8.0	89.9	90.2	90.0	91.3	90.8	90.4
AL-2	3,610,001	C	8.0	85.5	84.3	83.9	84.9	83.7	83.4
AL-4	4,899,406	C	8.0	88.6	88.9	89.2	88.7	88.7	89.0
AL-1	6,748,142	C	8.0	86.9	85.9	86.0	87.1	86.1	86.2
GA-1	8,803,521	F	8.0	91.1	91.9	91.8	91.6	92.1	91.4
AL-3	8,861,352	C	8.0	88.9	89.1	-	88.8	89.1	-
KS-1	10,075,962	F	8.0	86.4	88.1	87.3	86.7	87.9	87.1
KY-2	12,438,605	C	8.0	81.3	84.8	-	80.9	84.4	-
MO-2	12,517,675	C	8.0	-	86.2	84.5	85.6	86.5	84.1
WI-1	14,614,748	C	8.0	87.0	87.5	87.6	86.4	87.5	87.6
MI-1	15,966,398	C	9.0	84.3	85.0	84.2	83.7	84.3	84.0
NE-4	20,084,248	F	8.0	90.1	90.6	90.0	89.7	90.6	90.2
IL-1	26,285,917	C	8.0	83.8	84.5	84.2	84.0	83.9	84.0
MO-1	27,546,007	C	9.0	84.6	85.9	86.1	86.0	86.4	85.7
IL-3	44,466,336	C	8.0	83.6	84.0	83.3	84.7	84.7	84.2
IN-2	45,150,555	C	9.0	88.7	88.5	87.1	88.1	88.4	86.9
IL-2	46,344,297	C	8.0	84.5	86.2	86.1	85.3	87.0	86.8
AR-1	48,726,562	C	9.0	85.0	86.8	86.1	85.0	86.5	86.0
MO-3	53,683,941	C	9.0	85.5	86.5	86.4	85.6	86.4	86.4
NC-1	73,918,507	F	9.0	-	89.3	89.2	89.2	87.7	88.8
AR-2	91,370,805	C	9.0	85.7	85.3	-	85.3	85.5	-
AR-4	97,890,077	C	9.0	85.5	86.3	86.2	85.7	85.9	86.3
AR-3	170,842,507	C	9.0	87.5	85.5	-	84.7	86.0	-

demonstrated that good performing mixes frequently passed through the restricted zone. Ninitial is sensitive to gradation and the presence of rounded fine aggregate particles.

4.2.8 Evaluation of Nmaximum

The densities at Nmaximum, corrected to a DIA of 1.16 degrees, are shown in Table 4.18. AASHTO M 323 specifies that the density at Nmaximum be less than 98 percent. At the agency specified Nmaximum, 36 percent of the Pine samples and 40 percent of the Troxler samples failed the Nmaximum density criteria. One or more samples exceeded the maximum density at Nmaximum for 25 of the 40 projects. When NCAT collected the field data, samples were compacted to both 100 and 160 gyrations. Therefore, sample densities for Nmaximum gyrations greater than 160 gyrations are extrapolated. Although there is a very good relationship between sample density and log of gyrations, at high gyration levels (above the mixtures locking point and Ndesign), this relationship tends to breakdown with additional gyrations producing little increase in sample density. The sample densities at Nmaximum are extrapolated above Nmaximum for 10 of the 25 projects which failed the density requirements at Nmaximum. These extrapolations may be erroneous. However, this still leaves 15 of 40 projects which failed Nmaximum. The maximum rutting for a sample that failed density at Nmaximum occurred for project MI-1, sublot 2, with an average rut depth of 7 mm after four years of traffic. Sublot 1 of MI-1 actually had a slightly higher average rut depth (9 mm) but the sample did not fail the Nmaximum density criteria. Further, as evidenced by Table 4.16, all of the mixes have been extremely rut resistant. Based on the data, the Nmaximum criteria should be eliminated.

TABLE 4.18 Summary of Densities, % Gmm, at Nmaximum

Project	Nmax	Pine			Troxler		
		1	2	3	1	2	3
AL-1	169	98.5	97.5	97.6	98.9	98.0	97.8
AL-2	160	97.7	97.0	97.2	98.3	96.3	96.4
AL-3	160	97.9	97.7	-	98.0	98.7	-
AL-4	160	95.5	96.2	97.0	95.3	96.0	97.1
AL-5	115	98.1	98.0	97.8	98.4	97.8	98.2
FL-1	134	95.6	97.0	-	95.0	96.5	-
MI-1	205	97.4	98.5	97.4	96.1	97.4	96.3
MI-2	115	97.9	98.4	98.0	97.3	97.9	97.5
WI-1	160	96.2	97.2	97.3	95.9	96.5	97.0
CO-1	104	98.8	99.9	98.8	98.5	100.0	99.0
CO-2	134	99.4	99.5	99.4	99.5	99.4	99.4
CO-3	174	98.8	98.9	-	98.7	99.2	-
CO-4	134	98.7	98.7	97.9	98.3	98.3	97.8
CO-5	134	97.5	97.9	97.8	96.8	97.3	97.1
IN-1	160	97.4	98.7	98.6	96.2	97.6	97.4
IN-2	205	98.2	98.8	97.2	97.5	97.9	96.5
KY-1	75	96.7	96.8	-	95.8	95.5	-
KY-2	160	94.9	98.4	-	93.5	97.2	-
KY-3	115	97.0	97.5	97.8	96.7	96.8	97.2
AL-6	150	97.2	97.8	-	97.4	98.0	-
AR-1	205	97.3	99.5	98.4	97.5	99.0	98.9
AR-2	205	97.5	98.4	-	97.8	98.5	-
AR-3	205	96.8	97.8	-	97.3	99.2	-
AR-4	205	95.8	96.5	96.3	95.8	96.5	96.9
GA-1	160	98.3	99.1	98.3	98.1	100.0	99.1
IL-1	140	96.5	97.4	97.3	96.3	96.7	96.8
IL-2	140	96.7	98.7	98.5	98.1	99.7	99.4
IL-3	165	95.7	96.6	96.3	97.1	97.6	97.7
KS-1	160	96.9	96.2	96.5	96.2	96.8	96.6
MI-3	115	97.0	97.0	-	96.6	96.5	-
MO-1	205	99.0	100.0	100.0	100.0	100.0	100.0
MO-2	160	-	98.5	97.6	98.6	99.3	97.8
MO-3	205	99.5	100.0	100.0	99.9	100.0	100.0
NC-1	205	-	96.5	95.9	96.5	96.3	96.9
NE-1	104	96.5	97.8	-	97.0	98.4	-
NE-2	152	97.0	97.6	97.4	97.3	97.9	97.7
NE-3	117	96.6	97.6	96.9	96.9	98.2	97.3
NE-4	174	98.5	98.9	98.4	99.0	99.4	98.5
TN-1	160	97.9	97.7	97.7	98.6	98.5	98.4
UT-1	115	97.9	99.0	98.7	97.4	98.7	98.6

4.2.9 Summary and Discussion of Test Results

The asphalt content of HMA mixture, as-constructed density and ultimate density are all critical to the performance of an HMA pavement. These values are all interrelated since mixes with higher asphalt contents, for a given aggregate structure, are generally easier to compact initially, and will tend to densify more under traffic. The determination of a HMA mixture's optimum asphalt content has changed significantly since the first asphalt pavements were introduced in the 1870's. Optimum asphalt contents were initially selected by experience. As the popularity of HMA grew, there were not enough experienced individuals to determine the optimum asphalt content for all of the HMA being placed. In the late 1930's and 1940's, asphalt technologists began to develop laboratory compaction methods with the goal of matching the ultimate pavement density. It had been observed that an HMA pavement densified under traffic from its as-constructed density to an ultimate density, typically within 2 to 3 years after construction. Initially, only one laboratory compaction level was used for a given system, but as tire pressures and traffic volumes grew, the concept of a tiered design system, illustrated in Figure 2.7 (*25*) was developed where laboratory compaction increased for increasing tire pressures or traffic volumes. The concept of a tiered laboratory compaction was to address the tendency for increased tire pressure, or traffic volumes to produce a denser aggregate skeleton. However, if the laboratory compaction effort was too high, it could be difficult for the contractor to achieve the required as-constructed density in the field. A general summary of the historic HMA mix design philosophy would be to put as much asphalt in a mix as possible without compromising rut resistance. Hveem (*5*) suggested just enough asphalt to allow adequate compaction in the field with the equipment available. Marshall was quoted as emphasizing the importance of designing the densest (i.e., minimum VMA) possible aggregate structure (*6*).

A tiered system was adopted for the Superpave mix design system. In the Superpave mix design system, minimum required aggregate properties, such as angularity, recommendations for high temperature binder grade, volumetric properties, and laboratory compaction effort all change with design traffic levels.

Buchanan (*71*) demonstrated that for a given gradation, VMA was reduced approximately 1 percent when the Ndesign level was increased by 30 gyrations. Thus, a mixture designed for minimum VMA at an Ndesign level of 125 gyrations would be expected to have a measured VMA of approximately 2 percent above the value at 125 gyrations when compacted to 75 gyrations. Thus, higher Ndesign levels tend to force the aggregate gradation away from the maximum density line. If traffic does not densify these mixtures to as dense of an aggregate structure as the SGC, then the mix gradation may be coarser or finer than is needed. Cooley et al. (*81*) discussed the influence of gradation on pavement permeability. Coarser mixes tend to be more permeable at a given pavement density than finer mixes are. It is also expected that as the Ndesign level is increased, more compaction effort is required to achieve acceptable density in the field, though this has been difficult to quantify.

It should be noted that asphalt content is generally considered to be independent of Ndesign (although dependent for a given mix) and instead dependent on the design (minimum) VMA and air void content. However, Watson et al. (*74*) indicated that the average design VMA for Georgia DOT mixes, using similar aggregates, was higher for

Marshall designed mixes than for Superpave mixes, even though the minimum VMA was the same in both cases. If Ndesign levels are too high, the designer is forced to design closer to the minimum VMA requirement and cannot allow a cushion for production variability.

The field data from this study indicated that the as-constructed density, based on cores, for 55 percent of the projects tested was less than 92 percent of Gmm. Statistical analyses indicated that the agency specifications or practices significantly affected the as-constructed density. Two of the agencies with the best as-constructed densities, Colorado and Georgia, have specifications which tend to increase the asphalt content of the mixture. Colorado DOT designs with 100 mm diameter SGC molds. Samples compacted in a 100 mm diameter molds tend to result in lower sample densities as compared to samples compacted in 150 mm diameter molds for the same number of gyrations. Georgia DOT will field-adjust a mixture's asphalt content in order to ensure specified levels of as-constructed density.

The field projects reached their ultimate density after two years of traffic. The majority of the densification occurred in the first three months. The month in which the project was constructed significantly affected the amount of densification which occurred. Projects constructed in the month of May tended to densify the most (approximately 4.0 percent). Projects constructed in April or June on average densified approximately 0.5 percent less than those constructed in May. Projects constructed in July or August densified slightly less than the average of all of the projects, approximately 3.0 percent. Projects constructed in September of October densified the least, an average of approximately 2.3 percent. High temperature PG or the number of high temperature PG bumps as compared to the climatic PG significantly affected pavement densification. Mixes containing PG 76-22 or with two high temperature PG bumps densified less than softer binders. The majority of the samples from the field projects did not achieve the laboratory air void content at the agency specified Ndesign level (Figure 4.17). At a laboratory air void content of 4 percent, the average in-place air void content was 5.5 percent after two-years of traffic. This indicates that the laboratory compaction effort is higher than the combined compaction during construction and from traffic. Brown et al. (*78*) showed that mixtures designed to 100 gyrations at the 2000 NCAT Test Track compacted to their ultimate density when 10 million ESALs were applied in two years. This equates to more than 100 million ESALs for a 20-year design life, indicating the mixes should have been designed at 125 gyrations using the AASHTO R35-04 Ndesign table. Further, the mixes were designed using an SGC with a low (approximately 1.02) DIA, which would provide less laboratory compaction than an SGC set to a DIA of 1.16 ± 0.02 degrees.

Three different analyses were used to try and determine where the Ndesign levels should be set. In the first analysis, the numbers of gyrations to match the 2-year (ultimate) in-place densities were related to the accumulated traffic. The two different compactors used in the study produced back-calculated Ndesign values which differed by approximately 20 gyrations. These differences were attributed to differences in the DIA for the two compactors. This indicates the affect of DIA on the density of laboratory compacted samples. AASHTO (*4*) has adopted a DIA of 1.16 ± 0.02 degrees as an alternate to an external angle of gyration of 1.25 ± 0.02 degrees. The data were adjusted to a DIA of 1.16 degrees and the resulting back-calculated Ndesign values for the two

SGCs compared well (Figure 4.16). A relationship was developed between Log of design traffic (ESALs) and the Log of Ndesign. There was a good deal of scatter in the data, but this was expected based on the literature review. The exclusion of projects constructed with PG 76-22 improved the relationship. Using this relationship the Ndesign values for the currently specified traffic levels could be calculated. The best fit (R^2 = 0.57) indicated reduced gyration levels at all traffic levels (Figure 4.24). The high side of the 80 percent prediction interval approximated the currently specified Ndesign levels. The 80th percentile for the projects within each category were also calculated; these also indicated reduced Ndesign levels though the reduction in the 0.3 to 1 million ESAL category was minimal. The original Ndesign levels were determined using the best fit of the data, without any adjustment for the confidence or prediction interval (*64*). However, several projects which could not clearly be identified as outliers were excluded from this analysis and it did not address the use of modified binders.

The second analysis looked at the predicted gyrations to match the in-place density at each of the sampling periods (3 months, 6 months, 1 year, 2 years and 4 years). The original Ndesign table was determined by a log-log regression analysis between the gyrations to match the as-constructed density and the density after 12 or more years of traffic and accumulated ESALs (Figure 4.26). This second analysis is then closer to what was originally done to determine the Ndesign levels. This second analysis indicated design gyration levels (Table 4.11) close to those currently specified by AASHTO R 35. However, there is a tremendous amount of scatter in the data (R^2 = 0.37 for Pine Compactor and R^2 = 0.34 for Troxler compactor).

The third analysis attempted to reduce the scatter in the data and to adjust the data for the effect of as-constructed density. As noted previously, 55 percent of the projects had as-constructed densities less than 92 percent. It was demonstrated that the as-constructed density affected the 2-year or ultimate density. Models were developed to relate the 2-year percent of laboratory density at 100 gyrations to as-constructed density, high PG grade, and accumulated ESALs. It was found that the predicted gyrations to match a given percentage of laboratory density represented a small range with a standard deviation between 3.44 and 8.99 gyrations. A matrix of expected percentages of laboratory density was developed based on high PG grade and traffic (Table 4.14). The as-constructed density was set to 92 percent in all cases. The number of gyrations to match the percentage of laboratory density determined in the matrix was calculated for each of the projects. An equation was then developed to relate the average gyrations determined to match the in-place densities to high PG grade and traffic, assuming an as-constructed density of 92 percent. Table 4.15 summarizes these results which are similar to the results determined using the first analysis (Table 4.10).

Rut depth measurements were taken in the field at the two-year and four-year sampling intervals. A maximum average rut depth for a project after four years of traffic was 7.4 mm with an overall average of 2.7 mm. The rut depth measurements alone support lowering the Ndesign levels since even at 95 percent reliability 2 of 40 pavements would be expected to have unacceptable levels of rutting. Similar findings were reported for the 2000 NCAT Test Track. It was also noted that sections constructed with PG 76-22 at the 2000 NCAT Test Track rutted 60 percent less than sections constructed with PG 67-22. Most of the rutting at the 2000 NCAT Test Track was attributed to pavement densification.

Combined, these data indicate that the Ndesign levels can be reduced. As noted previously, the predicted Ndesign levels change very rapidly at 20-year design traffic levels less than 3 million ESALs; therefore, caution must be used in this region. Though lower Ndesign values than currently specified are recommended based on the first analysis for the lowest traffic levels (Table 4.10), there is little or no experience with these levels. Further, density and therefore optimum asphalt content can change very rapidly at lower gyration levels. If the levels are low enough, the compacted samples are not stable immediately after compaction. Therefore, it is recommended that 50 gyrations be maintained for the lowest traffic levels.

The combined data from the field projects and the 2000 NCAT Test Track indicate that a maximum Ndesign level of 100 gyrations will provide good performance for very high traffic levels. This is a 25 gyration decrease from the currently specified levels. Table 4.19 summarizes the recommended Ndesign levels for all traffic levels. The values in Table 4.19 are based on Equations 13 and 14. The predicted values from Equation 13 were presented in Table 4.15. The values in Table 4.15 were rounded to produce 4 levels. The largest rounding occurred at 30 million ESALs where the predicted value was 88 and 86 based on Equations 13 and 14, respectively. The recommended Ndesign levels from Table 4.15 are slightly more conservative than the Ndesign levels recommended in Table 4.10. The recommended Ndesign values based on Table 4.15 also account for the effect of PG 76-22. Values are presented for two binder grades, PG 64-22 and PG 76-22.

TABLE 4.19 Proposed Ndesign Levels for an SGC DIA of 1.16 ± 0.02 Degrees

20-Year Design Traffic, ESALs	2-Year Design Traffic, ESALs	Ndesign Unmodified	Ndesign PG 76-22
< 300,000	< 30,000	50	NA
300,000 to 3,000,000	30,000 to 230,000	65	50
3,000,000 to 10,000,000	230,000 to 925,000	80	65
10,000,000 to 30,000,000	925,000 to 2,500,000	80	65
> 30,000,000	> 2,500,000	100	80

In addition to the 20-year design traffic, a two-year design traffic level is shown. The two-year ESALs were used to develop most of the relationships in this study. A 20-year design for a surface course is most likely unreasonably long. Further, the specified traffic growth rate has a large effect on the 20-year design traffic. The WesTrack experiment noted that rate of loading was important, especially for temporary pavements designed for short periods (*87*).

The use of lower Ndesign levels will tend to allow mixtures to be designed with gradations closer to the maximum density line and still meet minimum VMA requirements. The use of lower Ndesign levels will tend to increase optimum asphalt contents slightly since contractors will most likely design with a slightly larger cushion above the minimum specified VMA. However, to ensure the optimum asphalt contents increased, the minimum VMA requirements would also need to be increased. An increase in the minimum VMA requirements of 0.5 percent would result in an increase of approximately 0.2 percent in optimum asphalt content. Thus, the adoption of the recommended Ndesign levels in Table 4.19 along with an increase in minimum VMA of 0.5 percent would have a combined effect of allowing somewhat denser gradations and increasing the optimum asphalt content slightly.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

The three objectives of this research were 1) to evaluate the field densification of pavements designed using the Superpave mix design system, 2) to verify or determine the correct Ndesign levels, and 3) to evaluate the locking point concept. A wide range of climates, design traffic levels, PG Binder grades, lift thickness to NMAS, gradations and aggregate types were included in this study.

The general goal of previous studies to determine the appropriate laboratory compaction effort has been to determine the laboratory compaction effort that matches the ultimate density of the pavement after the application of traffic. Previous studies to determine or confirm laboratory compaction efforts have indicated a great deal of variability between field and laboratory compaction; therefore, variability was expected in this study. The variability in this study may have been acerbated by three factors:

1. Field and traffic compaction are generally constant stress while the SGC is a constant strain device,
2. The mixes sampled in this study contained a wide range of binder grades, not typical of previous studies,
3. The mixes in this study were designed under a tiered system of aggregate properties and Ndesign levels.

5.1 CONCLUSIONS

Based on the results from this research study, the following conclusions can be made.

1. Pavements appear to reach their ultimate density after two years of traffic. The average in-place density for all of the projects was the same at 2- and 4-years (94.6 percent of Gmm). A fair relationship was determined between the as-constructed density and the density after two years of traffic. The majority of pavement densification, approximately 66 percent, occurs during the first three months after construction. Both the high PG binder grade and the high temperature bumps between the climatic and specified PG were found to significantly affect pavement densification, with stiffer binders resulting in less densification. The ultimate in-place densities of the pavements evaluated in this study were approximately 1.5 percent less than the densities of the laboratory compacted samples at the agency specified Ndesign.
2. The number of gyrations to match the ultimate in-place density was calculated for each project in this study. The calculated values for the two compactors used in this study differed by approximately 20 gyrations. This was attributed to differences in their DIA. The predicted gyrations, adjusted to a DIA of 1.16 degrees showed good agreement between the two machines.
3. A relationship was developed between predicted Ndesign and design traffic for the projects which were not constructed using PG 76-22. Although there was a great deal of scatter in the data, this was expected. The predicted gyration levels were generally less than those currently specified.

4. A relationship was also developed to relate the 2-year percent of laboratory density at 100 gyrations to as-constructed density, high PG grade, and accumulated ESALs. It was found that the predicted gyrations to match a given percentage of laboratory density represented a small range with a standard deviation between 3.44 and 8.99 gyrations. A matrix of expected percentages of laboratory density was developed based on high PG grade, traffic and an as-constructed density of 92 percent. The numbers of gyrations to match the percentages of laboratory density determined in the matrix were calculated for all of the projects. An equation was then developed to relate the average gyrations determined to match the in-place densities to high PG grade and traffic. The predicted gyrations were very similar to those determined using the first analysis. However, this analysis accounted for the use of PG 76-22. It was found that Ndesign could be reduced by approximately 15 gyrations when PG 76-22 was specified.
5. All of the projects in this study were very rut resistant. The maximum observed rutting for the field projects was 7.4 mm with an average rut depth for all of the projects of 2.7 mm after 4 years of traffic.
6. The requirements for Ninitial were evaluated based on the field project data. AASHTO M 35 specifies a tiered density requirement at Ninitial depending on traffic level. In the 300,000 to 3,000,000 ESAL range, 32 percent of the samples failed Ninitial requirement. In the greater than 3,000,000 million ESAL range, 20 percent of samples failed Ninitial requirement. The majority of the projects which failed Ninitial were fine-graded. All of the projects are performing well in

terms of rutting resistance. Only one project failed Ninital and was tender in the field. There is no strong evidence to keep the requirements for Ninitial

7. The requirement for Nmaximum was evaluated based on the field project data. AASHTO M 35 specifies a density requirement of less than 98 percent at Nmaximum to guard against the potential for rutting. Thirty-six percent of the samples tested with the Pine compactor and 40 percent of the samples tested with the Troxler compactor failed the density requirements at Nmaximum. However, the projects have all been extremely rut resistant. Therefore, the density requirement at Nmaximum does not appear to be a good indicator of rutting potential and should be eliminated.

5.2 RECOMMENDATIONS

Based on the research conducted in this study, the following recommendations are made:

The specification for angle of gyration should be revised to only allow a DIA of 1.16 ± 0.02 degrees. The Ndesign levels shown in Table 5.1 should be adopted for the design of Superpave HMA. Consideration should be given to the use of the 2-year design traffic volume to determine Ndesign as opposed to the 20-year design traffic volume. The criteria for Ninitial and Nmaximum should be eliminated.

TABLE 5.1 Recommended Ndesign Levels for an SGC DIA of 1.16 ± 0.02 Degrees

20-Year Design Traffic, ESALs	2-Year Design Traffic, ESALs	Ndesign Unmodified	Ndesign PG 76-22
< 300,000	< 30,000	50	NA
300,000 to 3,000,000	30,000 to 230,000	65	50
3,000,000 to 10,000,000	230,000 to 925,000	80	65
10,000,000 to 30,000,000	925,000 to 2,500,000	80	65
> 30,000,000	> 2,500,000	100	80

CHAPTER 6 REFERENCES

1. Cominsky, R., "The Superpave Mix Design Manual for New Construction and Overlays," SHRP-A-407, Strategic Highway Research Program, National Research Council, Washington, DC, 1994.

2. Brown, E. R., and M. S. Buchanan, "Superpave Gyratory Compaction Guidelines," Research Results Digest No. 237, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D. C., 1999.

3. Anderson, R. M., R. B. McGennis, W. On Tam, and T. W. Kennedy, "Sensitivity of Mixture Performance Properties to Changes in Laboratory Compaction Using the Superpave Gyratory Compactor," In Journal of the Association of Asphalt Paving Technologists, Vol. 69, Reno, NV, 2000, Pp 1-33.

4. American Association of State Highway and Transportation Officials, Standard Specifications for Transportation Materials and Methods of Sampling and Testing Part 1B: Specifications,25th Ed., Washington, DC, 2005.

5. Hveem, F. N., "Asphalt Pavements from the Ancient East to the Modern West," Fifth Annual Nevada Street and Highway Conference, 1970.

6. Leahy, R. B., and R. B. McGennis, "Asphalt Mixes: Materials, Design and Characterization," In Journal of the Association of Asphalt Paving Technologists, Vol. 68A, Chicago, IL, 1999, Pp 70-127.

7. Holley *.pdf history of Asphalt Pavements from Internet

8. Halstead, W. J., and J. Y. Welborn, "History of the Development of Asphalt Testing Apparatus and Asphalt Specifications," In Proceedings Association of Asphalt Paving Technologists Historical Session, Vol. 43A, Williamsburg, VA, 1974, Pp 89-120.

9. Crawford, C. "The Rocky Road of Mix Design" Hot Mix Asphalt Technology, Winter 1989, National Asphalt Pavement Association, Riverdale, MD.

10. Davis, R. L. "Large Stone Mixes: A Historical Insight" Information Series 103, National Asphalt Pavement Association, Riverdale, MD, 1988.

11. Roberts, F. L. , P. S. Kandhal, E. R. Brown, D-Y, Lee, and T. W. Kennedy, Hot Mix Asphalt Materials, Mixture Design and Construction Second Edition, NAPA Education Foundation, Lanham, MD, 1996.

12. Vallegra, B. A., and W. R. Lovering, "Evolution of the Hveem Stabilometer Method of Designing Asphalt Paving Mixtures," In Proceedings of the Association of Asphalt Paving Technologists, Vol. 54, San Antonio, 1985, Pp 243-265.

13. Hveem, F. N. "Use of the Centrifuge Kerosene Equivalent as Applied to Determine the Required Oil Content for Dense Graded Bituminous Mixes", In Proceedings of The Association of Asphalt Paving Technologists, Vol. 13, Chicago, 1942, Pp 9-41.

14. War Department, Corps of Engineers, U. S. Army, Mississippi River Commission, "Investigation of the Design and Control of Asphalt Paving Mixtures," Technical Memorandum No. 3-254, Vol. 1, *U.S. Waterways Experiment Station*, Vicksburg, MS, May 1948.

15. White, T. D., "Marshall Procedures for Design and Quality Control of Asphalt Mixtures," In Proceedings of the Association of Asphalt Paving Technologists, Vol. 54, San Antonio, 1985, Pp 265-284.

16. McLeod, N.W. Relationship Between Density, Bitumen Content, and Voids Properties of Compacted Paving Mixtures. *Proceedings of the Highway Research Board,* Vol. 35, 1956.

17. McLeod, N.W. Selecting the Aggregate Specific Gravity for Bituminous Paving Mixtures. *Proceedings of the Highway Research Board,* Vol. 36, 1957.

18. The Asphalt Institute. *Mix Design Methods for Asphalt Concrete (MS-2)*, 2nd ed., May 1963.

19. Foster, C. R., "Development of Marshall Procedures for Designing Asphalt Paving Mixtures," Information Series 84, National Asphalt Pavement Association, 1982.

20. Kandhal, P. S., and W. S. Koehler, "Marshall Mix Design Method: Current Practices," In Proceedings of the Association of Asphalt Paving Technologists, Vol. 54, San Antonio, 1985, Pp 284-303.

21. Ortolani, L., and H. A. Sandberg, Jr., "The Gyratory-Shear Method of Molding Asphaltitc Concrete Test Specimens; Its Development and Correlation with Field Compaction Methods. A Texas Highway Department Standard Procedure," In

Proceedings of the Association of Asphalt Paving Technologists, Vol. 21, Cincinnati, OH, 1951, Pp 280-297.

22. Philippi, O. A., "Compaction of Bituminous Concrete," In Proceedings of the Association of Asphalt Paving Technologists, Vol. 26, Atlanta, GA, 1957, Pp 191-201.

23. American Society for Testing and Materials, Annual Book of Standards, Vol. 4.03 Road and Paving Materials; Vehicle Pavement Systems, West Conshohocken, PA, 2004.

24. Texas Department of Transportation, "Bituminous Test Procedures Manual," 200-F, Austin, TX, 2005.

25. McRae, J. L., "Compaction of Bituminous Concrete," In Proceedings of the Association of Asphalt Paving Technologists, Vol. 26, Atlanta, GA, 1957, Pp 206-213.

26. McRae, J. L., and A. R. McDaniel, "Progress Report on the Corps of Engineers' Kneading Compactor for Bituminous Mixtures," In Proceedings of the Association of Asphalt Paving Technologists, Vol. 27, Montreal, Quebec, Canada, 1958, Pp- 357-382.

27. Ruth, B. E., X. Shen, L-H, Wang, "Gyratory Evaluation of Aggregate Blends to Determine their Effect on Shear Resistance and Sensitivity to Asphalt Content, " Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance, ASTM STP 1147, R. C. Meininger, Ed., American Society for Testing and Materials, Philaelphia, 1992, Pp 252-264.

28. Bonnot, J., "Asphalt Aggregate Mixtures," In Transportation Research Record 1096, Transportation Research Board, National Research Council, Washington, D. C., 1986, Pp 42-51.

29. Cominsky, R., R. B. Leahy, and E. T. Harrigan, "Level One Mix Design: Materials Selection, Compaction, and Conditioning," SHRP-A-408, Strategic Highway Research Program, National Research Council, Washington, DC, 1994.

30. Hughes, C. S., "Compaction of Asphalt Pavement," NCHRP Synthesis No. 152, Transportation Research Board, National Research Council, Washington, DC, 1989.

31. Consuegra, A., D. N. Little, H. Von Quintus, and J. Burati., "Comparative Evaluation of Laboratory Compaction Devices Based on Their Ability to Produce Mixtures with Engineering Properties Similar to Those Produced in the Field," In

Transportation Research Record 1228, Transportation Research Board, National Research Council, Washington, DC, 1989, Pp 80-87.

32. Von Quintus, H. L., J. A. Scherocman, C. S. Hughes, and T. W. Kennedy, "Asphalt-Aggregate Mixture Analysis System AAMAS," NCHRP Report No. 338, Transportation Research Board, National Research Council, Washington, DC, 1991.

33. Sousa, J. B., J. Harvey, L. Painter, J. A. Deacon, and C. L. Monismith, "Evaluation of Laboratory Procedures for Compacting Asphalt-Aggregate Mixtures," Report No. SHRP-A/UWP-91-523, Strategic Highway Research Program, National Research Council, Washington, D. C., 1991.

34. Button, J. W., D. N. Little, V. Jagadam, and O. J. Pendleton, "Correlation of Selected Laboratory Compaction Methods with Field Compaction," In Transportation Research Record No. 1454, Transportation Research Boards, National Research Council, Washington, D. C. 1994, Pp 193-201.

35. Moultier, R., "Utilization and Possibilities of the Gyratory Shear Compacting Press," In Liaison Bulletin of the Bridge and Road Laboratories Special Issue: Bitumens and Bituminous Concrete, 1977.

36. American Association of State Highway and Transportation Officials, AASHTO Provisional Standards. Washington, DC, 2001.

37. Yildirim, Y., M. Solaimanian, R. B. McGennis, and T. W. Kennedy. "Comparative Analysis of Volumetric Properties for Superpave Gyratory Compactors." In Transportation Research Record 1712, TRB, National Research Council, Washington, DC. 2000.

38. McGennis, R. B., R. M. Anderson, D. Perdomo and P. Turner. "Issues Pertaining to Use of Superpave Gyratory Compactor." In Transportation Research Record 1543, TRB, National Research Council, Washington, DC. 1996. Pp. 139-144.

39. McGennis, R. B., D. Perdomo, T. W. Kennedy, and M. Solaimanian. "Ruggedness Evaluation of AASHTO TP4: The Superpave Gyratory Compactor, Journal of the Association of Asphalt Paving Technologists, Vol. 66, 1997, pp.277-311.

40. American Association of State Highway and Transportation Officials, AASHTO Provisional Standards. Washington, DC, 2002.

41. "Application Brief Investigating Measurable Gmb Differences Among Gyratory Compacted Specimens A Round Robin Study." Troxler Electronic Laboratories, Research Triangle Park, NC, 2001.

42. Hinrichsen, J., “Comparison of Four Brands of Superpave Gyratory Compactors.” In Transportation Research Record 1767, TRB, National Research Council, Washington, DC. 2001. Pp. 167-172.

43. “Operating Instructions for the Dynamic Angle Validation Kit (DAV) for Superpave Gyratory Compactors.” Test Quip Corporation, New Brighton, MN. 2001.

44. Dalton, F. “Observations of SGC End Plate Deflection Using the FHWA Angle Validation Kit.” Report 2000-02 Revision C. Pine Instrument Company, PA. 2000.

45. Dalton, F. “Investigation of Pine Superpave Gyratory Compactors Using the FHWA Angle Validation Kit” Report 2000-03 Revision A. Pine Instrument Company, PA. 2000.

46. Harman, T. “Dynamic Internal Angle (DIA) Measurement in the Troxler Superpave Gyratory Compactor – Linearity Study.” Turner-Fairbank Highway Research Center, Federal Highway Administration. McLean, VA. 2001.

47. Dalton, F. “Application of Internal Angle Verification to Obtain Equivalent Results from Various SGC Models” Report 2001-01 Revision A. Pine Instrument Company, PA. 2001.

48. Al-Khaateeb, G., C. Paugh, K. Stuart, T. Harman, and J. D’Angelo. “Target and Tolerance for the Angle of Gyration Used in the Superpave Gyratory Compactor (SGC).” In Transportation Research Record 1789, TRB, National Research Council, Washington, DC. 2002. Pp. 208-215.

49. Prowell, B. D., E. R. Brown and M. Huner, Evaluation of the Internal Angle of Gyration of Superpave Gyratory Compactors in Alabama, NCAT 03-04. National Center for Asphalt Technology, 2003.

50. War Department, Corps of Engineers, U. S. Army, Mississippi River Commission, “Investigation of the Design and Control of Asphalt Paving Mixtures,” Technical Memorandum No. 3-254, Vol. 3, *U.S. Waterways Experiment Station*, Vicksburg, MS, May 1948.

51. Dillard, J. H. “Comparison of Density of Marshall Specimens and Pavement Cores.” In Proceeding of the Association of Asphalt Paving Technologists, Vol. 24 Louisville, KY, 1955. pp. 178-232.

52. Campen, W. H., J. R. Smith, L. G. Erickson and L. R. Mertz, “The Effect of Traffic on the Density of Bituminous Paving Mixtures,” In Proceedings of the

Association of Asphalt Paving Technologists, Vol. 30, Charleston, SC, 1960, Pp 378-397.

53. Graham, M. D., W. C. Burnett, J. J. Thomas, and W. C. Dixon. “Pavement Density-What Influences It.” In Proceedings of the Association of Asphalt Paving Technologists, Vol. 34, Philadelphia, 1965. pp. 286-308.

54. Woodward, E. J., Jr., and J. L. Vicelja, “Aviation Boulevard – Evaluation of Materials, Equipment and Construction Procedures,” In Proceedings of the Association of Asphalt Paving Technologists, Vol. 34, Philadelphia, PA, 1965, Pp 215-233.

55. Bright, R., B. Steed, J. Steele, and A. Justice, “The Effect of Viscosity of Asphalt on Properties of Bituminous Wearing Surface Mixtures,” In Proceedings of the Association of Asphalt Paving Technologists, Vol. 35, Minneapolis, MN, 1966, Pp 164-206.

56. Serafin, P. J., L. L. Kole and A. P. Chritz. “Michigan Bituminous Experimental Road: Final Report.” In Proceedings of The Association of Asphalt Paving Technologists, Vol. 36, Denver, CO, 1967. pp. 582-614.

57. Palmer, R. K. and J. J. Thomas. “Pavement Density-How It Changes.” In Proceedings of The Association of Asphalt Paving Technologists, Vol. 37, Atlanta, 1968. pp. 542-583.

58. Epps, J. A., B. M. Gallaway and W. W. Scott, Jr. “Long-Term Compaction of Asphalt Concrete Pavements.” In *Highway Research Record*, No. 313. Highway Research Board, National Research Council, Washington, DC, 1970. pp. 79-91.

59. Kandahl, P. S., and M. E. Wenger. “Asphalt Properties in Relation to Pavement Performance.” In Transportation Research Record, No. 544, Transportation Research Board, National Research Council, Washington, DC, 1975. pp. 1-13.

60. Brown, E. R., and S. A. Cross, “Comparison of Laboratory and Field Density of Asphalt Mixtures,” In Transportation Research Record 1300, Transportation Research Board, National research Council, Washington, D.C., 1991, Pp 1-12.

61. Hanson, D. I., R. B. Mallick and E. R. Brown. “Five-Year Evaluation of HMA Properties at the AAMAS Test Projects.” In *Transportation Research Record*, No. 1454, Transportation Research Board, National Research Council, Washington, DC, 1994. pp. 134-143.

62. Stroup-Gardiner, M., D. E. Newcomb, R. Olson, and J. Teig. “Traffic Densification of Asphalt Concrete Pavements.” In *Transportation Research*

Record, No. 1675, Transportation Research Board, National Research Council, Washington, DC, 1997. pp. 1-9.

63. Brown, E. R., and R. B. Mallick, "An Initial Evaluation for N_{design} Superpave Gyratory Compactor," In Journal of the Association of Asphalt Paving Technologists, Vol. 67, Boston, MA, 1998, Pp 101-124.

64. Blankenship, P. B., "Gyratory Compaction Characteristics: Relation to Service Densities of Asphalt Mixtures," Master's Thesis, University of Kentucky, Lexington, KY, 1993.

65. Sousa, J.B., G. Way, J. T. Harvey, and M. Hines, "Comparison of Mix Design Concepts," In Transportation Research Record 1492, Transportation Research Board, National Research Council, Washington, D.C., 1995, Pp 151-160.

66. Harman, T. P., J. D'Angelo, and J. R. Bukowski, "Evaluation of SUPERPAVE Gyratory Compactor in the Field Management of Asphalt Mixes: Four Simulation Studies," In Transportation Research Record 1513, Transportation Research Board, National Research Council, Washington, D. C., 1995, Pp 1-8.

67. Gowda, G., K. Hall, R. Elliot, A. Meadors, "Critical Evaluation of Superpave Volumetric Mix Design Using Arkansas Surface Course Mixes," In Journal of the Association of Asphalt Paving Technologists, Vol. 66, Salt Lake City, UT, 1997, Pp 250-276.

68. Habib, A., M. Hossain, R. Kaldate, and G. A. Fager, "Comparison of Superpave and Marshall Mixtures for Low-Volume Roads and Shoulders," In Transportation Research Record 1609, Transportation Research Board, National Research Council, Washington, D. C., 1998, Pp 44-50.

69. Mallick, R. B., S. Buchanan, E. R. Brown, and M. Huner, "Evaluation of Superpave Gyratory Compaction of Hot Mix Asphalt," In Transportation Research Record 1638, Transportation Research Board, National Research Council, Washington, D. C., 1998, Pp 111-119.

70. Forstie, D. A., and D. K. Corum, "Determination of Key Gyratory Compaction Points for Superpave Mix Design in Arizona," In Progress of Superpave (Superior Performing Asphalt Pavement): Evaluation and Implementation, ASTM STP 1322, R. N. Jester, Ed., American Society for Testing and Materials, 1997. Pp 201-208.

71. Buchanan, M. S., "Evaluation of the Superpave Compaction Procedure," Doctoral Dissertation, Auburn University, Auburn, AL, 1999.

72. American Association of State Highway and Transportation Officials, AASHTO Provisional Standards. Washington, DC, 2001.

73. Harmelink, D. and T. Aschenbrener, In-Place Voids Monitoring of Hot Mix Asphalt Pavements Report No. CDOT-DTD-R-2002-11, Colorado Department of Transportation, Denver, CO, 2002.

74. Watson, D. E., J. Moore and E. R. Brown, Verification of Superpave N_{design} Compaction Levels Phase I Final Report, Georgia Department of Transportation, Atlanta, GA, 2005.

75. Pine, W. J., Superpave Gyratory Compaction and the Ndesign Table, Internal Report to Illinois Department of Transportation, Illinois, 1997.

76. Vavrik, W. R., and S. H. Carpenter, "Calculating Air Voids at Specified Numbers of Gyrations in Superpave Gyratory Compactor," In Transportation Research Record 1630, Transportation Research Board, National Research Council, Washington, DC, 1998, Pp 117-125.

77. Federal Highway Administration. *LTPPBIND*, Version 2.1. Washington, D. C. 1999.

78. Brown, E. R., B. D. Prowell, L. A. Cooley, Jr., J. Zhang, and R. B. Powell, *Evaluation of Rutting Performance on 2000 NCAT Test Track*, In Journal of Association of Asphalt Paving Technologists, Vol. 73, Baton Rouge, LA 2004, Pp 287-336.

79. American Association of State Highway and Transportation Officials, AASHTO Guide for Design of Pavement Structures 1986. AASHTO, Washington, D.C. 1986.

80. Brown, E. R. Density of Asphalt Concrete – How Much Is Needed? Transportation Research Record 1282, Transportation Research Board, National Academy of Sciences, Washington, D.C. 1990. pp. 27-32.

81. Cooley, Jr., L.A., E.R. Brown, and S. Maghsoodloo. Development of Critical Field Permeability and Pavement Density Values for Coarse Graded Superpave Pavements. In Transportation Research Record 1761, Transportation Research Board, National Research Council. 2001. pp 41-49.

82. Linden, R. N., J. P. Mahoney, and N. C. Jackson. Effect of Compaction on Asphalt Concrete Performance. In *Transportation Research Record 1217,* TRB, National Research Council, Washington, D.C. 1989, Pp 20-28.

83. Harvey, J. T., T. Hoover, N. F. Coetzee, W. A. Nokes, and F. C. Rust. CALTRANS Accelerated Pavement Test (CAL/APT) Program-Test Results: 1994-1997. *Journal of the Association of Asphalt Paving Technologists,* Vol. 67, 1998, Pp 644-689.

84. Brown, E. R., and S. A. Cross, "A National Study of Rutting in Hot Mix Asphalt (HMA) Pavements," In Journal of the Association of Asphalt Paving Technologists, Vol. 61, Charleston, SC, 1992, Pp 535-583.

85. Montgomery, D. C., Design and Analysis of Experiments, Fifth Ed., John Wiley & Sons, New York, 2001.

86. Moseley, H. L., G. C. Page, J. A. Musselman, G. A. Sholar, and P. B. Upshaw, Evaluation of Dynamic Angle Validator. In *Transportation Research Record 1891,* TRB, National Research Council, Washington, D.C. 1891, Pp 191-197.

87. Federal Highway Administration. "Performance of Coarse-Graded Mixes at WesTrack–Premature Rutting," Final Report. Washington, D.C. 1998.

Appendix

Field Project Data

TABLE A.1 SGC Data for Project AL-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.549	2.473	2.504	87.1	91.7	94.3	95.9	97.0	97.3	98.2
1-2	2.549	2.472	2.502	87.1	91.6	94.3	95.8	97.0	97.2	98.2
1-3	2.549	2.475	2.514	87.5	92.0	94.5	96.0	97.1	97.7	98.6
AVG				87.2	91.7	94.4	95.9	97.0	97.4	98.3
2-1	2.566	2.472	2.506	86.7	91.2	93.8	95.2	96.3	96.8	97.7
2-2	2.566	2.458	2.493	86.1	90.6	93.3	94.7	95.8	96.2	97.2
2-3	2.566	2.453	2.507	85.7	90.3	93.0	94.5	95.6	96.8	97.7
AVG				86.2	90.7	93.4	94.8	95.9	96.6	97.5
3-1	2.548	2.414	2.488	85.4	89.6	92.2	93.6	94.7	96.8	97.6
3-2	2.548	2.468	2.489	87.2	91.8	94.4	95.8	96.9	96.7	97.7
3-3	2.548	2.443	2.490	86.0	90.6	93.2	94.7	95.9	96.8	97.7
AVG				86.2	90.7	93.3	94.7	95.8	96.8	97.7

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.549	2.450	2.489	86.0	90.6	93.4	94.9	96.1	96.7	97.6
1-2	2.549	2.476	2.502	87.3	91.9	94.6	96.0	97.1	97.2	98.2
1-3	2.549	2.462	2.494	86.7	91.3	94.0	95.4	96.6	96.9	97.8
AVG				86.7	91.3	94.0	95.4	96.6	96.9	97.9
2-1	2.566	2.435	2.490	84.9	89.5	92.2	93.7	94.9	96.0	97.0
2-2	2.566	2.468	2.471	86.4	91.0	93.6	95.0	96.2	95.3	96.3
2-3	2.566	2.445	2.521	85.6	90.0	92.5	94.0	95.3	97.4	98.2
AVG				85.6	90.1	92.8	94.2	95.5	96.2	97.2
3-1	2.548	2.414	2.476	85.4	89.7	92.1	93.6	94.7	96.2	97.2
3-2	2.548	2.438	2.467	85.8	90.4	93.0	94.5	95.7	95.8	96.8
3-3	2.548	2.436	2.478	86.0	90.4	92.9	94.4	95.6	96.2	97.3
AVG				85.8	90.1	92.7	94.2	95.3	96.1	97.1

TABLE A.2 SGC Data for Project AL-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.466	2.397	2.430	86.0	91.3	94.5	96.1	97.2	97.8	98.5
1-2	2.466	2.390	2.409	85.6	91.0	94.2	95.9	96.9	96.9	97.7
1-3	2.466	2.387	2.408	85.8	91.1	94.1	95.8	96.8	96.8	97.6
AVG				85.8	91.1	94.3	95.9	97.0	97.1	98.0
2-1	2.455	2.363	2.375	84.7	90.2	93.3	95.1	96.3	95.8	96.7
2-2	2.455	2.357	2.398	84.7	90.0	93.2	94.9	96.0	96.7	97.7
2-3	2.455	2.339	2.396	84.2	89.2	92.3	94.1	95.3	96.8	97.6
AVG				84.5	89.8	92.9	94.7	95.8	96.4	97.3
3-1	2.460	2.359	2.405	84.6	89.9	93.1	94.8	95.9	96.9	97.8
3-2	2.460	2.341	2.396	83.7	89.0	92.2	94.0	95.2	96.6	97.4
3-3	2.460	2.352	2.394	84.3	89.5	92.7	94.5	95.6	96.4	97.3
AVG				84.2	89.5	92.6	94.4	95.6	96.6	97.5

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.466	2.386	2.407	85.4	90.6	93.6	95.5	96.8	96.7	97.6
1-2	2.466	2.370	2.411	83.8	90.1	93.2	95.0	96.1	96.9	97.8
1-3	2.466	2.367	2.410	83.9	89.8	92.9	94.8	96.0	96.8	97.7
AVG				84.4	90.1	93.2	95.1	96.3	96.8	97.7
2-1	2.455	2.326	2.342	83.4	88.5	91.7	93.6	94.7	94.4	95.4
2-2	2.455	2.328	2.342	83.8	88.8	91.9	93.6	94.8	94.4	95.4
2-3	2.455	2.303	2.364	82.5	87.7	90.8	92.6	93.8	95.3	96.3
AVG				83.2	88.4	91.5	93.3	94.5	94.7	95.7
3-1	2.460	2.314	2.345	83.0	88.1	91.1	92.9	94.1	94.4	95.3
3-2	2.460	2.315	2.365	82.8	87.9	91.0	92.8	94.1	95.2	96.1
3-3	2.460	2.313	2.365	82.9	88.0	91.1	92.9	94.0	95.2	96.1
AVG				82.9	88.0	91.1	92.9	94.1	94.9	95.9

TABLE A.3 SGC Data for Project AL-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.472	2.396	2.428	89.3	93.0	95.1	96.3	96.9	97.6	98.2
1-2	2.472	2.391	2.423	89.2	93.1	95.0	96.1	96.7	97.5	98.0
1-3	2.472	2.395	2.428	88.9	92.9	95.1	96.1	96.9	97.6	98.2
AVG				89.1	93.0	95.1	96.2	96.8	97.6	98.2
2-1	2.487	2.430	2.439	89.4	93.6	95.8	97.0	97.7	97.5	98.1
2-2	2.487	2.429	2.428	89.7	93.7	95.9	97.0	97.7	97.0	97.6
2-3	2.487	2.429	2.448	89.2	93.4	95.7	96.9	97.7	97.8	98.4
AVG				89.4	93.6	95.8	97.0	97.7	97.4	98.0
3-1										
3-2										
3-3										
AVG										

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.472	2.380	2.417	88.5	92.3	94.5	95.5	96.3	97.1	97.8
1-2	2.472	2.373	2.406	88.4	91.9	94.1	95.2	96.0	96.7	97.3
1-3	2.472	2.372	2.400	88.0	91.9	94.1	95.2	96.0	96.5	97.1
AVG				88.3	92.0	94.2	95.3	96.1	96.8	97.4
2-1	2.487	2.412	2.448	88.7	92.7	94.9	96.2	97.0	97.8	98.4
2-2	2.487	2.412	2.436	88.5	92.6	95.0	96.1	97.0	97.3	97.9
2-3	2.487	2.415	2.436	88.7	92.7	95.1	96.3	97.1	97.4	97.9
AVG				88.6	92.7	95.0	96.2	97.0	97.5	98.1
3-1										
3-2										
3-3										
AVG										

TABLE A.4 SGC Data for Project AL-4

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.472	2.396	2.428	89.3	93.0	95.1	96.3	96.9	97.6	98.2
1-2	2.472	2.391	2.423	89.2	93.1	95.0	96.1	96.7	97.5	98.0
1-3	2.472	2.395	2.428	88.9	92.9	95.1	96.1	96.9	97.6	98.2
AVG				89.1	93.0	95.1	96.2	96.8	97.6	98.2
2-1	2.487	2.430	2.439	89.4	93.6	95.8	97.0	97.7	97.5	98.1
2-2	2.487	2.429	2.428	89.7	93.7	95.9	97.0	97.7	97.0	97.6
2-3	2.487	2.429	2.448	89.2	93.4	95.7	96.9	97.7	97.8	98.4
AVG				89.4	93.6	95.8	97.0	97.7	97.4	98.0
3-1										
3-2										
3-3										
AVG										

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.472	2.380	2.417	88.5	92.3	94.5	95.5	96.3	97.1	97.8
1-2	2.472	2.373	2.406	88.4	91.9	94.1	95.2	96.0	96.7	97.3
1-3	2.472	2.372	2.400	88.0	91.9	94.1	95.2	96.0	96.5	97.1
AVG				88.3	92.0	94.2	95.3	96.1	96.8	97.4
2-1	2.487	2.412	2.448	88.7	92.7	94.9	96.2	97.0	97.8	98.4
2-2	2.487	2.412	2.436	88.5	92.6	95.0	96.1	97.0	97.3	97.9
2-3	2.487	2.415	2.436	88.7	92.7	95.1	96.3	97.1	97.4	97.9
AVG				88.6	92.7	95.0	96.2	97.0	97.5	98.1
3-1										
3-2										
3-3										
AVG										

TABLE A.5 SGC Data for Project AL-5

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.487	2.437	2.458	91.9	95.0	96.6	97.5	98.0	98.5	98.8
1-2	2.487	2.442	2.454	92.0	95.2	96.7	97.6	98.2	98.2	98.7
1-3	2.487	2.439	2.458	91.8	95.0	96.7	97.5	98.1	98.5	98.8
AVG				91.9	95.1	96.7	97.5	98.1	98.4	98.8
2-1	2.493	2.445	2.458	91.9	95.0	96.7	97.5	98.1	98.3	98.6
2-2	2.493	2.441	2.458	91.6	94.9	96.6	97.4	97.9	98.2	98.6
2-3	2.493	2.444	2.462	91.8	95.0	96.7	97.5	98.0	98.4	98.8
AVG				91.8	95.0	96.6	97.5	98.0	98.3	98.6
3-1	2.493	2.426	2.456	91.1	94.2	95.9	96.7	97.3	98.2	98.5
3-2	2.493	2.441	2.461	91.8	95.0	96.7	97.5	97.9	98.3	98.7
3-3	2.493	2.438	2.462	91.7	94.9	96.5	97.3	97.8	98.4	98.8
AVG				91.5	94.7	96.3	97.2	97.7	98.3	98.7

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.487	2.418	2.443	90.9	94.0	95.8	96.6	97.2	97.8	98.2
1-2	2.487	2.406	2.438	90.6	93.6	95.4	96.2	96.7	97.6	98.0
1-3	2.487	2.420	2.489	91.0	94.2	95.9	96.7	97.3	99.6	100.1
AVG				90.8	93.9	95.7	96.5	97.1	98.3	98.8
2-1	2.493	2.370	2.446	88.8	92.0	93.7	94.5	95.1	97.6	98.1
2-2	2.493	2.435	2.444	91.1	94.5	96.3	97.1	97.7	97.5	98.0
2-3	2.493	2.421	2.445	90.7	93.9	95.7	96.5	97.1	97.6	98.1
AVG				90.2	93.5	95.2	96.0	96.6	97.6	98.1
3-1	2.493	2.427	2.440	91.0	94.2	95.9	96.8	97.4	97.4	97.9
3-2	2.493	2.426	2.449	90.6	94.1	95.9	96.7	97.3	97.7	98.2
3-3	2.493	2.426	2.446	90.9	94.1	95.9	96.7	97.3	97.8	98.1
AVG				90.8	94.1	95.9	96.7	97.3	97.6	98.1

TABLE A.6 SGC Data for Project AL-6

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.548	2.479	2.488	91.2	94.4	96.1	96.9	97.3	97.3	97.6
1-2	2.548	2.478	2.482	91.1	94.3	96.0	96.8	97.3	97.0	97.4
1-3	2.548	2.478	2.489	91.0	94.3	96.0	96.8	97.3	97.4	97.7
AVG				91.1	94.3	96.0	96.8	97.3	97.2	97.6
2-1	2.530	2.475	2.487	91.5	94.7	96.5	97.3	97.8	98.0	98.3
2-2	2.530	2.470	2.482	91.1	94.5	96.3	97.1	97.6	97.8	98.1
2-3	2.530	2.472	2.485	91.2	94.5	96.3	97.2	97.7	97.9	98.2
AVG				91.3	94.6	96.3	97.2	97.7	97.9	98.2
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.548	2.450	2.471	90.0	93.1	94.8	95.6	96.2	96.5	97.0
1-2	2.548	2.456	2.474	90.3	93.3	95.1	95.9	96.4	96.7	97.1
1-3	2.548	2.454	2.465	90.1	93.2	94.9	95.8	96.3	96.3	96.7
AVG				90.1	93.2	94.9	95.8	96.3	96.5	96.9
2-1	2.530	2.450	2.469	90.5	93.7	95.4	96.2	96.8	97.2	97.6
2-2	2.530	2.450	2.467	90.4	93.6	95.4	96.2	96.8	97.1	97.5
2-3	2.530	2.448	2.468	90.5	93.6	95.4	96.2	96.8	97.1	97.5
AVG				90.5	93.6	95.4	96.2	96.8	97.1	97.5
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.7 SGC Data for Project AR-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.437	2.325	2.347	85.2	90.0	92.8	94.5	95.4	95.5	96.3
1-2	2.437	2.311	2.361	84.6	89.4	92.3	93.8	94.8	96.2	96.9
1-3	2.437	2.307	2.331	84.6	89.4	92.2	93.7	94.7	94.9	95.7
AVG				84.8	89.6	92.4	94.0	95.0	95.5	96.3
2-1	2.429	2.363	2.378	86.7	91.8	94.7	96.3	97.3	97.2	97.9
2-2	2.429	2.353	2.380	86.4	91.3	94.3	95.9	96.9	97.3	98.0
2-3	2.429	2.361	0.000	86.8	91.8	94.7	96.2	97.2	0.0	0.0
AVG				86.6	91.6	94.6	96.1	97.1	97.3	97.9
3-1	2.436	2.350	2.370	86.0	91.0	93.9	95.5	96.5	96.5	97.3
3-2	2.436	2.351	2.371	86.1	91.1	94.0	95.5	96.5	96.5	97.3
3-3	2.436	2.334	2.370	85.5	90.5	93.3	94.8	95.8	96.5	97.3
AVG				85.9	90.8	93.7	95.3	96.3	96.5	97.3

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.437	2.309	2.317	84.5	89.4	92.1	93.7	94.7	94.2	95.1
1-2	2.437	2.326	2.330	85.1	89.9	92.8	94.4	95.4	94.8	95.6
1-3	2.437	2.263	2.340	82.2	87.3	90.2	91.8	92.9	95.2	96.0
AVG				84.0	88.8	91.7	93.3	94.4	94.7	95.6
2-1	2.429	2.341	2.363	85.6	90.7	93.6	95.3	96.4	96.5	97.3
2-2	2.429	2.314	2.352	84.9	89.7	92.6	94.1	95.3	96.1	96.8
2-3	2.429	2.345	2.338	85.8	91.1	94.0	95.5	96.5	95.5	96.3
AVG				85.5	90.5	93.4	95.0	96.1	96.0	96.8
3-1	2.436	2.325	2.380	84.8	89.8	92.7	94.3	95.4	96.9	97.7
3-2	2.436	2.329	2.340	84.9	90.0	93.0	94.5	95.6	95.2	96.1
3-3	2.436	2.330	2.364	85.1	90.2	93.1	94.6	95.6	96.4	97.0
AVG				84.9	90.0	92.9	94.5	95.6	96.1	96.9

TABLE A.8 SGC Data for Project AR-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.464	2.348	2.379	85.4	90.0	92.8	94.3	95.3	95.8	96.6
1-2	2.464	2.342	2.367	85.0	89.8	92.5	94.0	95.0	95.3	96.1
1-3	2.464	2.373	2.375	86.0	90.8	93.8	95.4	96.3	95.6	96.4
AVG				85.4	90.2	93.0	94.6	95.5	95.6	96.3
2-1	2.448	2.344	2.378	84.9	89.9	93.0	94.7	95.8	96.3	97.1
2-2	2.448	2.348	2.383	85.2	90.3	93.2	94.9	95.9	96.6	97.3
2-3	2.448	2.340	2.384	85.0	90.0	93.0	94.6	95.6	96.6	97.4
AVG				85.0	90.1	93.1	94.7	95.8	96.5	97.3
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.464	2.340	2.362	84.5	89.4	92.3	93.9	95.0	95.0	95.9
1-2	2.464	2.340	2.363	84.4	89.3	92.3	93.9	95.0	95.1	95.9
1-3	2.464	2.327	2.356	84.0	88.8	91.7	93.3	94.4	94.8	95.6
AVG				84.3	89.2	92.1	93.7	94.8	95.0	95.8
2-1	2.448	2.328	2.353	84.3	89.4	92.3	94.0	95.1	95.3	96.1
2-2	2.448	2.340	2.360	84.7	89.8	92.8	94.5	95.6	95.5	96.4
2-3	2.448	2.332	2.370	84.3	89.4	92.5	94.2	95.3	96.0	96.8
AVG				84.4	89.5	92.5	94.2	95.3	95.6	96.4
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.9 SGC Data for Project AR-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.426	2.322	2.323	92.6	93.6	94.6	95.2	95.7	95.0	95.8
1-2	2.426	2.296	2.343	84.6	89.4	92.1	93.6	94.6	95.9	96.6
1-3	2.426	2.309	2.329	85.1	89.9	92.7	94.2	95.2	95.2	96.0
AVG				87.4	91.0	93.1	94.4	95.2	95.3	96.1
2-1	2.436	2.338	2.359	85.5	90.6	93.5	95.0	96.0	96.1	96.8
2-2	2.436	2.313	2.343	84.9	89.7	92.5	94.0	95.0	95.5	96.2
2-3	2.436	2.326	0.000	85.4	90.2	93.0	94.5	95.5	0.0	0.0
AVG				85.2	90.2	93.0	94.5	95.5	95.8	96.5
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.426	2.280	2.312	83.6	88.5	91.3	92.9	94.0	94.5	95.3
1-2	2.426	2.289	2.310	83.7	88.7	91.7	93.3	94.4	94.4	95.2
1-3	2.426	2.279	2.316	83.9	88.7	91.5	92.9	93.9	94.8	95.5
AVG				83.7	88.6	91.5	93.0	94.1	94.6	95.3
2-1	2.436	2.331	2.337	85.2	90.2	93.1	94.7	95.7	#DIV/0!	95.9
2-2	2.436	2.321	2.354	84.8	89.8	92.7	94.2	95.3	#DIV/0!	96.6
2-3	2.436	2.325	0.000	84.8	90.0	92.9	94.4	95.4	#DIV/0!	0.0
AVG				85.0	90.0	92.9	94.4	95.5	#DIV/0!	96.3
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.10 SGC Data for Project AR-4

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.409	2.251	2.302	85.4	89.1	91.3	92.6	93.4	95.0	95.6
1-2	2.409	2.243	2.298	85.1	88.9	91.1	92.3	93.1	94.7	95.4
1-3	2.409	2.254	2.293	85.6	89.4	91.6	92.8	93.6	94.6	95.2
AVG				85.4	89.1	91.3	92.6	93.4	94.8	95.4
2-1	2.392	2.253	2.294	85.9	89.8	92.1	93.3	94.2	95.2	95.9
2-2	2.392	2.266	2.296	86.6	90.4	92.7	93.9	94.7	95.3	96.0
2-3	2.392	2.255	2.287	85.9	89.8	92.1	93.4	94.3	94.9	95.6
AVG				86.1	90.0	92.3	93.5	94.4	95.2	95.8
3-1	2.401	2.261	2.295	85.9	89.8	92.1	93.4	94.2	94.9	95.6
3-2	2.401	2.275	2.295	86.4	90.4	92.6	94.0	94.8	94.9	95.6
3-3	2.401	2.263	2.298	85.9	89.9	92.2	93.5	94.3	95.1	95.7
AVG				86.0	90.0	92.3	93.6	94.4	95.0	95.6

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.409	2.276	2.277	86.1	90.2	92.4	93.6	94.5	93.9	94.5
1-2	2.409	2.274	2.285	85.8	89.9	92.3	93.6	94.4	94.3	94.9
1-3	2.409	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				86.0	90.1	92.4	93.6	94.4	94.1	94.7
2-1	2.392	2.272	2.278	86.3	90.4	92.8	94.1	95.0	94.6	95.2
2-2	2.392	2.274	2.283	86.1	90.4	92.8	94.2	95.1	94.8	95.4
2-3	2.392	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				86.2	90.4	92.8	94.1	95.0	94.7	95.3
3-1	2.401	2.283	2.284	86.5	90.5	92.8	94.2	95.1	94.6	95.1
3-2	2.401	2.286	2.320	86.7	90.7	93.0	94.3	95.2	95.9	96.6
3-3	2.401	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				86.6	90.6	92.9	94.2	95.1	95.2	95.9

TABLE A.11 SGC Data for Project CO-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.451	2.431	2.451	91.7	95.5	97.6	98.6	99.2	99.6	100.0
1-2	2.451	2.417	2.454	91.2	95.1	97.1	98.1	98.6	99.6	100.1
1-3	2.451	2.433	2.443	91.1	95.2	97.5	98.6	99.3	99.3	99.7
AVG				91.3	95.3	97.4	98.4	99.0	99.5	99.9
2-1	2.436	2.444	2.454	93.4	97.3	99.3	100.0	100.3	100.4	100.7
2-2	2.436	2.435	2.454	92.4	97.0	98.8	99.6	100.0	100.5	100.7
2-3	2.436	2.444	2.451	92.7	96.8	98.8	99.8	100.3	100.2	100.6
AVG				92.8	97.0	99.0	99.8	100.2	100.3	100.7
3-1	2.450	2.429	2.431	92.3	96.2	98.1	98.8	99.1	99.1	99.2
3-2	2.450	2.429	2.437	92.0	96.2	98.0	98.8	99.1	99.3	99.5
3-3	2.450	2.431	2.437	92.3	96.2	98.1	98.9	99.2	99.3	99.5
AVG				92.2	96.2	98.1	98.8	99.2	99.2	99.4

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.451	2.409	2.424	89.2	92.8	94.9	97.7	98.3	98.4	98.9
1-2	2.451	2.394	2.427	88.6	92.2	94.3	97.1	97.7	98.4	99.0
1-3	2.451	2.407	2.436	90.7	94.5	96.6	97.6	98.2	98.8	99.4
AVG				89.5	93.2	95.2	97.4	98.1	98.5	99.1
2-1	2.436	2.421	2.441	91.7	95.4	97.5	98.7	99.4	99.8	100.2
2-2	2.436	2.424	2.464	91.7	95.7	97.8	98.9	99.5	100.7	101.1
2-3	2.436	2.425	2.437	92.0	95.9	98.0	98.9	99.5	99.7	100.0
AVG				91.8	95.7	97.8	98.8	99.5	100.1	100.5
3-1	2.450	2.405	2.426	91.0	94.6	96.7	97.6	98.2	98.8	99.0
3-2	2.450	2.407	2.427	91.1	94.9	96.9	97.7	98.2	98.7	99.1
3-3	2.450	2.416	2.426	91.4	95.1	97.1	98.0	98.6	98.7	99.0
AVG				91.1	94.9	96.9	97.8	98.3	98.7	99.0

TABLE A.12 SGC Data for Project CO-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.428	2.425	2.428	92.8	97.0	98.9	99.6	99.9	99.9	100.0
1-2	2.428	2.417	2.423	92.4	96.7	98.5	99.2	99.5	99.6	99.8
1-3	2.428	2.421	2.417	93.0	97.3	99.0	99.5	99.7	99.4	99.5
AVG				92.7	97.0	98.8	99.5	99.7	99.6	99.8
2-1	2.449	2.431	2.445	91.4	95.7	97.8	98.7	99.3	99.6	99.8
2-2	2.449	2.431	2.452	91.6	95.8	97.9	98.7	99.3	99.9	100.1
2-3	2.449	2.433	2.448	91.6	95.7	97.8	98.7	99.3	99.8	100.0
AVG				91.5	95.7	97.8	98.7	99.3	99.7	100.0
3-1	2.449	2.434	2.438	91.7	95.9	98.0	99.0	99.4	99.5	99.6
3-2	2.449	2.419	2.447	91.1	95.3	97.4	98.3	98.8	99.7	99.9
3-3	2.449	2.436	2.446	92.0	96.2	98.3	99.1	99.5	99.8	99.9
AVG				91.6	95.8	97.9	98.8	99.2	99.7	99.8

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.428	2.409	2.419	91.8	95.8	97.8	98.7	99.2	99.5	99.6
1-2	2.428	2.407	2.398	91.6	95.6	97.7	98.5	99.1	98.7	98.8
1-3	2.428	2.411	2.393	91.9	95.9	97.9	98.9	99.3	98.4	98.6
AVG				91.8	95.8	97.8	98.7	99.2	98.8	99.0
2-1	2.449	2.427	2.438	91.1	95.0	97.3	98.4	99.1	99.1	99.6
2-2	2.449	2.421	2.423	90.6	94.8	97.1	98.2	98.9	97.7	98.9
2-3	2.449	2.416	2.437	90.6	94.6	96.9	97.9	98.7	99.2	99.5
AVG				90.8	94.8	97.1	98.1	98.9	98.7	99.3
3-1	2.449	2.410	2.427	90.7	94.7	96.8	97.8	98.4	98.8	99.1
3-2	2.449	2.420	2.426	90.9	94.9	97.0	98.1	98.8	98.6	99.1
3-3	2.449	2.409	2.429	90.7	94.6	96.8	97.8	98.4	98.7	99.2
AVG				90.8	94.7	96.9	97.9	98.5	98.7	99.1

TABLE A.13 SGC Data for Project CO-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.427	2.326	2.398	87.2	91.4	93.8	95.1	95.8	98.3	98.8
1-2	2.427	2.369	2.386	88.5	93.1	95.6	96.9	97.6	97.9	98.3
1-3	2.427	2.366	2.392	88.6	93.2	95.7	96.8	97.5	98.0	98.6
AVG				88.1	92.6	95.0	96.3	97.0	98.1	98.6
2-1	2.435	2.372	2.396	88.5	92.9	95.4	96.7	97.4	97.9	98.4
2-2	2.435	2.364	2.397	88.5	92.9	95.2	96.4	97.1	98.0	98.4
2-3	2.435	2.379	2.395	88.6	93.2	95.8	97.0	97.7	97.9	98.4
AVG				88.5	93.0	95.5	96.7	97.4	97.9	98.4
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.427	2.338	2.367	87.6	91.8	94.2	95.5	96.3	96.9	97.5
1-2	2.427	2.335	2.369	87.4	91.6	94.1	95.4	96.2	97.0	97.6
1-3	2.427	2.335	2.373	87.6	91.8	94.2	95.4	96.2	97.2	97.8
AVG				87.5	91.7	94.2	95.4	96.3	97.0	97.6
2-1	2.435	2.362	2.383	88.1	92.4	95.0	96.2	97.0	97.3	97.9
2-2	2.435	2.342	2.389	87.4	91.7	94.1	95.4	96.2	97.5	98.1
2-3	2.435	2.368	2.387	87.9	92.5	95.2	96.4	97.2	97.4	98.0
AVG				87.8	92.2	94.8	96.0	96.8	97.4	98.0
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.14 SGC Data for Project CO-4

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.501	2.445	2.485	89.2	93.3	95.7	96.9	97.8	98.7	99.4
1-2	2.501	2.453	2.484	89.1	93.5	96.0	97.2	98.1	98.8	99.3
1-3	2.501	2.440	2.485	89.3	93.4	95.7	96.9	97.6	98.7	99.4
AVG				89.2	93.4	95.8	97.0	97.8	98.8	99.3
2-1	2.497	2.452	2.475	89.6	93.8	96.1	97.4	98.2	98.5	99.1
2-2	2.497	2.453	2.473	89.6	93.9	96.3	97.5	98.2	98.4	99.0
2-3	2.497	2.456	2.469	89.7	94.0	96.4	97.6	98.4	98.3	98.9
AVG				89.6	93.9	96.3	97.5	98.3	98.4	99.0
3-1	2.510	2.448	2.470	88.3	92.7	95.3	96.7	97.5	97.8	98.4
3-2	2.510	2.430	2.467	87.7	92.0	94.5	95.9	96.8	97.7	98.3
3-3	2.510	2.444	2.466	88.2	92.6	95.2	96.5	97.4	97.5	98.2
AVG				88.1	92.4	95.0	96.4	97.2	97.7	98.3

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.501	2.425	2.455	88.4	92.5	94.9	96.1	97.0	97.5	98.2
1-2	2.501	2.424	2.455	88.2	92.2	94.8	96.0	96.9	97.5	98.2
1-3	2.501	2.415	2.447	88.0	92.1	94.5	95.7	96.6	97.1	97.8
AVG				88.2	92.3	94.7	95.9	96.8	97.4	98.1
2-1	2.497	2.415	2.442	88.4	92.2	94.6	95.9	96.7	97.1	97.8
2-2	2.497	2.424	2.455	88.7	92.7	95.1	96.3	97.1	97.6	98.3
2-3	2.497	2.414	2.445	88.0	92.1	94.5	95.8	96.7	97.2	97.9
AVG				88.4	92.3	94.7	96.0	96.8	97.3	98.0
3-1	2.510	2.416	2.453	87.5	91.6	94.1	95.3	96.3	97.1	97.7
3-2	2.510	2.427	2.442	87.7	91.9	94.5	95.8	96.7	96.6	97.3
3-3	2.510	2.420	2.434	87.6	91.7	94.2	95.5	96.4	96.3	97.0
AVG				87.6	91.7	94.3	95.5	96.5	96.7	97.3

TABLE A.15 SGC Data for Project CO-5

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.451	2.365	2.404	88.1	92.1	94.5	95.7	96.5	97.5	98.1
1-2	2.451	2.358	2.413	87.9	92.0	94.2	95.5	96.2	97.8	98.4
1-3	2.451	2.380	2.409	88.6	92.8	95.0	96.3	97.1	97.6	98.3
AVG				88.2	92.3	94.6	95.8	96.6	97.6	98.3
2-1	2.462	2.396	2.418	88.6	92.8	95.3	96.5	97.3	97.6	98.2
2-2	2.462	2.397	2.425	88.6	92.9	95.3	96.6	97.4	98.0	98.5
2-3	2.462	2.399	2.423	88.7	93.0	95.4	96.7	97.4	97.8	98.4
AVG				88.6	92.9	95.3	96.6	97.4	97.8	98.4
3-1	2.462	2.401	2.418	88.5	92.9	95.4	96.7	97.5	97.6	98.2
3-2	2.462	2.393	2.417	88.3	92.6	95.1	96.4	97.2	97.5	98.2
3-3	2.462	2.391	2.421	88.2	92.6	95.0	96.3	97.1	97.7	98.3
AVG				88.3	92.7	95.1	96.4	97.3	97.6	98.2

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.451	2.340	2.369	87.6	91.3	93.5	94.7	95.5	96.0	96.7
1-2	2.451	2.332	2.367	87.1	91.0	93.2	94.3	95.1	96.0	96.6
1-3	2.451	2.338	2.369	87.2	91.1	93.3	94.5	95.4	96.0	96.7
AVG				87.3	91.1	93.3	94.5	95.3	96.0	96.6
2-1	2.462	2.352	2.397	87.5	91.3	93.5	94.7	95.5	96.7	97.4
2-2	2.462	2.363	2.387	87.5	91.6	94.0	95.2	96.0	96.4	97.0
2-3	2.462	2.360	2.389	87.5	91.4	93.8	95.0	95.9	96.4	97.0
AVG				87.5	91.4	93.8	95.0	95.8	96.5	97.1
3-1	2.462	2.358	2.384	87.3	91.3	93.7	94.9	95.8	96.2	96.8
3-2	2.462	2.361	2.371	87.2	91.4	93.8	95.1	95.9	95.6	96.3
3-3	2.462	2.361	2.386	87.5	91.5	93.8	95.0	95.9	96.2	96.9
AVG				87.4	91.4	93.8	95.0	95.9	96.0	96.7

TABLE A.16 SGC Data for Project FL-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.460	2.359	2.362	88.3	92.1	94.2	95.3	95.9	95.5	96.0
1-2	2.460	2.291	2.354	85.6	89.4	91.4	92.5	93.1	95.1	95.7
1-3	2.460	2.346	2.390	87.6	91.5	93.6	94.7	95.4	96.6	97.2
AVG				87.1	91.0	93.1	94.1	94.8	95.7	96.3
2-1	2.450	2.359	2.382	88.0	92.2	94.4	95.5	96.3	96.6	97.2
2-2	2.450	2.363	2.392	88.1	92.3	94.5	95.7	96.4	97.1	97.6
2-3	2.450	2.362	2.390	88.1	92.2	94.5	95.6	96.4	97.0	97.6
AVG				88.1	92.2	94.5	95.6	96.4	96.9	97.5
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.460	2.290	2.304	85.2	89.0	91.2	92.4	93.1	93.1	93.7
1-2	2.460	2.295	2.322	85.4	89.3	91.5	92.6	93.3	93.8	94.4
1-3	2.460	2.328	2.358	86.6	90.5	92.8	93.8	94.6	95.3	95.9
AVG				85.7	89.6	91.8	92.9	93.7	94.1	94.6
2-1	2.450	2.325	2.357	87.2	91.0	93.2	94.2	94.9	95.5	96.2
2-2	2.450	2.329	2.364	87.3	91.1	93.2	94.3	95.1	95.9	96.5
2-3	2.450	2.343	2.326	87.6	91.6	93.8	94.8	95.6	94.4	94.9
AVG				87.4	91.2	93.4	94.5	95.2	95.3	95.9
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.17 SGC Data for Project GA-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.540	2.478	2.501	91.4	94.6	96.3	97.1	97.6	98.1	98.5
1-2	2.540	2.483	2.509	91.6	94.8	96.4	97.3	97.8	98.4	98.8
1-3	2.540	2.482	2.506	91.3	94.6	96.3	97.2	97.7	98.2	98.7
AVG				91.4	94.7	96.4	97.2	97.7	98.3	98.6
2-1	2.520	2.485	2.506	91.7	95.2	97.1	98.0	98.6	99.1	99.4
2-2	2.520	2.496	2.505	92.3	95.8	97.7	98.5	99.0	99.1	99.4
2-3	2.520	2.499	2.505	92.4	96.0	97.8	98.7	99.2	99.0	99.4
AVG				92.1	95.7	97.5	98.4	98.9	99.1	99.4
3-1	2.537	2.498	2.497	91.9	95.4	97.1	98.0	98.5	98.0	98.4
3-2	2.537	2.527	2.500	93.2	96.6	98.3	99.1	99.6	98.1	98.5
3-3	2.537	2.490	2.504	91.2	94.8	96.7	97.6	98.1	98.4	98.7
AVG				92.1	95.6	97.4	98.2	98.7	98.2	98.6

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.540	2.471	2.448	91.1	94.0	95.7	96.6	97.3	96.0	96.4
1-2	2.540	2.489	2.493	91.5	94.8	96.5	97.4	98.0	97.7	98.1
1-3	2.540	2.476	2.495	90.8	94.3	96.0	96.9	97.5	97.8	98.2
AVG				91.1	94.4	96.1	97.0	97.6	97.2	97.6
2-1	2.520	2.482	2.504	91.7	95.2	97.0	97.9	98.5	99.0	99.4
2-2	2.520	2.480	2.517	91.5	95.0	96.8	97.8	98.4	99.4	99.9
2-3	2.520	2.485	2.501	91.6	95.3	97.1	98.0	98.6	98.8	99.2
AVG				91.6	95.2	97.0	97.9	98.5	99.1	99.5
3-1	2.537	2.477	2.479	90.5	94.2	96.0	97.1	97.6	97.3	97.7
3-2	2.537	2.474	2.498	90.9	94.3	96.1	96.9	97.5	98.0	98.5
3-3	2.537	2.484	2.520	91.1	94.6	96.4	97.3	97.9	98.8	99.3
AVG				90.8	94.3	96.2	97.1	97.7	98.1	98.5

TABLE A.18 SGC Data for Project IL-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.502	2.383	2.435	84.1	89.2	92.4	94.1	95.2	96.5	97.3
1-2	2.502	2.381	2.424	84.0	89.1	92.3	94.0	95.2	96.0	96.9
1-3	2.502	2.384	2.437	84.1	89.3	92.4	94.1	95.3	96.5	97.4
AVG				84.1	89.2	92.4	94.1	95.2	96.3	97.2
2-1	2.499	2.415	2.439	85.0	90.4	93.7	95.6	96.6	96.7	97.6
2-2	2.499	2.404	2.443	84.7	90.1	93.3	95.1	96.2	96.9	97.8
2-3	2.499	2.403	2.446	84.5	89.9	93.2	95.0	96.2	96.9	97.9
AVG				84.7	90.1	93.4	95.2	96.3	96.9	97.7
3-1	2.491	2.398	2.439	84.5	89.9	93.3	95.1	96.3	97.1	97.9
3-2	2.491	2.402	2.431	84.6	90.1	93.4	95.3	96.4	96.7	97.6
3-3	2.491	2.387	2.440	84.2	89.6	92.9	94.7	95.8	97.0	98.0
AVG				84.4	89.9	93.2	95.0	96.2	96.9	97.8

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.502	2.388	2.428	85.1	90.0	92.8	94.4	95.4	96.3	97.0
1-2	2.502	2.396	2.418	84.2	89.7	92.9	94.6	95.8	95.7	96.6
1-3	2.502	2.375	2.422	83.5	88.9	92.0	93.8	94.9	95.9	96.8
AVG				84.3	89.5	92.6	94.3	95.4	96.0	96.8
2-1	2.499	2.383	2.417	84.1	89.3	93.3	94.2	95.4	95.9	96.7
2-2	2.499	2.384	2.436	84.0	89.4	93.3	94.2	95.4	96.6	97.5
2-3	2.499	2.389	2.423	84.2	89.5	92.7	94.4	95.6	96.1	97.0
AVG				84.1	89.4	93.1	94.3	95.5	96.2	97.1
3-1	2.491	2.379	2.423	84.2	89.4	92.6	94.3	95.5	96.4	97.3
3-2	2.491	2.385	2.424	84.2	89.5	93.5	94.5	95.7	96.5	97.3
3-3	2.491	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				84.2	89.5	93.1	94.4	95.6	96.4	97.3

TABLE A.19 SGC Data for Project IL-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.446	2.345	2.370	84.8	89.8	93.1	94.7	95.9	96.0	96.9
1-2	2.446	2.338	2.387	84.7	89.6	92.8	94.4	95.6	96.8	97.6
1-3	2.446	2.343	2.377	84.9	89.9	93.0	94.7	95.8	96.3	97.2
AVG				84.8	89.8	92.9	94.6	95.7	96.4	97.2
2-1	2.428	2.372	2.401	86.4	91.7	94.9	96.6	97.7	98.1	98.9
2-2	2.428	2.366	2.395	86.2	91.3	94.6	96.4	97.4	97.8	98.6
2-3	2.428	2.376	2.385	86.7	91.9	95.1	96.8	97.9	97.5	98.2
AVG				86.4	91.6	94.8	96.6	97.7	97.8	98.6
3-1	2.433	2.370	2.405	86.0	91.2	94.5	96.3	97.4	98.1	98.8
3-2	2.433	2.376	2.409	86.4	91.6	94.8	96.6	97.7	98.2	99.0
3-3	2.433	2.382	2.402	86.7	91.9	95.1	96.8	97.9	97.9	98.7
AVG				86.3	91.6	94.8	96.6	0.0	98.1	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.446	2.363	2.393	85.0	90.4	93.6	95.5	96.6	96.9	97.8
1-2	2.446	2.354	2.389	84.8	90.2	93.3	95.0	96.2	96.8	97.7
1-3	2.446	2.353	2.385	84.7	90.0	93.3	95.0	96.2	96.6	97.5
AVG				84.8	90.2	93.4	95.2	96.3	96.8	97.7
2-1	2.428	2.378	0.000	86.5	91.8	95.0	96.7	97.9	#DIV/0!	0.0
2-2	2.428	2.369	0.000	86.1	91.4	94.5	96.3	97.6	#DIV/0!	0.0
2-3	2.428	2.374	2.391	86.8	91.7	94.9	96.7	97.8	97.6	98.5
AVG				86.5	91.6	94.8	96.6	0.0	#DIV/0!	0.0
3-1	2.433	2.381	2.403	86.3	91.7	94.9	96.7	97.9	97.9	98.8
3-2	2.433	2.378	2.404	86.3	91.6	94.8	96.5	97.7	98.0	98.8
3-3	2.433	2.383	2.403	86.3	91.7	95.0	96.7	97.9	97.9	98.8
AVG				86.3	91.7	94.9	96.6	0.0	97.9	0.0

TABLE A.20 SGC Data for Project IL-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.505	2.353	2.396	83.9	88.6	91.3	92.9	93.9	94.9	95.6
1-2	2.505	2.336	2.409	83.5	88.1	90.8	92.3	93.3	95.3	96.2
1-3	2.505	2.361	2.400	84.0	88.8	91.7	93.2	94.3	95.0	95.8
AVG				83.8	88.5	91.3	92.8	93.8	95.1	95.9
2-1	2.493	2.377	2.404	84.4	89.4	92.6	94.2	95.3	95.5	96.4
2-2	2.493	2.367	2.386	84.3	89.1	92.1	93.8	94.9	94.9	95.7
2-3	2.493	2.365	2.396	84.3	89.1	92.2	93.8	94.9	95.3	96.1
AVG				84.3	89.2	92.3	93.9	95.1	95.2	96.1
3-1	2.493	2.365	2.404	83.8	88.8	92.0	93.7	94.9	95.7	96.4
3-2	2.493	2.359	2.393	83.6	88.6	91.7	93.5	94.6	95.1	96.0
3-3	2.493	2.352	2.394	83.5	88.4	91.5	93.2	94.3	95.1	96.0
AVG				83.6	88.6	91.7	93.5	94.6	95.3	96.1

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.505	2.379	2.407	84.3	89.2	92.2	93.8	95.0	95.3	96.1
1-2	2.505	2.367	2.409	84.1	88.9	91.8	93.4	94.5	95.3	96.2
1-3	2.505	2.370	2.403	84.3	89.1	91.9	93.5	94.6	95.0	95.9
AVG				84.2	89.1	92.0	93.6	94.7	95.2	96.1
2-1	2.493	2.381	2.407	84.3	89.5	92.5	94.3	95.5	95.6	96.6
2-2	2.493	2.370	2.412	84.2	89.2	92.3	93.9	95.1	95.8	96.8
2-3	2.493	2.371	2.405	84.1	89.1	92.2	93.9	95.1	95.6	96.5
AVG				84.2	89.3	92.3	94.0	95.2	95.7	96.6
3-1	2.493	2.370	2.411	83.6	88.8	92.0	93.8	95.1	95.7	96.7
3-2	2.493	2.368	2.414	83.5	88.7	91.9	93.8	95.0	95.9	96.8
3-3	2.493	2.383	2.418	84.1	89.4	92.6	94.4	95.6	96.1	97.0
AVG				83.7	89.0	92.2	94.0	95.2	95.9	96.8

TABLE A.21 SGC Data for Project IN-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.465	2.358	2.411	84.8	89.8	92.8	94.5	95.7	96.9	97.8
1-2	2.465	2.338	2.406	84.0	89.0	92.1	93.8	94.8	96.8	97.6
1-3	2.465	2.375	2.404	85.0	90.3	93.5	95.2	96.3	96.8	97.5
AVG				84.6	89.7	92.8	94.5	95.6	96.8	97.6
2-1	2.469	2.407	2.443	86.0	91.4	94.6	96.3	97.5	98.1	98.9
2-2	2.469	2.408	2.445	86.1	91.5	94.7	96.4	97.5	98.2	99.0
2-3	2.469	2.407	2.445	86.1	91.5	94.6	96.4	97.5	98.2	99.0
AVG				86.1	91.5	94.6	96.4	97.5	98.2	99.0
3-1	2.471	2.409	2.443	86.2	91.5	94.6	96.4	97.5	98.1	98.9
3-2	2.471	2.405	2.445	85.9	91.3	94.5	96.2	97.3	98.1	98.9
3-3	2.471	2.408	2.446	86.1	91.5	94.7	96.3	97.5	98.1	99.0
AVG				86.1	91.4	94.6	96.3	97.4	98.1	98.9

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.465	2.312	2.361	83.8	88.1	91.1	92.6	93.8	94.9	95.8
1-2	2.465	2.327	2.357	84.0	88.7	91.6	93.2	94.4	94.7	95.6
1-3	2.465	2.321	2.351	83.6	88.4	91.4	93.0	94.2	94.5	95.4
AVG				83.8	88.4	91.4	93.0	94.1	94.7	95.6
2-1	2.469	2.357	2.396	85.0	89.7	92.8	94.4	95.5	96.2	97.0
2-2	2.469	2.352	2.396	84.8	89.5	92.5	94.1	95.3	96.2	97.0
2-3	2.469	2.346	2.394	84.2	89.2	92.4	94.0	95.0	96.0	97.0
AVG				84.7	89.5	92.6	94.2	95.2	96.1	97.0
3-1	2.471	2.349	2.389	84.6	89.4	92.4	93.9	95.1	95.8	96.7
3-2	2.471	2.354	2.397	84.7	89.6	92.6	94.1	95.3	96.1	97.0
3-3	2.471	2.356	2.395	84.8	89.6	92.6	94.2	95.3	96.0	96.9
AVG				84.7	89.5	92.6	94.1	95.2	96.0	96.9

TABLE A.22 SGC Data for Project IN-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.684	2.575	2.620	88.3	91.9	94.0	95.2	95.9	97.0	97.6
1-2	2.684	2.594	2.614	88.7	92.6	94.7	95.8	96.6	96.8	97.4
1-3	2.684	2.596	2.618	88.7	92.6	94.8	95.9	96.7	96.9	97.5
AVG				88.6	92.4	94.5	95.7	96.4	96.9	97.5
2-1	2.673	2.564	2.626	88.1	91.8	94.0	95.2	95.9	97.6	98.2
2-2	2.673	2.586	2.628	88.5	92.4	94.8	96.0	96.7	97.7	98.3
2-3	2.673	2.584	2.624	88.4	92.4	94.6	95.9	96.7	97.5	98.2
AVG				88.4	92.2	94.4	95.7	96.4	97.6	98.2
3-1	2.698	2.539	2.606	86.4	90.0	92.2	93.4	94.1	95.9	96.6
3-2	2.698	2.574	2.612	87.2	91.2	93.5	94.6	95.4	96.1	96.8
3-3	2.698	2.577	2.608	87.3	91.2	93.5	94.7	95.5	96.0	96.7
AVG				87.0	90.8	93.0	94.2	95.0	96.0	96.7

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.684	2.548	2.585	87.4	90.9	93.0	94.2	94.9	95.7	96.3
1-2	2.684	2.542	2.578	87.3	90.7	92.8	93.9	94.7	95.4	96.1
1-3	2.684	2.551	2.572	87.1	91.0	93.1	94.3	95.0	95.2	95.8
AVG				87.2	90.9	93.0	94.1	94.9	95.4	96.1
2-1	2.673	2.551	2.583	87.6	91.3	93.5	94.6	95.4	96.0	96.6
2-2	2.673	2.541	2.577	87.4	91.0	93.1	94.3	95.1	95.8	96.4
2-3	2.673	2.552	2.576	87.7	91.3	93.5	94.6	95.5	95.7	96.4
AVG				87.5	91.2	93.4	94.5	95.3	95.8	96.5
3-1	2.698	2.523	2.570	85.9	89.5	91.7	92.7	93.5	94.7	95.3
3-2	2.698	2.520	2.569	85.8	89.4	91.5	92.6	93.4	94.5	95.2
3-3	2.698	2.545	2.558	86.6	90.2	92.4	93.5	94.3	94.1	94.8
AVG				86.1	89.7	91.8	92.9	93.7	94.4	95.1

TABLE A.23 SGC Data for Project KS-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.435	2.308	2.344	86.7	90.5	92.8	94.0	94.8	95.7	96.3
1-2	2.435	2.308	2.410	86.7	90.6	92.8	94.0	94.8	98.4	99.0
1-3	2.435	2.305	2.345	86.6	90.5	92.8	93.9	94.7	95.7	96.3
AVG				86.7	90.6	92.8	94.0	94.7	96.6	97.2
2-1	2.421	2.340	2.336	88.5	92.5	94.8	95.9	96.7	95.9	96.5
2-2	2.421	2.335	2.339	88.3	92.3	94.5	95.7	96.4	96.0	96.6
2-3	2.421	2.338	2.365	88.3	92.5	94.7	95.8	96.6	97.2	97.7
AVG				88.4	92.4	94.7	95.8	96.6	96.4	96.9
3-1	2.413	2.315	2.340	87.5	91.7	94.0	95.2	95.9	96.5	97.0
3-2	2.413	2.316	2.337	87.6	91.7	94.0	95.2	96.0	96.3	96.9
3-3	2.413	2.308	2.328	87.6	91.6	93.8	94.9	95.6	95.9	96.5
AVG				87.6	91.7	94.0	95.1	95.9	96.2	96.8

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.435	2.316	2.324	86.7	90.8	93.1	94.3	95.1	94.9	95.4
1-2	2.435	2.292	2.326	86.0	89.9	92.2	93.3	94.1	95.0	95.5
1-3	2.435	2.296	2.335	85.9	90.0	92.2	93.5	94.3	95.3	95.9
AVG				86.2	90.2	92.5	93.7	94.5	95.0	95.6
2-1	2.421	2.323	2.331	87.5	91.7	93.9	95.1	96.0	95.7	96.3
2-2	2.421	2.324	2.328	87.8	91.8	94.1	95.2	96.0	95.6	96.2
2-3	2.421	2.305	2.333	86.8	91.0	93.1	94.4	95.2	95.7	96.4
AVG				87.4	91.5	93.7	94.9	95.7	95.7	96.3
3-1	2.413	2.302	2.315	87.0	91.1	93.4	94.6	95.4	95.4	95.9
3-2	2.413	2.287	2.317	86.4	90.5	92.7	94.0	94.8	95.4	96.0
3-3	2.413	2.291	2.317	86.5	90.5	92.8	94.1	94.9	95.4	96.0
AVG				86.6	90.7	93.0	94.2	95.0	95.4	96.0

TABLE A.24 SGC Data for Project KY-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.480	2.420	2.440	86.7	91.8	95.0	96.7	97.6	97.8	98.4
1-2	2.480	2.431	2.451	86.9	92.4	95.5	97.1	98.0	98.3	98.8
1-3	2.480	2.434	2.447	87.1	92.6	95.7	97.3	98.1	98.1	98.7
AVG				86.9	92.3	95.4	97.0	97.9	98.1	98.6
2-1	2.453	2.408	2.438	86.5	92.0	95.3	97.1	98.2	98.7	99.4
2-2	2.453	2.411	2.436	86.6	92.2	95.5	97.2	98.3	98.6	99.3
2-3	2.453	2.410	2.435	86.6	92.2	95.4	97.1	98.2	98.6	99.3
AVG				86.6	92.1	95.4	97.1	98.2	98.6	99.3
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.480	2.386	2.408	85.7	90.5	93.6	95.1	96.2	96.4	97.1
1-2	2.480	2.391	2.415	86.0	90.8	93.8	95.3	96.4	96.6	97.4
1-3	2.480	2.383	2.412	85.5	90.5	93.5	95.1	96.1	96.5	97.3
AVG				85.7	90.6	93.7	95.2	96.2	96.5	97.2
2-1	2.453	2.356	2.393	85.1	90.1	93.3	94.9	96.0	96.6	97.6
2-2	2.453	2.362	2.383	85.4	90.4	93.5	95.1	96.3	96.2	97.1
2-3	2.453	2.356	2.390	85.1	90.1	93.2	94.9	96.0	96.6	97.4
AVG				85.2	90.2	93.3	95.0	96.1	96.5	97.4
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.25 SGC Data for Project KY-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.488	2.313	2.366	81.3	86.7	90.0	91.7	93.0	94.1	95.1
1-2	2.488	2.319	2.369	81.6	87.0	90.3	92.1	93.2	94.3	95.2
1-3	2.488	2.329	2.373	81.8	87.2	90.6	92.4	93.6	94.4	95.4
AVG				81.6	87.0	90.3	92.1	93.3	94.3	95.2
2-1	2.470	2.412	2.438	85.1	91.0	94.6	96.5	97.7	98.0	98.7
2-2	2.470	2.409	2.441	85.0	91.0	94.6	96.4	97.5	98.1	98.8
2-3	2.470	2.412	2.438	85.2	91.1	94.7	96.5	97.7	97.9	98.7
AVG				85.1	91.0	94.6	96.5	97.6	98.0	98.7
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.488	2.277	2.313	80.5	85.5	88.7	90.3	91.5	92.0	93.0
1-2	2.488	2.279	2.313	80.4	85.4	88.7	90.4	91.6	92.0	93.0
1-3	2.488	2.278	2.311	80.4	85.4	88.6	90.3	91.6	92.0	92.9
AVG				80.4	85.4	88.7	90.4	91.6	92.0	92.9
2-1	2.470	2.359	2.384	83.9	89.1	92.5	94.3	95.5	95.7	96.5
2-2	2.470	2.362	2.388	84.0	89.3	92.6	94.4	95.6	95.8	96.7
2-3	2.470	2.346	2.390	83.9	88.9	92.2	93.8	95.0	95.8	96.8
AVG				83.9	89.1	92.4	94.2	95.4	95.7	96.7
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.26 SGC Data for Project KY-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.484	2.411	2.432	89.6	93.4	95.4	96.4	97.1	97.4	97.9
1-2	2.484	2.414	2.432	89.5	93.3	95.3	96.4	97.2	97.4	97.9
1-3	2.484	2.403	2.435	89.3	93.0	95.0	96.1	96.7	97.5	98.0
AVG				89.5	93.2	95.2	96.3	97.0	97.4	97.9
2-1	2.481	2.420	2.441	89.8	93.6	95.7	96.8	97.5	97.9	98.4
2-2	2.481	2.420	2.439	89.8	93.6	95.8	96.9	97.5	97.7	98.3
2-3	2.481	2.420	2.440	89.9	93.7	95.8	96.9	97.5	97.8	98.3
AVG				89.8	93.6	95.7	96.8	97.5	97.8	98.3
3-1	2.486	2.430	2.455	89.8	93.8	95.9	97.1	97.7	98.2	98.8
3-2	2.486	2.420	2.457	89.5	93.3	95.4	96.6	97.3	98.3	98.8
3-3	2.486	2.433	2.457	89.8	93.8	96.0	97.2	97.9	98.2	98.8
AVG				89.7	93.6	95.8	96.9	97.7	98.3	98.8

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.484	2.386	2.399	88.6	92.2	94.2	95.3	96.1	96.0	96.6
1-2	2.484	2.383	2.399	88.7	92.2	94.2	95.2	95.9	96.1	96.6
1-3	2.484	2.387	2.401	88.8	92.3	94.3	95.3	96.1	96.1	96.7
AVG				88.7	92.2	94.3	95.3	96.0	96.0	96.6
2-1	2.481	2.377	2.407	88.4	91.9	94.0	95.1	95.8	96.4	97.0
2-2	2.481	2.378	2.405	88.9	92.0	94.0	95.1	95.8	96.3	96.9
2-3	2.481	2.380	2.407	88.6	92.1	94.1	95.2	95.9	96.5	97.0
AVG				88.6	92.0	94.1	95.1	95.9	96.4	97.0
3-1	2.486	2.395	2.419	88.6	92.3	94.5	95.6	96.3	96.7	97.3
3-2	2.486	2.382	2.423	88.3	91.9	94.0	95.1	95.8	96.9	97.5
3-3	2.486	2.393	2.423	88.6	92.3	94.4	95.5	96.3	96.9	97.5
AVG				88.5	92.1	94.3	95.4	96.1	96.8	97.4

TABLE A.27 SGC Data for Project MI-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.478	2.340	2.387	83.9	88.9	91.9	93.4	94.4	95.5	96.3
1-2	2.478	2.353	2.393	84.0	89.2	92.3	93.9	95.0	95.7	96.6
1-3	2.478	2.357	2.385	84.0	89.4	92.4	94.1	95.1	95.4	96.2
AVG				84.0	89.2	92.2	93.8	94.8	95.6	96.4
2-1	2.472	2.355	2.406	84.4	89.7	92.7	94.3	95.3	96.6	97.3
2-2	2.472	2.367	2.390	84.8	90.0	93.2	94.8	95.8	95.9	96.7
2-3	2.472	2.372	2.445	84.9	90.2	93.3	94.9	96.0	98.2	98.9
AVG				84.7	90.0	93.1	94.7	95.7	96.9	97.6
3-1	2.497	2.367	2.421	83.8	89.0	92.1	93.8	94.8	96.2	97.0
3-2	2.497	2.364	2.404	83.7	88.9	92.0	93.6	94.7	95.4	96.3
3-3	2.497	2.376	2.400	84.2	89.4	92.5	94.1	95.2	95.0	96.1
AVG				83.9	89.1	92.2	93.8	94.9	95.5	96.4

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.478	2.298	2.336	82.7	87.4	90.3	91.7	92.7	93.5	94.3
1-2	2.478	2.298	2.342	82.5	87.3	90.2	91.7	92.7	93.8	94.5
1-3	2.478	2.307	2.339	82.8	87.6	90.6	92.0	93.1	93.6	94.4
AVG				82.7	87.4	90.4	91.8	92.9	93.6	94.4
2-1	2.472	2.307	2.366	82.9	87.8	90.8	92.3	93.3	94.9	95.7
2-2	2.472	2.328	2.370	83.6	88.5	91.5	93.1	94.2	95.0	95.9
2-3	2.472	2.325	2.364	83.5	88.4	91.5	93.0	94.1	94.8	95.6
AVG				83.3	88.3	91.3	92.8	93.9	94.9	95.7
3-1	2.497	2.337	2.351	83.2	88.0	91.0	92.5	93.6	93.3	94.2
3-2	2.497	2.334	2.353	83.0	87.9	90.8	92.4	93.5	93.5	94.2
3-3	2.497	2.324	2.363	82.8	87.6	90.5	92.0	93.1	93.9	94.6
AVG				83.0	87.8	90.8	92.3	93.4	93.6	94.3

TABLE A.28 SGC Data for Project MI-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.446	2.387	2.416	88.4	92.8	95.4	96.8	97.6	98.1	98.8
1-2	2.446	2.389	2.422	88.5	93.0	95.5	96.8	97.7	98.4	99.0
1-3	2.446	2.396	2.421	88.7	93.2	95.7	97.0	98.0	98.4	99.0
AVG				88.6	93.0	95.6	96.9	97.7	98.3	98.9
2-1	2.440	2.395	2.424	88.9	93.4	95.9	97.3	98.2	98.8	99.3
2-2	2.440	2.402	2.420	89.3	93.8	96.4	97.7	98.4	98.7	99.2
2-3	2.440	2.401	2.421	89.2	93.7	96.2	97.6	98.4	98.8	99.2
AVG				89.1	93.6	96.2	97.5	98.3	98.8	99.2
3-1	2.458	2.403	2.436	88.6	93.0	95.6	96.9	97.8	98.5	99.1
3-2	2.458	2.407	2.433	88.9	93.2	95.8	97.1	97.9	98.5	99.0
3-3	2.458	2.403	2.430	88.1	93.0	95.6	96.9	97.8	98.3	98.9
AVG				88.5	93.1	95.7	97.0	97.8	98.4	99.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.446	2.372	2.377	88.3	92.4	94.9	96.2	97.0	96.5	97.2
1-2	2.446	2.352	2.388	87.5	91.6	94.1	95.3	96.2	96.9	97.6
1-3	2.446	2.356	2.385	87.7	91.7	94.2	95.4	96.3	96.8	97.5
AVG				87.8	91.9	94.4	95.6	96.5	96.7	97.4
2-1	2.440	2.367	2.398	88.3	92.3	94.8	96.1	97.0	97.6	98.3
2-2	2.440	2.367	2.390	88.0	92.3	94.8	96.1	97.0	97.3	98.0
2-3	2.440	2.365	2.395	88.0	92.2	94.7	96.0	96.9	97.4	98.2
AVG				88.1	92.3	94.8	96.1	97.0	97.4	98.1
3-1	2.458	2.370	2.402	87.8	91.8	94.2	95.5	96.4	97.0	97.7
3-2	2.458	2.377	2.400	87.8	92.0	94.5	95.8	96.7	97.0	97.6
3-3	2.458	2.372	2.399	87.9	91.9	94.4	95.6	96.5	96.9	97.6
AVG				87.9	91.9	94.4	95.6	96.5	97.0	97.7

TABLE A.29 SGC Data for Project MI-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.468	2.391	2.412	89.5	93.2	95.1	96.2	96.9	97.2	97.7
1-2	2.468	2.397	2.421	89.7	93.4	95.4	96.4	97.1	97.7	98.1
1-3	2.468	2.390	2.418	89.3	93.0	95.1	96.2	96.8	97.5	98.0
AVG				89.5	93.2	95.2	96.3	96.9	97.4	97.9
2-1	2.466	2.378	2.410	89.4	92.8	94.7	95.8	96.4	97.2	97.7
2-2	2.466	2.390	2.414	89.7	93.3	95.3	96.2	96.9	97.5	97.9
2-3	2.466	2.394	2.416	89.9	93.5	95.4	96.4	97.1	97.5	98.0
AVG				89.6	93.2	95.1	96.1	96.8	97.4	97.9
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.468	2.363	2.387	88.6	92.1	94.0	95.1	95.7	96.1	96.7
1-2	2.468	2.359	2.388	88.4	91.9	93.9	94.9	95.6	96.2	96.8
1-3	2.468	2.360	2.384	88.4	91.8	93.9	94.9	95.6	96.0	96.6
AVG				88.4	91.9	94.0	95.0	95.7	96.1	96.7
2-1	2.466	2.361	2.381	88.8	92.2	94.1	95.0	95.7	96.0	96.6
2-2	2.466	2.357	2.380	88.7	92.0	94.0	94.9	95.6	96.0	96.5
2-3	2.466	2.359	2.383	88.7	92.0	94.0	94.9	95.7	96.1	96.6
AVG				88.7	92.1	94.0	94.9	95.7	96.1	96.6
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.30 SGC Data for Project MO-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.474	2.388	2.435	84.4	89.8	93.3	95.2	96.5	97.4	98.4
1-2	2.474	2.347	2.440	83.5	88.6	91.9	93.6	94.9	97.7	98.6
1-3	2.474	2.399	2.398	84.9	90.3	93.7	95.7	97.0	95.9	96.9
AVG				84.3	89.6	92.9	94.8	96.1	97.0	98.0
2-1	2.476	2.422	2.454	85.5	91.1	94.5	96.6	97.8	98.2	99.1
2-2	2.476	2.424	2.452	85.9	91.6	94.8	96.7	97.9	98.0	99.0
2-3	2.476	2.416	2.445	85.4	91.0	94.4	96.3	97.6	97.8	98.7
AVG				85.6	91.2	94.6	96.5	97.8	98.0	99.0
3-1	2.485	2.439	2.450	85.7	91.3	94.9	96.9	98.1	97.6	98.6
3-2	2.485	2.423	2.444	85.3	90.7	94.3	96.2	97.5	97.3	98.4
3-3	2.485	2.421	2.454	86.5	91.2	94.3	96.2	97.4	97.8	98.8
AVG				85.8	91.1	94.5	96.4	97.7	97.6	98.6

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.474	2.410	2.435	85.1	90.7	94.1	96.0	97.4	97.4	98.4
1-2	2.474	2.408	2.432	85.1	90.6	94.0	95.9	97.3	97.3	98.3
1-3	2.474	2.396	2.431	84.7	90.2	93.6	95.5	96.8	97.3	98.3
AVG				85.0	90.5	93.9	95.8	97.2	97.3	98.3
2-1	2.476	2.420	2.442	85.4	91.0	94.5	96.4	97.7	97.7	98.6
2-2	2.476	2.406	2.448	85.2	90.7	94.1	95.9	97.2	97.9	98.9
2-3	2.476	2.411	2.423	85.3	90.9	94.2	96.1	97.4	96.9	97.9
AVG				85.3	90.8	94.3	96.1	97.4	97.5	98.5
3-1	2.485	2.401	2.439	84.6	90.0	93.4	95.3	96.6	97.2	98.1
3-2	2.485	2.400	2.441	84.7	90.1	93.5	95.4	96.6	97.2	98.2
3-3	2.485	2.407	2.433	84.7	90.2	93.7	95.6	96.9	96.9	97.9
AVG				84.7	90.1	93.5	95.4	96.7	97.1	98.1

TABLE A.31 SGC Data for Project MO-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.360	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1-2	2.360	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1-3	2.360	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0
2-1	2.376	2.321	2.348	86.5	91.7	94.9	96.6	97.7	98.1	98.8
2-2	2.376	2.316	2.345	86.3	91.5	94.7	96.4	97.5	98.1	98.7
2-3	2.376	2.313	2.359	86.5	91.6	94.7	96.3	97.3	98.6	99.3
AVG				86.4	91.6	94.8	96.4	97.5	98.3	98.9
3-1	2.360	2.260	2.308	84.4	89.6	92.8	94.5	95.8	96.9	97.8
3-2	2.360	2.270	2.319	85.0	90.1	93.3	95.1	96.2	97.4	98.3
3-3	2.360	2.274	2.308	85.1	90.3	93.5	95.2	96.4	97.0	97.8
AVG				84.8	90.0	93.2	94.9	96.1	97.1	98.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.360	2.288	2.312	85.3	90.7	94.0	95.7	96.9	97.0	98.0
1-2	2.360	2.286	2.314	85.0	90.4	93.8	95.6	96.9	97.2	98.1
1-3	2.360	2.289	2.315	84.8	90.7	94.0	95.8	97.0	97.2	98.1
AVG				85.1	90.6	93.9	95.7	96.9	97.1	98.0
2-1	2.376	2.318	2.348	86.0	91.4	94.7	96.5	97.6	98.0	98.8
2-2	2.376	2.306	2.344	85.7	90.9	94.2	95.9	97.1	97.8	98.7
2-3	2.376	2.324	2.347	86.3	91.6	94.9	96.6	97.8	97.9	98.8
AVG				86.0	91.3	94.6	96.3	97.5	97.9	98.8
3-1	2.360	2.215	2.293	82.6	87.8	90.9	92.7	93.9	96.3	97.2
3-2	2.360	2.237	2.292	83.5	88.7	91.8	93.6	94.8	96.2	97.1
3-3	2.360	2.262	2.294	84.5	89.6	92.8	94.7	95.8	96.3	97.2
AVG				83.5	88.7	91.9	93.6	94.8	96.3	97.2

TABLE A.32 SGC Data for Project MO-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.444	2.361	2.401	85.1	90.6	93.8	95.6	96.6	97.4	98.2
1-2	2.444	2.361	2.408	85.0	90.4	93.7	95.5	96.6	97.7	98.5
1-3	2.444	2.372	2.399	85.4	90.8	94.1	95.9	97.1	97.4	98.2
AVG				85.2	90.6	93.9	95.6	96.8	97.5	98.3
2-1	2.434	2.382	2.416	86.3	91.7	95.0	96.7	97.9	98.4	99.3
2-2	2.434	2.380	2.414	86.0	91.5	94.8	96.6	97.8	98.3	99.2
2-3	2.434	2.384	2.412	86.1	91.7	95.0	96.8	97.9	98.2	99.1
AVG				86.1	91.6	95.0	96.7	97.9	98.3	99.2
3-1	2.436	2.377	2.415	86.0	91.4	94.7	96.5	97.6	98.3	99.1
3-2	2.436	2.390	2.415	86.2	91.8	95.1	96.9	98.1	98.3	99.1
3-3	2.436	2.381	2.408	86.0	91.5	94.8	96.7	97.7	98.1	98.9
AVG				86.1	91.6	94.9	96.7	97.8	98.2	99.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.444	2.348	2.398	84.4	89.8	93.1	94.9	96.1	97.1	98.1
1-2	2.444	2.345	2.393	84.7	89.9	93.0	94.7	95.9	97.0	97.9
1-3	2.444	2.353	2.395	84.6	90.1	93.3	95.1	96.3	97.1	98.0
AVG				84.6	89.9	93.1	94.9	96.1	97.1	98.0
2-1	2.434	2.374	2.396	85.6	91.1	94.5	96.3	97.5	97.5	98.4
2-2	2.434	2.363	2.401	85.1	90.6	94.0	95.9	97.1	97.7	98.6
2-3	2.434	2.367	2.395	85.4	90.9	94.2	96.0	97.2	97.4	98.4
AVG				85.4	90.9	94.2	96.1	97.3	97.5	98.5
3-1	2.436	2.368	2.393	85.4	90.9	94.3	96.0	97.2	97.3	98.2
3-2	2.436	2.369	2.398	85.5	91.0	94.2	96.1	97.2	97.5	98.4
3-3	2.436	2.366	2.400	85.2	90.7	94.1	95.8	97.1	97.6	98.5
AVG				85.4	90.9	94.2	96.0	97.2	97.5	98.4

TABLE A.33 SGC Data for Project NC-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.640	2.529	2.554	89.3	92.6	94.3	95.3	95.8	96.3	96.7
1-2	2.640	2.525	2.542	89.6	92.7	94.3	95.1	95.6	95.9	96.3
1-3	2.640	2.521	2.556	89.1	92.4	94.1	95.0	95.5	96.4	96.8
AVG				89.3	92.6	94.2	95.1	95.6	96.2	96.6
2-1	2.638	2.511	2.522	89.3	92.4	93.9	94.7	95.2	95.2	95.6
2-2	2.638	2.511	2.536	89.2	92.3	93.9	94.7	95.2	95.8	96.1
2-3	2.638	2.507	2.550	89.0	92.1	93.7	94.5	95.0	96.2	96.7
AVG				89.2	92.3	93.9	94.6	95.1	95.7	96.1
3-1	2.649	2.526	2.529	89.4	92.5	94.0	94.9	95.4	95.1	95.5
3-2	2.649	2.509	2.525	88.7	91.8	93.4	94.2	94.7	95.0	95.3
3-3	2.649	2.514	2.515	89.3	92.1	93.7	94.4	94.9	94.5	94.9
AVG				89.1	92.1	93.7	94.5	95.0	94.9	95.2

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.640	2.489	2.495	88.6	91.5	93.0	93.8	94.3	94.2	94.5
1-2	2.640	2.482	2.522	88.4	91.2	92.8	93.5	94.0	95.1	95.5
1-3	2.640	2.505	2.520	88.0	91.5	93.3	94.3	94.9	95.0	95.5
AVG				88.3	91.4	93.0	93.9	94.4	94.8	95.2
2-1	2.638	2.361	2.531	83.9	86.8	88.3	89.0	89.5	95.5	95.9
2-2	2.638	2.492	2.511	88.4	91.5	93.1	94.0	94.5	94.8	95.2
2-3	2.638	2.492	2.523	88.3	91.4	93.1	94.0	94.5	95.3	95.6
AVG				86.8	89.9	91.5	92.3	92.8	95.2	95.6
3-1	2.649	2.512	2.530	88.6	91.8	93.4	94.2	94.8	95.1	95.5
3-2	2.649	2.491	2.538	87.9	91.0	92.6	93.5	94.0	95.3	95.8
3-3	2.649	2.482	2.525	87.5	90.7	92.3	93.1	93.7	94.9	95.3
AVG				88.0	91.2	92.8	93.6	94.2	95.1	95.5

TABLE A.34 SGC Data for Project NE-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.414	2.330	2.357	90.8	93.7	95.2	96.0	96.5	97.3	97.6
1-2	2.414	2.336	2.349	91.1	93.9	95.5	96.3	96.8	96.9	97.3
1-3	2.414	2.334	2.354	91.0	93.9	95.4	96.3	96.7	97.1	97.5
AVG				91.0	93.8	95.4	96.2	96.7	97.1	97.5
2-1	2.405	2.356	2.366	92.5	95.4	96.8	97.5	98.0	97.9	98.4
2-2	2.405	2.360	2.372	92.5	95.4	96.9	97.6	98.1	98.3	98.6
2-3	2.405	2.356	2.367	92.4	95.3	96.8	97.5	98.0	98.1	98.4
AVG				92.5	95.4	96.8	97.6	98.0	98.1	98.5
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.414	2.327	2.348	90.6	93.5	95.1	95.9	96.4	96.8	97.3
1-2	2.414	2.329	2.340	90.6	93.5	95.2	96.0	96.5	96.5	96.9
1-3	2.414	2.327	2.342	90.6	93.5	95.0	95.8	96.4	96.6	97.0
AVG				90.6	93.5	95.1	95.9	96.4	96.7	97.1
2-1	2.405	2.352	2.364	91.8	94.9	96.5	97.3	97.8	98.0	98.3
2-2	2.405	2.469	2.361	96.7	99.7	101.3	102.1	102.7	97.8	98.2
2-3	2.405	2.216	2.364	86.7	89.5	90.9	91.7	92.1	98.0	98.3
AVG				91.7	94.7	96.2	97.0	97.5	97.9	98.3
3-1	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-2	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3-3	0.000	0.000	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVG				0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE A.35 SGC Data for Project NE-2

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.437	2.347	2.374	89.4	92.8	94.6	95.7	96.3	96.9	97.4
1-2	2.437	2.357	2.377	89.8	93.2	95.0	96.1	96.7	97.0	97.5
1-3	2.437	2.358	2.367	89.9	93.3	95.1	96.1	96.8	96.6	97.1
AVG				89.7	93.1	94.9	95.9	96.6	96.9	97.4
2-1	2.437	2.322	2.386	88.4	91.8	93.7	94.6	95.3	97.4	97.9
2-2	2.437	2.373	2.392	90.6	93.9	95.8	96.8	97.4	97.6	98.2
2-3	2.437	2.369	0.000	90.5	93.8	95.6	96.6	97.2	0.0	0.0
AVG				89.8	93.2	95.0	96.0	96.6	97.5	98.0
3-1	2.443	2.367	2.388	90.0	93.4	95.2	96.2	96.9	97.2	97.7
3-2	2.443	2.365	2.388	89.8	93.2	95.1	96.1	96.8	97.2	97.7
3-3	2.443	2.371	2.391	90.1	93.5	95.4	96.4	97.1	97.4	97.9
AVG				89.9	93.4	95.3	96.2	96.9	97.3	97.8

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.437	2.337	2.361	89.0	92.4	94.2	95.2	95.9	96.4	96.9
1-2	2.437	2.343	2.365	89.2	92.6	94.4	95.5	96.1	96.5	97.0
1-3	2.437	2.340	2.357	89.0	92.4	94.3	95.3	96.0	96.1	96.7
AVG				89.1	92.5	94.3	95.3	96.0	96.3	96.9
2-1	2.437	2.356	2.376	89.6	93.0	94.9	96.0	96.7	97.0	97.5
2-2	2.437	2.358	2.375	89.7	93.1	95.0	96.1	96.8	96.9	97.5
2-3	2.437	2.354	0.000	89.5	93.0	94.9	95.8	96.6	0.0	0.0
AVG				89.6	93.0	94.9	96.0	96.7	96.9	97.5
3-1	2.443	2.355	2.378	89.4	92.8	94.7	95.7	96.4	96.8	97.3
3-2	2.443	2.350	2.374	89.2	92.7	94.5	95.5	96.2	96.7	97.2
3-3	2.443	2.351	2.375	89.1	92.5	94.4	95.5	96.2	96.7	97.2
AVG				89.2	92.7	94.5	95.6	96.3	96.7	97.2

TABLE A.36 SGC Data for Project NE-3

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.405	2.317	2.346	91.0	93.7	95.1	95.8	96.3	97.1	97.5
1-2	2.405	2.316	2.341	90.9	93.6	95.0	95.8	96.3	96.9	97.3
1-3	2.405	2.329	2.339	91.5	94.2	95.6	96.4	96.8	96.8	97.3
AVG				91.1	93.8	95.2	96.0	96.5	97.0	97.4
2-1	2.390	2.337	2.350	92.4	95.2	96.6	97.4	97.8	98.0	98.3
2-2	2.390	2.338	2.350	92.6	95.3	96.7	97.4	97.8	97.9	98.3
2-3	2.390	2.321	2.349	91.8	94.5	95.9	96.6	97.1	97.9	98.3
AVG				92.3	95.0	96.4	97.1	97.6	97.9	98.3
3-1	2.398	2.320	2.358	91.6	94.3	95.6	96.3	96.7	98.0	98.3
3-2	2.398	2.322	2.341	91.3	94.3	95.7	96.4	96.8	97.3	97.6
3-3	2.398	2.304	2.341	91.0	93.5	94.9	95.7	96.1	97.3	97.6
AVG				91.3	94.0	95.4	96.1	96.6	97.5	97.9

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.405	2.318	2.341	90.8	93.6	95.1	95.9	96.4	97.0	97.3
1-2	2.405	2.276	2.334	89.3	92.0	93.4	94.1	94.6	96.6	97.0
1-3	2.405	2.315	2.323	90.6	93.5	95.0	95.7	96.3	96.2	96.6
AVG				90.2	93.0	94.5	95.2	95.8	96.6	97.0
2-1	2.390	2.328	2.343	92.0	94.7	96.2	97.0	97.4	97.7	98.0
2-2	2.390	2.326	2.334	91.9	94.7	96.1	96.8	97.3	97.3	97.7
2-3	2.390	2.323	2.347	91.9	94.6	96.0	96.8	97.2	97.9	98.2
AVG				91.9	94.7	96.1	96.9	97.3	97.6	98.0
3-1	2.398	2.316	2.331	91.2	93.9	95.4	96.1	96.6	96.9	97.2
3-2	2.398	2.312	2.325	91.2	93.8	95.2	96.0	96.4	96.5	97.0
3-3	2.398	2.310	2.331	91.1	93.8	95.2	95.8	96.3	96.9	97.2
AVG				91.2	93.8	95.3	96.0	96.4	96.8	97.1

TABLE A.37 SGC Data for Project NE-4

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.444	2.386	2.409	90.5	94.0	95.9	97.0	97.6	98.1	98.6
1-2	2.444	2.384	2.408	90.4	93.9	95.9	96.9	97.5	98.0	98.5
1-3	2.444	2.383	2.414	90.3	93.9	95.8	96.8	97.5	98.3	98.8
AVG				90.4	93.9	95.9	96.9	97.6	98.1	98.6
2-1	2.438	2.396	2.407	91.2	94.7	97.0	97.6	98.3	98.3	98.7
2-2	2.438	2.386	2.421	90.8	94.3	96.6	97.2	97.9	98.8	99.3
2-3	2.438	2.385	2.407	90.6	94.2	96.6	97.2	97.8	98.3	98.7
AVG				90.9	94.4	96.7	97.3	98.0	98.5	98.9
3-1	2.449	2.383	2.416	90.1	93.7	95.6	96.6	97.3	98.1	98.7
3-2	2.449	2.394	2.411	90.5	94.1	96.0	97.1	97.8	97.9	98.4
3-3	2.449	2.388	2.415	90.2	93.8	95.8	96.8	97.5	98.1	98.6
AVG				90.2	93.8	95.8	96.8	97.5	98.1	98.6

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.444	2.357	2.395	89.1	92.7	94.6	95.7	96.4	97.5	98.0
1-2	2.444	2.354	2.403	89.1	92.6	94.5	95.6	96.3	97.8	98.3
1-3	2.444	2.366	2.405	89.4	92.9	94.9	96.1	96.8	97.9	98.4
AVG				89.2	92.7	94.7	95.8	96.5	97.7	98.2
2-1	2.438	2.374	2.402	90.1	93.6	95.6	96.7	97.4	98.0	98.5
2-2	2.438	2.368	2.410	89.9	93.4	95.3	96.4	97.1	98.3	98.9
2-3	2.438	2.383	2.406	90.3	93.9	96.0	97.0	97.7	98.2	98.7
AVG				90.1	93.7	95.6	96.7	97.4	98.2	98.7
3-1	2.449	2.378	2.404	89.6	93.3	95.3	96.3	97.1	97.6	98.2
3-2	2.449	2.379	2.386	89.7	93.3	95.4	96.5	97.1	96.8	97.4
3-3	2.449	2.382	2.393	89.8	93.4	95.5	96.5	97.3	97.2	97.7
AVG				89.7	93.3	95.4	96.4	97.2	97.2	97.8

TABLE A.38 SGC Data for Project TN-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.459	2.388	2.415	90.0	93.6	95.5	96.5	97.1	97.7	98.2
1-2	2.459	2.392	2.413	90.4	93.8	95.7	96.7	97.3	97.6	98.1
1-3	2.459	2.389	2.418	90.2	93.7	95.6	96.6	97.2	97.8	98.3
AVG				90.2	93.7	95.6	96.6	97.2	97.7	98.2
2-1	2.467	2.403	2.420	90.3	93.9	95.8	96.8	97.4	97.6	98.1
2-2	2.467	2.404	2.416	90.6	94.0	95.9	96.8	97.4	97.4	97.9
2-3	2.467	2.400	2.419	90.6	93.9	95.8	96.7	97.3	97.5	98.1
AVG				90.5	93.9	95.8	96.8	97.4	97.5	98.0
3-1	2.464	2.398	2.412	90.3	93.8	95.6	96.6	97.3	97.5	97.9
3-2	2.464	2.397	2.413	90.3	93.8	95.6	96.6	97.3	97.4	97.9
3-3	2.464	2.398	2.420	90.2	93.8	95.7	96.7	97.3	97.7	98.2
AVG				90.3	93.8	95.6	96.7	97.3	97.5	98.0

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.459	2.384	2.415	90.8	93.9	95.6	96.4	96.9	97.7	98.2
1-2	2.459	2.392	2.406	90.3	93.7	95.6	96.6	97.3	97.4	97.8
1-3	2.459	2.394	2.411	91.2	94.3	96.0	96.8	97.4	97.6	98.0
AVG				90.8	94.0	95.7	96.6	97.2	97.6	98.0
2-1	2.467	2.399	2.418	90.1	93.7	95.6	96.6	97.2	97.5	98.0
2-2	2.467	2.398	2.416	90.3	93.8	95.5	96.5	97.2	97.4	97.9
2-3	2.467	2.399	2.419	90.4	93.8	95.7	96.3	97.2	97.5	98.1
AVG				90.3	93.7	95.6	96.5	97.2	97.5	98.0
3-1	2.464	2.396	2.417	90.1	93.6	95.5	96.5	97.2	97.6	98.1
3-2	2.464	2.381	2.409	89.5	93.0	94.9	96.0	96.6	97.3	97.8
3-3	2.464	2.394	2.408	90.0	93.6	95.5	96.4	97.2	97.2	97.7
AVG				89.8	93.4	95.3	96.3	97.0	97.4	97.9

TABLE A.39 SGC Data for Project UT-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.470	2.410	2.441	88.4	92.9	95.4	96.7	97.6	98.3	98.8
1-2	2.470	2.418	2.442	88.8	93.3	95.8	97.1	97.9	98.3	98.9
1-3	2.470	2.413	2.441	88.7	93.1	95.7	96.9	97.7	98.4	98.8
AVG				88.6	93.1	95.6	96.9	97.7	98.3	98.8
2-1	2.458	2.428	2.445	89.5	94.2	96.8	97.2	98.8	99.0	99.5
2-2	2.458	2.427	2.445	89.7	94.4	96.9	98.1	98.7	99.0	99.5
2-3	2.458	2.432	2.446	89.8	94.5	97.1	98.3	98.9	99.1	99.5
AVG				89.7	94.3	96.9	97.8	98.8	99.0	99.5
3-1	2.465	2.436	2.451	89.7	94.3	96.9	98.1	98.8	99.0	99.4
3-2	2.465	2.432	2.449	89.9	94.4	96.9	98.1	98.7	98.9	99.4
3-3	2.465	2.430	2.449	89.5	94.2	96.8	98.0	98.6	99.0	99.4
AVG				89.7	94.3	96.9	98.1	98.7	99.0	99.4

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.470	2.388	2.410	87.9	92.1	94.6	95.9	96.7	97.0	97.6
1-2	2.470	2.383	2.412	87.6	91.8	94.3	95.6	96.5	97.1	97.7
1-3	2.470	2.374	2.415	87.6	91.7	94.1	95.3	96.1	97.2	97.8
AVG				87.7	91.9	94.3	95.6	96.4	97.1	97.7
2-1	2.458	2.391	2.428	88.3	92.6	95.0	96.4	97.3	98.3	98.8
2-2	2.458	2.405	2.423	88.6	93.1	96.3	97.0	97.8	0.0	98.6
2-3	2.458	2.394	2.424	88.4	92.7	95.2	96.6	97.4	98.0	98.6
AVG				88.4	92.8	95.5	96.7	97.5	98.1	98.7
3-1	2.465	2.407	2.433	88.9	93.2	95.6	96.9	97.6	98.1	98.7
3-2	2.465	2.412	2.428	88.7	93.2	95.8	97.1	97.8	97.9	98.5
3-3	2.465	2.404	2.422	88.8	93.0	95.4	96.8	97.5	97.7	98.3
AVG				88.8	93.1	95.6	96.9	97.7	97.9	98.5

TABLE A.40 SGC Data for Project WI-1

Sample	Gmm	Gmb's		%Gmm - Gyratory A						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.563	2.454	2.475	87.4	91.5	93.8	95.0	95.7	95.9	96.6
1-2	2.563	2.443	2.490	86.9	91.0	93.3	94.5	95.3	96.5	97.2
1-3	2.563	2.453	2.457	87.5	91.4	93.7	95.0	95.7	95.2	95.9
AVG				87.3	91.3	93.6	94.8	95.6	95.9	96.5
2-1	2.558	2.459	2.490	87.9	91.9	94.2	95.4	96.1	96.7	97.3
2-2	2.558	2.456	2.495	87.7	91.8	94.0	95.3	96.0	96.9	97.5
2-3	2.558	2.458	2.494	87.6	91.8	94.0	95.3	96.1	96.9	97.5
AVG				87.7	91.8	94.1	95.3	96.1	96.9	97.5
3-1	2.546	2.451	2.486	87.5	91.7	94.1	95.4	96.3	97.0	97.6
3-2	2.546	2.466	2.474	88.2	92.5	94.9	96.1	96.9	96.5	97.2
3-3	2.546	2.453	2.490	87.9	92.0	94.3	95.6	96.3	97.2	97.8
AVG				87.9	92.1	94.4	95.7	96.5	96.9	97.5

Sample	Gmm	Gmb's		%Gmm - Gyratory B						
		@ 100 (Ndesign)	@ 160 (Nmax)	@ 8 (Ninitial)	@ 25	@ 50	@ 75	@ 100 (Ndesign)	@ 125	@ 160 (Nmax)
1-1	2.563	2.405	2.447	85.7	89.5	91.8	92.9	93.8	94.8	95.5
1-2	2.563	2.411	2.446	86.1	89.9	92.1	93.3	94.1	94.8	95.4
1-3	2.563	2.414	2.435	86.1	89.9	92.2	93.3	94.2	94.3	95.0
AVG				85.9	89.7	92.1	93.2	94.0	94.6	95.3
2-1	2.558	2.433	2.452	87.1	90.9	93.2	94.3	95.1	95.2	95.9
2-2	2.558	2.434	2.459	87.0	90.8	93.1	94.3	95.2	95.5	96.1
2-3	2.558	2.429	2.454	87.0	90.7	92.9	94.1	95.0	95.3	95.9
AVG				87.0	90.8	93.1	94.3	95.1	95.3	96.0
3-1	2.546	2.425	2.460	87.0	90.9	93.2	94.3	95.2	96.0	96.6
3-2	2.546	2.426	2.449	86.9	90.8	93.2	94.4	95.3	95.5	96.2
3-3	2.546	2.435	2.455	87.3	91.3	93.7	94.8	95.6	95.8	96.4
AVG				87.1	91.0	93.4	94.5	95.4	95.8	96.4

TABLE A.41 Core Data for Project AL-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.549	2.202	2.391	2.440	2.398	2.406	0.000	86.4	93.8	95.7	94.1	94.4	0.0
1-2	2.549	2.259	2.399	2.396	2.384	2.397	0.000	88.6	94.1	94.0	93.5	94.0	0.0
1-3	2.549	2.281	2.395	2.421	2.370	2.398	0.000	89.5	94.0	95.0	93.0	94.1	0.0
Avg.								88.2	94.0	94.9	93.5	94.2	0.0
Std.								1.60	0.16	0.87	0.55	0.19	0.00
2-1	2.566	2.333	2.393	2.386	2.388	2.420	2.431	90.9	93.3	93.0	93.1	94.3	94.7
2-2	2.566	2.283	2.348	2.361	2.355	2.368	2.389	89.0	91.5	92.0	91.8	92.3	93.1
2-3	2.566	2.278	2.359	2.381	2.357	2.392	2.423	88.8	91.9	92.8	91.9	93.2	94.4
Avg.								89.6	92.2	92.6	92.2	93.3	94.1
Std.								1.19	0.91	0.52	0.72	1.01	0.87
3-1	2.548	2.256	2.386	2.412	2.402	2.416	2.430	88.5	93.6	94.7	94.3	94.8	95.4
3-2	2.548	2.256	2.361	2.366	2.355	2.408	2.392	88.5	92.7	92.9	92.4	94.5	93.9
3-3	2.548	2.244	2.401	2.362	2.362	2.378	2.402	88.1	94.2	92.7	92.7	93.3	94.3
Avg.								88.4	93.5	93.4	93.1	94.2	94.5
Std.								0.27	0.79	1.09	1.00	0.79	0.77

TABLE A.42 Core Data for Project AL-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.466	2.182	2.196	2.204	2.194	2.186	2.199	88.5	89.1	89.4	89.0	88.6	89.2
1-2	2.466	2.142	2.164	2.185	BROKEN	2.383	2.238	86.9	87.8	88.6	0.0	96.6	90.8
1-3	2.466	2.184	2.179	2.213	2.185	2.215	2.237	88.6	88.4	89.7	88.6	89.8	90.7
Avg.								88.0	88.4	89.2	88.8	91.7	90.2
Std.								0.96	0.65	0.58	0.26	4.31	0.90
2-1	2.455	2.176	2.214	2.206	2.200	2.226	2.256	88.6	90.2	89.9	89.6	90.7	91.9
2-2	2.455	2.179	2.238	2.210	BROKEN	2.224	2.231	88.8	91.2	90.0	0.0	90.6	90.9
2-3	2.455	2.169	2.230	2.201	BROKEN	2.215	2.232	88.4	90.8	89.7	0.0	90.2	90.9
Avg.								88.6	90.7	89.8	89.6	90.5	91.2
Std.								0.21	0.50	0.18	0.00	0.24	0.58
3-1	2.460	2.155	2.228	2.234	2.251	2.276	2.292	87.6	90.6	90.8	91.5	92.5	93.2
3-2	2.460	2.179	2.259	2.242	2.271	2.311	2.320	88.6	91.8	91.1	92.3	93.9	94.3
3-3	2.460	2.183	2.296	2.274	2.285	2.283	2.292	88.7	93.3	92.4	92.9	92.8	93.2
Avg.								88.3	91.9	91.5	92.2	93.1	93.6
Std.								0.62	1.38	0.86	0.69	0.75	0.66

TABLE A.43 Core Data for Project AL-3

Sample	Gmm	Roadway Core - Gmb					Roadway Core - %Gmm				
		In-Place	3-Month	6-Month	1-Year	2-Year	In-Place	3-Month	6-Month	1-Year	2-Year
1-1	2.472	2.190	2.277	2.285	2.243	2.301	88.6	92.1	92.4	90.7	93.1
1-2	2.472	2.217	2.292	2.315	2.307	2.317	89.7	92.7	93.6	93.3	93.7
1-3	2.472	2.204	2.285	2.285	2.314	2.307	89.2	92.4	92.4	93.6	93.3
AVG							89.1	92.4	92.8	92.6	93.4
2-1	2.487	2.259	2.330	2.350	2.349	2.354	90.8	93.7	94.5	94.5	94.7
2-2	2.487	2.232	2.332	2.319	2.338	2.334	89.7	93.8	93.2	94.0	93.8
2-3	2.487	2.238	2.290	2.316	2.328	2.315	90.0	92.1	93.1	93.6	93.1
AVG							90.2	93.2	93.6	94.0	93.9
3-1											
3-2											
3-3											
AVG											

TABLE A.44 Core Data for Project AL-4

Sample	Gmm	Roadway Core - Gmb					Roadway Core - %Gmm				
		In-Place	3-Month	6-Month	1-Year	2-Year	In-Place	3-Month	6-Month	1-Year	2-Year
1-1	2.525	2.238	2.300	2.327	2.339	2.366	88.6	91.1	92.2	92.6	93.7
1-2	2.525	2.234	2.319	2.326	2.325	2.353	88.5	91.8	92.1	92.1	93.2
1-3	2.525	2.189	2.328	2.339	2.336	2.363	86.7	92.2	92.6	92.5	93.6
AVG							87.9	91.7	92.3	92.4	93.5
2-1	2.528	2.199	2.353	2.366	2.348	2.377	87.0	93.1	93.6	92.9	94.0
2-2	2.528	2.185	2.341	2.345	2.338	2.423	86.4	92.6	92.8	92.5	95.8
2-3	2.528	2.248	2.321	2.330	2.316	2.356	88.9	91.8	92.2	91.6	93.2
AVG							87.4	92.5	92.8	92.3	94.4
3-1	2.514	2.238	2.365	2.359	2.348	2.392	89.0	94.1	93.8	93.4	95.1
3-2	2.514	2.224	2.360	2.351	2.348	2.380	88.5	93.9	93.5	93.4	94.7
3-3	2.514	2.302	2.381	2.382	2.332	2.404	91.6	94.7	94.7	92.8	95.6
AVG							89.7	94.2	94.0	93.2	95.1

TABLE A.45 Core Data for Project AL-5

Sample	Gmm	Roadway Core - Gmb					Roadway Core - %Gmm				
		In-Place	3-Month	6-Month	1-Year	2-Year	In-Place	3-Month	6-Month	1-Year	2-Year
1-1	2.487	2.294	2.362	2.354	2.339	2.363	92.2	95.0	94.7	94.0	95.0
1-2	2.487	2.281	2.344	2.347	2.345	2.374	91.7	94.3	94.4	94.3	95.5
1-3	2.487	2.186	2.308	2.324	2.292	2.344	87.9	92.8	93.4	92.2	94.3
AVG							90.6	94.0	94.2	93.5	94.9
2-1	2.493	2.229	2.318	2.330	2.298	2.367	89.4	93.0	93.5	92.2	94.9
2-2	2.493	2.246	2.338	2.332	2.322	2.355	90.1	93.8	93.5	93.1	94.5
2-3	2.493	2.203	2.325	2.336	2.317	2.351	88.4	93.3	93.7	92.9	94.3
AVG							89.3	93.3	93.6	92.8	94.6
3-1	2.493	2.261	2.363	2.363	2.353	2.379	90.7	94.8	94.8	94.4	95.4
3-2	2.493	2.185	2.296	2.330	2.317	2.344	87.6	92.1	93.5	92.9	94.0
3-3	2.493	2.218	2.324	2.308	2.294	2.335	89.0	93.2	92.6	92.0	93.7
AVG							89.1	93.4	93.6	93.1	94.4

TABLE A.46 Core Data for Project AL-6

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.548	2.359	2.372	2.385	2.396	2.386	2.380	92.6	93.1	93.6	94.0	93.6	93.4
1-2	2.548	2.366	2.379	2.364	2.376	2.381	2.380	92.9	93.4	92.8	93.2	93.4	93.4
1-3	2.548	2.291	2.340	2.339	2.349	2.342	2.354	89.9	91.8	91.8	92.2	91.9	92.4
Avg.								91.8	92.8	92.7	93.2	93.0	93.1
Std.								1.63	0.82	0.90	0.93	0.95	0.59
2-1	2.530	2.333	2.362	2.360	2.365	2.365	2.376	92.2	93.4	93.3	93.5	93.5	93.9
2-2	2.530	2.342	2.346	2.330	2.330	2.376	2.384	92.6	92.7	92.1	92.1	93.9	94.2
2-3	2.530	2.294	2.377	2.341	2.360	2.366	2.389	90.7	94.0	92.5	93.3	93.5	94.4
Avg.								91.8	93.3	92.6	93.0	93.6	94.2
Std.								1.01	0.61	0.60	0.75	0.24	0.26
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.47 Core Data for Project AR-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.437	2.237	2.265	2.262	2.294	2.295	2.282	91.8	92.9	92.8	94.1	94.2	93.6
1-2	2.437	2.283	2.285	2.308	2.292	2.306	2.299	93.7	93.8	94.7	94.1	94.6	94.3
1-3	2.437	2.231	2.258	2.257	2.279	2.283	2.290	91.5	92.7	92.6	93.5	93.7	94.0
Avg.								92.3	93.1	93.4	93.9	94.2	94.0
Std.								1.17	0.57	1.15	0.33	0.47	0.35
2-1	2.429	2.242	2.283	2.261	2.314	2.313	2.317	92.3	94.0	93.1	95.3	95.2	95.4
2-2	2.429	2.233	2.256	2.300	2.266	2.283	2.273	91.9	92.9	94.7	93.3	94.0	93.6
2-3	2.429	2.236	2.269	2.284	2.284	2.293	2.293	92.1	93.4	94.0	94.0	94.4	94.4
Avg.								92.1	93.4	93.9	94.2	94.5	94.5
Std.								0.19	0.56	0.81	1.00	0.63	0.91
3-1	2.436	2.233	2.264	2.269	2.292	2.318	2.309	91.7	92.9	93.1	94.1	95.2	94.8
3-2	2.436	2.229	2.247	2.255	2.284	2.276	2.267	91.5	92.2	92.6	93.8	93.4	93.1
3-3	2.436	2.231	2.273	2.276	2.299	2.275	2.283	91.6	93.3	93.4	94.4	93.4	93.7
Avg.								91.6	92.8	93.0	94.1	94.0	93.9
Std.								0.08	0.54	0.44	0.31	1.01	0.87

TABLE A.48 Core Data for Project AR-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.464	2.202	2.230	2.247	2.262	2.268	2.263	89.4	90.5	91.2	91.8	92.0	91.8
1-2	2.464	2.188	2.253	2.256	2.255	2.261	2.262	88.8	91.4	91.6	91.5	91.8	91.8
1-3	2.464	2.227	2.251	2.259	2.264	2.280	2.287	90.4	91.4	91.7	91.9	92.5	92.8
Avg.								89.5	91.1	91.5	91.7	92.1	92.2
Std.								0.80	0.52	0.25	0.19	0.39	0.57
2-1	2.448	2.177	2.220	2.224	2.310	2.231	2.247	88.9	90.7	90.8	94.4	91.1	91.8
2-2	2.448	2.174	2.210	2.230	2.167	2.235	2.242	88.8	90.3	91.1	88.5	91.3	91.6
2-3	2.448	2.201	2.234	2.246	2.267	2.257	2.269	89.9	91.3	91.7	92.6	92.2	92.7
Avg.								89.2	90.7	91.2	91.8	91.5	92.0
Std.								0.60	0.49	0.46	3.00	0.57	0.59
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.49 Core Data for Project AR-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.426	2.213	2.279	2.279	2.287	2.270	2.238	91.2	93.9	93.9	94.3	93.6	92.3
1-2	2.426	2.221	2.286	2.297	2.285	2.291	2.256	91.5	94.2	94.7	94.2	94.4	93.0
1-3	2.426	2.233	2.321	2.327	2.320	2.297	2.329	92.0	95.7	95.9	95.6	94.7	96.0
Avg.								91.6	94.6	94.8	94.7	94.2	93.7
Std.								0.41	0.93	1.00	0.81	0.58	1.99
2-1	2.436	2.214	2.286	2.298	2.304	2.309	2.310	90.9	93.8	94.3	94.6	94.8	94.8
2-2	2.436	2.249	2.334	2.326	2.327	2.337	2.338	92.3	95.8	95.5	95.5	95.9	96.0
2-3	2.436	2.221	2.293	2.302	2.306	2.310	2.322	91.2	94.1	94.5	94.7	94.8	95.3
Avg.								91.5	94.6	94.8	94.9	95.2	95.4
Std.								0.76	1.06	0.62	0.52	0.65	0.58
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.50 Core Data for Project AR-4

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.409	2.179	2.255	2.240	2.268	2.257	2.263	90.5	93.6	93.0	94.1	93.7	93.9
1-2	2.409	2.191	2.278	2.256	2.278	2.272	2.273	91.0	94.6	93.6	94.6	94.3	94.4
1-3	2.409	2.188	2.252	2.229	2.251	2.270	2.267	90.8	93.5	92.5	93.4	94.2	94.1
Avg.								90.7	93.9	93.1	94.1	94.1	94.1
Std.								0.26	0.59	0.56	0.57	0.34	0.21
2-1	2.392	2.159	2.253	2.236	2.248	2.261	2.274	90.3	94.2	93.5	94.0	94.5	95.1
2-2	2.392	2.170	2.268	2.242	2.276	2.261	2.276	90.7	94.8	93.7	95.2	94.5	95.2
2-3	2.392	2.212	2.289	2.260	2.286	2.281	2.277	92.5	95.7	94.5	95.6	95.4	95.2
Avg.								91.2	94.9	93.9	94.9	94.8	95.1
Std.								1.17	0.76	0.52	0.82	0.48	0.06
3-1	2.401	2.185	2.252	0.000	2.264	2.264	2.276	91.0	93.8	0.0	94.3	94.3	94.8
3-2	2.401	2.179	2.250	0.000	2.260	2.268	2.274	90.8	93.7	0.0	94.1	94.5	94.7
3-3	2.401	2.187	2.260	0.000	2.267	2.279	2.276	91.1	94.1	0.0	94.4	94.9	94.8
Avg.								90.9	93.9	0.0	94.3	94.6	94.8
Std.								0.17	0.22	0.00	0.15	0.32	0.05

TABLE A.51 Core Data for Project CO-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.451	2.255	2.329	2.346	2.370	2.399	2.378	92.0	95.0	95.7	96.7	97.9	97.0
1-2	2.451	2.236	2.310	2.334	2.364	2.388	2.348	91.2	94.2	95.2	96.5	97.4	95.8
1-3	2.451	2.238	2.447	2.288	2.329	2.358	2.377	91.3	99.8	93.3	95.0	96.2	97.0
Avg.								91.5	96.4	94.8	96.1	97.2	96.6
Std.								0.43	3.03	1.25	0.90	0.87	0.70
2-1	2.436	2.341	2.382	2.379	2.389	2.412	2.408	96.1	97.8	97.7	98.1	99.0	98.9
2-2	2.436	2.316	2.372	2.365	2.379	2.397	2.392	95.1	97.4	97.1	97.7	98.4	98.2
2-3	2.436	2.280	2.345	2.347	2.359	2.391	2.396	93.6	96.3	96.3	96.8	98.2	98.4
Avg.								94.9	97.1	97.0	97.5	98.5	98.5
Std.								1.26	0.79	0.66	0.63	0.44	0.34
3-1	2.450	2.329	2.392	2.390	2.386	2.413	2.399	95.1	97.6	97.6	97.4	98.5	97.9
3-2	2.450	2.330	2.370	2.382	2.402	2.421	2.405	95.1	96.7	97.2	98.0	98.8	98.2
3-3	2.450	2.324	2.392	2.401	2.407	2.424	2.409	94.9	97.6	98.0	98.2	98.9	98.3
Avg.								95.0	97.3	97.6	97.9	98.7	98.1
Std.								0.13	0.52	0.39	0.45	0.23	0.21

TABLE A.52 Core Data for Project CO-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.428	2.336	2.397	2.387	2.384	2.403	2.401	96.2	98.7	98.3	98.2	99.0	98.9
1-2	2.428	2.299	2.362	2.366	2.378	2.389	2.372	94.7	97.3	97.4	97.9	98.4	97.7
1-3	2.428	2.304	2.361	2.350	2.364	2.371	2.355	94.9	97.2	96.8	97.4	97.7	97.0
Avg.								95.3	97.7	97.5	97.8	98.3	97.9
Std.								0.83	0.84	0.76	0.42	0.66	0.96
2-1	2.449	2.326	2.374	2.385	2.369	2.356	2.363	95.0	96.9	97.4	96.7	96.2	96.5
2-2	2.449	2.320	2.348	2.359	2.371	2.377	2.359	94.7	95.9	96.3	96.8	97.1	96.3
2-3	2.449	2.302	2.356	2.350	2.365	2.369	2.369	94.0	96.2	96.0	96.6	96.7	96.7
Avg.								94.6	96.3	96.6	96.7	96.7	96.5
Std.								0.51	0.54	0.74	0.12	0.43	0.21
3-1	2.449	2.295	2.352	2.336	2.344	2.353	2.339	93.7	96.0	95.4	95.7	96.1	95.5
3-2	2.449	2.318	2.336	2.351	2.358	2.362	2.353	94.7	95.4	96.0	96.3	96.4	96.1
3-3	2.449	2.318	2.353	2.342	2.359	2.364	2.357	94.7	96.1	95.6	96.3	96.5	96.2
Avg.								94.3	95.8	95.7	96.1	96.4	95.9
Std.								0.54	0.39	0.31	0.34	0.24	0.39

TABLE A.53 Core Data for Project CO-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.427	2.283	2.302	2.337	2.316	2.322	2.316	94.1	94.8	96.3	95.4	95.7	95.4
1-2	2.427	2.257	2.280	2.335	2.309	2.313	2.308	93.0	93.9	96.2	95.1	95.3	95.1
1-3	2.427	2.250	2.279	2.363	2.299	2.293	2.294	92.7	93.9	97.4	94.7	94.5	94.5
Avg.								93.3	94.2	96.6	95.1	95.2	95.0
Std.								0.72	0.54	0.64	0.35	0.61	0.46
2-1	2.435	2.276	2.326	2.353	2.336	2.342	2.349	93.5	95.5	96.6	95.9	96.2	96.5
2-2	2.435	2.287	2.330	2.323	2.341	2.349	2.345	93.9	95.7	95.4	96.1	96.5	96.3
2-3	2.435	2.280	2.286	2.297	2.350	2.334	2.354	93.6	93.9	94.3	96.5	95.9	96.7
Avg.								93.7	95.0	95.5	96.2	96.2	96.5
Std.								0.23	1.00	1.15	0.29	0.31	0.19
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.54 Core Data for Project CO-4

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.501	2.350	2.382	2.297	2.356	2.348	2.357	94.0	95.2	91.8	94.2	93.9	94.2
1-2	2.501	2.352	2.274	2.295	2.366	2.362	2.335	94.0	90.9	91.8	94.6	94.4	93.4
1-3	2.501	2.375	2.363	2.290	2.379	2.348	2.386	95.0	94.5	91.6	95.1	93.9	95.4
Avg.								94.3	93.5	91.7	94.6	94.1	94.3
Std.								0.56	2.31	0.14	0.46	0.32	1.02
2-1	2.497	2.333	2.348	2.324	2.351	2.364	2.382	93.4	94.0	93.1	94.2	94.7	95.4
2-2	2.497	2.308	2.293	2.340	2.334	2.341	2.353	92.4	91.8	93.7	93.5	93.8	94.2
2-3	2.497	2.363	2.325	2.326	2.346	2.338	2.348	94.6	93.1	93.2	94.0	93.6	94.0
Avg.								93.5	93.0	93.3	93.9	94.0	94.6
Std.								1.10	1.11	0.35	0.35	0.57	0.74
3-1	2.510	2.348	2.342	2.349	2.366	2.362	2.376	93.5	93.3	93.6	94.3	94.1	94.7
3-2	2.510	2.343	2.355	2.353	2.378	2.388	2.375	93.3	93.8	93.7	94.7	95.1	94.6
3-3	2.510	2.329	2.336	2.324	2.347	2.374	2.360	92.8	93.1	92.6	93.5	94.6	94.0
Avg.								93.2	93.4	93.3	94.2	94.6	94.4
Std.								0.39	0.39	0.63	0.62	0.52	0.36

TABLE A.55 Core Data for Project CO-5

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.451	2.289	2.335	2.331	2.349	2.329	2.330	93.4	95.3	95.1	95.8	95.0	95.1
1-2	2.451	2.249	2.308	2.314	2.325	2.313	2.312	91.8	94.2	94.4	94.9	94.4	94.3
1-3	2.451	2.272	2.323	2.303	2.315	2.316	2.300	92.7	94.8	94.0	94.5	94.5	93.8
Avg.								92.6	94.7	94.5	95.0	94.6	94.4
Std.								0.82	0.55	0.58	0.71	0.35	0.62
2-1	2.462	2.244	2.281	2.299	2.304	2.293	2.259	91.1	92.6	93.4	93.6	93.1	91.8
2-2	2.462	2.247	2.289	2.295	2.309	2.295	2.289	91.3	93.0	93.2	93.8	93.2	93.0
2-3	2.462	2.253	2.292	2.290	2.305	2.301	2.300	91.5	93.1	93.0	93.6	93.5	93.4
Avg.								91.3	92.9	93.2	93.7	93.3	92.7
Std.								0.19	0.23	0.18	0.11	0.17	0.86
3-1	2.462	2.238	2.290	2.293	2.304	2.302	2.290	90.9	93.0	93.1	93.6	93.5	93.0
3-2	2.462	2.239	2.292	2.295	2.313	2.301	2.266	90.9	93.1	93.2	93.9	93.5	92.0
3-3	2.462	2.245	2.293	2.301	2.310	2.295	2.302	91.2	93.1	93.5	93.8	93.2	93.5
Avg.								91.0	93.1	93.3	93.8	93.4	92.9
Std.								0.15	0.06	0.17	0.19	0.15	0.74

TABLE A.56 Core Data for Project FL-1

Sample	Gmm	Roadway Core - Gmb					Roadway Core - %Gmm				
		In-Place	3-Month	6-Month	1-Year	2-Year	In-Place	3-Month	6-Month	1-Year	2-Year
1-1	2.460	2.233	2.298	2.317	2.303	2.318	90.8	93.4	94.2	93.6	94.2
1-2	2.460	2.285	2.319	2.332	2.336	2.349	92.9	94.3	94.8	95.0	95.5
1-3	2.460	2.277	2.302	2.331	2.329	2.337	92.6	93.6	94.8	94.7	95.0
AVG							92.1	93.8	94.6	94.4	94.9
2-1	2.450	2.258	2.337	2.330	2.353	2.352	92.2	95.4	95.1	96.0	96.0
2-2	2.450	2.196	2.282	2.313	2.320	2.331	89.6	93.1	94.4	94.7	95.1
2-3	2.450	2.274	2.333	2.340	2.253	2.343	92.8	95.2	95.5	92.0	95.6
AVG							91.5	94.6	95.0	94.2	95.6
3-1											
3-2											
3-3											
AVG											

TABLE A.57 Core Data for Project GA-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.540	2.448	2.433	2.442	2.447	2.462	2.447	96.4	95.8	96.1	96.3	96.9	96.3
1-2	2.540	2.414	2.419	2.426	2.417	2.444	2.450	95.0	95.2	95.5	95.2	96.2	96.5
1-3	2.540	2.396	2.432	2.423	2.415	2.435	2.422	94.3	95.7	95.4	95.1	95.9	95.4
Avg.								95.2	95.6	95.7	95.5	96.3	96.0
Std.								1.04	0.31	0.40	0.71	0.54	0.61
2-1	2.520	2.405	2.408	2.422	2.418	2.451	2.444	95.4	95.6	96.1	96.0	97.3	97.0
2-2	2.520	2.393	2.422	2.415	2.447	2.450	2.441	95.0	96.1	95.8	97.1	97.2	96.9
2-3	2.520	2.417	2.403	2.424	2.433	2.438	2.442	95.9	95.4	96.2	96.5	96.7	96.9
Avg.								95.4	95.7	96.0	96.5	97.1	96.9
Std.								0.48	0.39	0.19	0.58	0.29	0.06
3-1	2.537	2.415	2.440	2.433	2.435	2.442	2.431	95.2	96.2	95.9	96.0	96.3	95.8
3-2	2.537	2.385	2.428	2.424	2.435	2.423	2.432	94.0	95.7	95.5	96.0	95.5	95.9
3-3	2.537	2.368	2.431	2.426	2.436	2.443	2.449	93.3	95.8	95.6	96.0	96.3	96.5
Avg.								94.2	95.9	95.7	96.0	96.0	96.1
Std.								0.94	0.25	0.19	0.02	0.44	0.40

TABLE A.58 Core Data for Project IL-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.502	2.284	2.350	2.326	2.346	2.345	2.360	91.3	93.9	93.0	93.8	93.7	94.3
1-2	2.502	2.255	2.357	2.340	2.340	2.355	2.356	90.1	94.2	93.5	93.5	94.1	94.2
1-3	2.502	2.249	2.320	2.311	2.338	2.359	2.348	89.9	92.7	92.4	93.4	94.3	93.8
Avg.								90.4	93.6	93.0	93.6	94.0	94.1
Std.								0.75	0.79	0.58	0.17	0.29	0.24
2-1	2.499	2.247	2.325	2.327	2.350	2.349	2.366	89.9	93.0	93.1	94.0	94.0	94.7
2-2	2.499	2.312	2.378	2.377	2.369	2.373	2.382	92.5	95.2	95.1	94.8	95.0	95.3
2-3	2.499	2.346	2.395	2.407	2.405	2.402	2.404	93.9	95.8	96.3	96.2	96.1	96.2
Avg.								92.1	94.7	94.9	95.0	95.0	95.4
Std.								2.01	1.46	1.62	1.12	1.06	0.76
3-1	2.491	2.249	2.322	2.333	2.336	2.328	2.351	90.3	93.2	93.7	93.8	93.5	94.4
3-2	2.491	2.235	2.326	2.324	2.348	2.355	2.346	89.7	93.4	93.3	94.3	94.5	94.2
3-3	2.491	2.280	2.335	2.332	2.339	2.353	2.354	91.5	93.7	93.6	93.9	94.5	94.5
Avg.								90.5	93.4	93.5	94.0	94.2	94.4
Std.								0.92	0.27	0.20	0.25	0.60	0.16

TABLE A.59 Core Data for Project IL-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.446	2.247	2.313	2.305	2.292	2.324	2.333	91.9	94.6	94.2	93.7	95.0	95.4
1-2	2.446	2.246	2.300	2.297	2.310	2.321	2.318	91.8	94.0	93.9	94.4	94.9	94.8
1-3	2.446	2.255	2.295	2.301	2.312	2.329	2.328	92.2	93.8	94.1	94.5	95.2	95.2
Avg.								92.0	94.1	94.1	94.2	95.0	95.1
Std.								0.20	0.38	0.16	0.45	0.17	0.31
2-1	2.428	2.206	2.272	2.260	2.303	2.300	2.301	90.9	93.6	93.1	94.9	94.7	94.8
2-2	2.428	2.223	2.302	2.298	2.281	2.318	2.322	91.6	94.8	94.6	93.9	95.5	95.6
2-3	2.428	2.239	2.291	2.296	2.299	2.323	2.331	92.2	94.4	94.6	94.7	95.7	96.0
Avg.								91.5	94.2	94.1	94.5	95.3	95.5
Std.								0.68	0.63	0.88	0.48	0.50	0.63
3-1	2.433	2.242	2.298	2.298	2.301	2.322	2.325	92.1	94.5	94.5	94.6	95.4	95.6
3-2	2.433	2.214	2.290	2.281	2.291	2.318	2.324	91.0	94.1	93.8	94.2	95.3	95.5
3-3	2.433	2.208	2.285	2.271	2.278	2.299	2.305	90.8	93.9	93.3	93.6	94.5	94.7
Avg.								91.3	94.2	93.8	94.1	95.1	95.3
Std.								0.75	0.27	0.56	0.47	0.51	0.46

TABLE A.60 Core Data for Project IL-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.505	2.283	2.341	2.332	2.348	2.343	2.359	91.1	93.5	93.1	93.7	93.5	94.2
1-2	2.505	2.284	2.340	2.334	2.345	2.353	2.359	91.2	93.4	93.2	93.6	93.9	94.2
1-3	2.505	2.295	2.356	2.344	2.351	2.359	2.365	91.6	94.1	93.6	93.9	94.2	94.4
Avg.								91.3	93.6	93.3	93.7	93.9	94.3
Std.								0.27	0.36	0.26	0.12	0.32	0.14
2-1	2.493	2.321	2.365	2.370	2.371	2.376	2.371	93.1	94.9	95.1	95.1	95.3	95.1
2-2	2.493	2.286	2.335	2.317	2.350	2.354	2.365	91.7	93.7	92.9	94.3	94.4	94.9
2-3	2.493	2.294	2.342	2.329	2.348	2.337	2.350	92.0	93.9	93.4	94.2	93.7	94.3
Avg.								92.3	94.2	93.8	94.5	94.5	94.7
Std.								0.74	0.63	1.11	0.51	0.78	0.43
3-1	2.493	2.297	2.366	2.363	2.357	2.362	2.355	92.1	94.9	94.8	94.5	94.7	94.5
3-2	2.493	2.331	2.373	2.357	2.369	2.372	2.361	93.5	95.2	94.5	95.0	95.1	94.7
3-3	2.493	2.331	2.373	2.356	2.377	2.371	2.381	93.5	95.2	94.5	95.3	95.1	95.5
Avg.								93.0	95.1	94.6	95.0	95.0	94.9
Std.								0.79	0.16	0.15	0.40	0.22	0.55

TABLE A.61 Core Data for Project IN-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.465	2.200	2.178	2.168	2.243	2.257	2.310	89.2	88.4	88.0	91.0	91.6	93.7
1-2	2.465	2.233	2.205	2.201	2.262	2.295	2.291	90.6	89.5	89.3	91.8	93.1	92.9
1-3	2.465	2.221	2.243	2.247	2.299	2.309	2.260	90.1	91.0	91.2	93.3	93.7	91.7
Avg.								90.0	89.6	89.5	92.0	92.8	92.8
Std.								0.68	1.32	1.61	1.16	1.09	1.02
2-1	2.469	2.259	2.309	2.240	2.353	2.354	2.308	91.5	93.5	90.7	95.3	95.3	93.5
2-2	2.469	2.235	2.292	2.269	2.293	2.298	2.322	90.5	92.8	91.9	92.9	93.1	94.0
2-3	2.469	2.267	2.247	2.286	2.310	2.333	2.381	91.8	91.0	92.6	93.6	94.5	96.4
Avg.								91.3	92.5	91.7	93.9	94.3	94.7
Std.								0.67	1.30	0.94	1.25	1.15	1.57
3-1	2.471	2.262	2.253	2.258	2.305	2.320	2.354	91.5	91.2	91.4	93.3	93.9	95.3
3-2	2.471	2.321	2.162	2.200	2.263	2.297	2.350	93.9	87.5	89.0	91.6	93.0	95.1
3-3	2.471	2.282	2.177	2.201	2.259	2.318	2.329	92.4	88.1	89.1	91.4	93.8	94.3
Avg.								92.6	88.9	89.8	92.1	93.6	94.9
Std.								1.21	1.97	1.34	1.03	0.52	0.54

TABLE A.62 Core Data for Project IN-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.684	2.471	2.434	2.468	2.561	2.555	2.570	92.1	90.7	92.0	95.4	95.2	95.8
1-2	2.684	2.368	2.406	2.461	2.537	2.497	2.500	88.2	89.6	91.7	94.5	93.0	93.1
1-3	2.684	2.395	2.404	2.479	2.515	2.510	2.492	89.2	89.6	92.4	93.7	93.5	92.8
Avg.								89.8	90.0	92.0	94.5	93.9	93.9
Std.								1.99	0.62	0.34	0.86	1.13	1.60
2-1	2.673	2.423	2.417	2.437	2.533	2.571	2.557	90.6	90.4	91.2	94.8	96.2	95.7
2-2	2.673	2.475	2.395	2.423	2.538	2.505	2.559	92.6	89.6	90.6	94.9	93.7	95.7
2-3	2.673	2.472	2.432	2.447	2.519	2.528	2.531	92.5	91.0	91.5	94.2	94.6	94.7
Avg.								91.9	90.3	91.1	94.7	94.8	95.4
Std.								1.09	0.70	0.45	0.37	1.25	0.58
3-1	2.698	2.496	2.497	2.489	2.578	2.546	2.584	92.5	92.6	92.3	95.6	94.4	95.8
3-2	2.698	2.519	2.491	2.506	2.542	2.512	2.558	93.4	92.3	92.9	94.2	93.1	94.8
3-3	2.698	2.470	2.446	2.443	2.554	2.516	2.553	91.5	90.7	90.5	94.7	93.3	94.6
Avg.								92.5	91.8	91.9	94.8	93.6	95.1
Std.								0.91	1.03	1.21	0.68	0.69	0.62

TABLE A.63 Core Data for Project KS-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.435	2.130	2.190	2.203	2.227	2.295	2.294	87.5	89.9	90.5	91.5	94.3	94.2
1-2	2.435	2.140	2.229	2.254	2.319	2.306	2.255	87.9	91.5	92.6	95.2	94.7	92.6
1-3	2.435	2.203	2.229	2.242	2.273	2.282	2.253	90.5	91.5	92.1	93.3	93.7	92.5
Avg.								88.6	91.0	91.7	93.3	94.2	93.1
Std.								1.63	0.92	1.10	1.89	0.49	0.95
2-1	2.421	2.192	2.243	2.246	2.287	2.281	2.245	90.5	92.6	92.8	94.5	94.2	92.7
2-2	2.421	2.195	2.209	2.234	2.287	2.270	2.273	90.7	91.2	92.3	94.5	93.8	93.9
2-3	2.421	2.214	2.229	2.251	2.308	2.293	2.285	91.4	92.1	93.0	95.3	94.7	94.4
Avg.								90.9	92.0	92.7	94.8	94.2	93.7
Std.								0.49	0.71	0.36	0.50	0.48	0.85
3-1	2.413	2.225	2.196	2.216	2.235	2.231	2.214	92.2	91.0	91.8	92.6	92.5	91.8
3-2	2.413	2.183	2.181	2.232	2.252	2.248	2.256	90.5	90.4	92.5	93.3	93.2	93.5
3-3	2.413	2.114	2.174	2.208	2.214	2.202	2.215	87.6	90.1	91.5	91.8	91.3	91.8
Avg.								90.1	90.5	92.0	92.6	92.3	92.3
Std.								2.32	0.47	0.50	0.79	0.96	0.99

TABLE A.64 Core Data for Project KY-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.480	2.125	2.156	2.147	2.201	2.226	2.104	85.7	86.9	86.6	88.8	89.8	84.8
1-2	2.480	2.158	2.169	2.158	2.199	2.198	2.147	87.0	87.5	87.0	88.7	88.6	86.6
1-3	2.480	2.166	2.175	2.160	2.164	2.197	2.122	87.3	87.7	87.1	87.3	88.6	85.6
Avg.								86.7	87.4	86.9	88.2	89.0	85.7
Std.								0.88	0.39	0.28	0.84	0.66	0.87
2-1	2.453	2.025	2.068	2.124	2.153	2.171	2.198	82.6	84.3	86.6	87.8	88.5	89.6
2-2	2.453	2.125	2.153	2.137	2.170	2.181	2.186	86.6	87.8	87.1	88.5	88.9	89.1
2-3	2.453	2.059	2.194	2.099	2.090	2.126	2.214	83.9	89.4	85.6	85.2	86.7	90.3
Avg.								84.4	87.2	86.4	87.1	88.0	89.7
Std.								2.07	2.62	0.79	1.72	1.19	0.57
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.65 Core Data for Project KY-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.488	2.286	2.310	2.305	2.320	2.329	2.336	91.9	92.8	92.6	93.2	93.6	93.9
1-2	2.488	2.288	2.307	2.308	2.330	2.330	2.340	92.0	92.7	92.8	93.6	93.6	94.1
1-3	2.488	2.292	2.312	2.310	2.332	2.342	2.342	92.1	92.9	92.8	93.7	94.1	94.1
Avg.								92.0	92.8	92.8	93.5	93.8	94.0
Std.								0.12	0.10	0.10	0.26	0.29	0.12
2-1	2.470	2.292	2.328	2.340	2.337	2.344	2.345	92.8	94.3	94.7	94.6	94.9	94.9
2-2	2.470	2.280	2.318	2.330	2.346	2.340	2.347	92.3	93.8	94.3	95.0	94.7	95.0
2-3	2.470	2.270	2.284	2.283	2.305	2.313	2.326	91.9	92.5	92.4	93.3	93.6	94.2
Avg.								92.3	93.5	93.8	94.3	94.4	94.7
Std.								0.45	0.93	1.23	0.87	0.68	0.47
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.66 Core Data for Project KY-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.484	2.285	2.295	2.324	2.288	2.309	2.333	92.0	92.4	93.6	92.1	93.0	93.9
1-2	2.484	2.247	2.248	2.354	2.290	2.307	2.301	90.5	90.5	94.8	92.2	92.9	92.6
1-3	2.484	2.327	2.279	2.332	2.325	2.300	2.276	93.7	91.7	93.9	93.6	92.6	91.6
Avg.								92.0	91.5	94.1	92.6	92.8	92.7
Std.								1.61	0.96	0.63	0.84	0.19	1.15
2-1	2.481	2.274	2.305	2.327	2.345	2.313	2.358	91.7	92.9	93.8	94.5	93.2	95.0
2-2	2.481	2.317	2.345	2.261	2.362	2.343	2.351	93.4	94.5	91.1	95.2	94.4	94.8
2-3	2.481	2.238	2.309	2.279	2.348	2.339	2.332	90.2	93.1	91.9	94.6	94.3	94.0
Avg.								91.8	93.5	92.3	94.8	94.0	94.6
Std.								1.59	0.89	1.38	0.37	0.66	0.54
3-1	2.486	2.330	2.366	2.348	2.370	2.378	2.381	93.7	95.2	94.4	95.3	95.7	95.8
3-2	2.486	2.342	2.336	2.374	2.390	2.408	2.401	94.2	94.0	95.5	96.1	96.9	96.6
3-3	2.486	2.332	2.322	2.351	2.352	2.356	2.365	93.8	93.4	94.6	94.6	94.8	95.1
Avg.								93.9	94.2	94.8	95.4	95.8	95.8
Std.								0.26	0.90	0.57	0.76	1.05	0.73

TABLE A.67 Core Data for Project MI-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.478	2.263	2.285	2.313	2.324	2.349	2.369	91.3	92.2	93.3	93.8	94.8	95.6
1-2	2.478	2.272	2.285	2.292	2.315	2.354	2.341	91.7	92.2	92.5	93.4	95.0	94.5
1-3	2.478	2.271	2.275	2.294	2.308	2.347	2.331	91.6	91.8	92.6	93.1	94.7	94.1
Avg.								91.6	92.1	92.8	93.4	94.8	94.7
Std.								0.20	0.23	0.47	0.32	0.15	0.79
2-1	2.472	2.278	2.279	2.296	2.297	2.350	2.333	92.2	92.2	92.9	92.9	95.1	94.4
2-2	2.472	2.319	2.267	2.288	2.317	2.368	2.341	93.8	91.7	92.6	93.7	95.8	94.7
2-3	2.472	2.240	2.271	2.268	2.305	2.341	2.344	90.6	91.9	91.7	93.2	94.7	94.8
Avg.								92.2	91.9	92.4	93.3	95.2	94.6
Std.								1.60	0.25	0.58	0.41	0.56	0.23
3-1	2.497	2.244	2.310	2.318	2.332	2.359	2.338	89.9	92.5	92.8	93.4	94.5	93.6
3-2	2.497	2.247	2.304	2.332	2.350	2.358	2.343	90.0	92.3	93.4	94.1	94.4	93.8
3-3	2.497	2.266	2.291	2.296	2.317	2.343	2.345	90.7	91.8	92.0	92.8	93.8	93.9
Avg.								90.2	92.2	92.7	93.4	94.2	93.8
Std.								0.48	0.39	0.73	0.66	0.36	0.14

TABLE A.68 Core Data for Project MI-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.446	2.292	2.281	2.348	2.348	2.378	2.388	93.7	93.3	96.0	96.0	97.2	97.6
1-2	2.446	2.330	2.308	2.352	2.322	2.384	2.389	95.3	94.4	96.2	94.9	97.5	97.7
1-3	2.446	2.283	2.302	2.359	2.373	2.373	2.373	93.3	94.1	96.4	97.0	97.0	97.0
Avg.								94.1	93.9	96.2	96.0	97.2	97.4
Std.								1.02	0.58	0.23	1.04	0.23	0.37
2-1	2.440	2.275	2.380	2.364	2.387	2.394	2.401	93.2	97.5	96.9	97.8	98.1	98.4
2-2	2.440	2.291	2.385	2.386	2.399	2.391	2.408	93.9	97.7	97.8	98.3	98.0	98.7
2-3	2.440	2.277	2.366	2.387	2.384	2.388	2.397	93.3	97.0	97.8	97.7	97.9	98.2
Avg.								93.5	97.4	97.5	98.0	98.0	98.4
Std.								0.36	0.40	0.53	0.33	0.12	0.23
3-1	2.458	2.246	2.339	2.334	2.374	2.354	2.384	91.4	95.2	95.0	96.6	95.8	97.0
3-2	2.458	2.257	2.314	2.314	2.372	2.331	2.355	91.8	94.1	94.1	96.5	94.8	95.8
3-3	2.458	2.256	2.353	2.333	2.376	2.334	0.000	91.8	95.7	94.9	96.7	95.0	0.0
Avg.								91.7	95.0	94.7	96.6	95.2	96.4
Std.								0.25	0.80	0.46	0.08	0.51	0.83

TABLE A.69 Core Data for Project MI-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.468	2.288	2.304	2.335	0.000	2.387	2.396	92.7	93.4	94.6	0.0	96.7	97.1
1-2	2.468	2.336	2.311	2.338	0.000	2.375	2.378	94.7	93.6	94.7	0.0	96.2	96.4
1-3	2.468	2.282	2.308	2.324	0.000	2.388	2.392	92.5	93.5	94.2	0.0	96.8	96.9
Avg.								93.3	93.5	94.5	0.0	96.6	96.8
Std.								1.20	0.14	0.30	0.00	0.29	0.38
2-1	2.466	2.291	2.307	2.314	0.000	2.362	2.375	92.9	93.6	93.8	0.0	95.8	96.3
2-2	2.466	2.297	2.328	2.347	0.000	2.383	2.406	93.1	94.4	95.2	0.0	96.6	97.6
2-3	2.466	2.278	2.307	2.326	0.000	2.383	2.388	92.4	93.6	94.3	0.0	96.6	96.8
Avg.								92.8	93.8	94.4	0.0	96.4	96.9
Std.								0.39	0.49	0.68	0.00	0.49	0.63
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.70 Core Data for Project MO-1

Sample	Gmm	Roadway Core - Gmb					Roadway Core - %Gmm				
		In-Place	3-Month	6-Month	1-Year	2-Year	In-Place	3-Month	6-Month	1-Year	2-Year
1-1	2.474	2.287	2.381	2.354	2.360		92.4	96.2	95.1	95.4	
1-2	2.474	2.300	2.388	2.334	2.384		93.0	96.5	94.3	96.4	
1-3	2.474	2.359	2.381	2.345	2.378		95.4	96.2	94.8	96.1	
AVG							93.6	96.3	94.8	96.0	
2-1	2.476	2.302	2.410	2.404	2.369		93.0	97.3	97.1	95.7	
2-2	2.476	2.319	2.386	2.379	2.386		93.7	96.4	96.1	96.4	
2-3	2.476	2.328	2.396	2.384	2.379		94.0	96.8	96.3	96.1	
AVG							93.6	96.8	96.5	96.0	
3-1	2.485	2.309	2.392	2.377	2.358		92.9	96.3	95.7	94.9	
3-2	2.485	2.330	2.389	2.373	2.385		93.8	96.1	95.5	96.0	
3-3	2.485	2.309	2.378	2.373	2.373		92.9	95.7	95.5	95.5	
AVG							93.2	96.0	95.5	95.5	

TABLE A.71 Core Data for Project MO-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.360	2.166	2.223	2.163	2.196	2.219	2.221	91.8	94.2	91.7	93.1	94.0	94.1
1-2	2.360	2.189	2.177	2.178	2.219	2.252	2.237	92.8	92.2	92.3	94.0	95.4	94.8
1-3	2.360	2.217	2.239	2.249	2.260	2.259	2.290	93.9	94.9	95.3	95.8	95.7	97.0
Avg.								92.8	93.8	93.1	94.3	95.1	95.3
Std.								1.08	1.36	1.95	1.37	0.91	1.53
2-1	2.376	2.176	2.244	2.179	2.244	2.270	2.248	91.6	94.4	91.7	94.4	95.5	94.6
2-2	2.376	2.182	2.262	2.190	2.243	2.289	2.268	91.8	95.2	92.2	94.4	96.3	95.5
2-3	2.376	2.182	2.252	2.181	2.242	2.276	2.274	91.8	94.8	91.8	94.4	95.8	95.7
Avg.								91.8	94.8	91.9	94.4	95.9	95.3
Std.								0.15	0.38	0.25	0.04	0.41	0.57
3-1	2.360	2.194	2.214	2.205	2.242	2.234	2.226	93.0	93.8	93.4	95.0	94.7	94.3
3-2	2.360	2.215	2.223	2.182	2.214	2.230	2.219	93.9	94.2	92.5	93.8	94.5	94.0
3-3	2.360	2.201	2.224	2.197	2.239	2.212	2.244	93.3	94.2	93.1	94.9	93.7	95.1
Avg.								93.4	94.1	93.0	94.6	94.3	94.5
Std.								0.45	0.23	0.49	0.65	0.50	0.55

TABLE A.72 Core Data for Project MO-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.444	2.271	2.287	2.284	2.319	2.312	2.318	92.9	93.6	93.5	94.9	94.6	94.8
1-2	2.444	2.298	2.299	2.281	2.334	2.316	2.311	94.0	94.1	93.3	95.5	94.8	94.6
1-3	2.444	2.288	2.291	2.288	2.318	2.336	2.335	93.6	93.7	93.6	94.8	95.6	95.5
Avg.								93.5	93.8	93.5	95.1	95.0	95.0
Std.								0.56	0.25	0.14	0.37	0.53	0.51
2-1	2.434	2.266	2.293	2.303	2.331	2.321	2.331	93.1	94.2	94.6	95.8	95.4	95.8
2-2	2.434	2.272	2.295	2.292	2.306	2.313	2.326	93.3	94.3	94.2	94.7	95.0	95.6
2-3	2.434	2.267	2.297	2.300	2.322	2.346	2.327	93.1	94.4	94.5	95.4	96.4	95.6
Avg.								93.2	94.3	94.4	95.3	95.6	95.6
Std.								0.13	0.08	0.23	0.52	0.71	0.11
3-1	2.436	2.286	2.320	2.315	2.337	2.346	2.344	93.8	95.2	95.0	95.9	96.3	96.2
3-2	2.436	2.294	2.313	2.326	2.329	2.339	2.335	94.2	95.0	95.5	95.6	96.0	95.9
3-3	2.436	2.281	2.313	2.312	2.325	2.337	2.323	93.6	95.0	94.9	95.4	95.9	95.4
Avg.								93.9	95.0	95.1	95.7	96.1	95.8
Std.								0.27	0.17	0.30	0.25	0.19	0.43

TABLE A.73 Core Data for Project NC-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.640	2.418	2.469	2.407	2.454	2.458	2.492	91.6	93.5	91.2	93.0	93.1	94.4
1-2	2.640	2.396	2.448	2.417	2.457	2.472	2.472	90.8	92.7	91.6	93.1	93.6	93.6
1-3	2.640	2.330	2.414	2.403	2.422	2.433	2.458	88.3	91.4	91.0	91.7	92.2	93.1
Avg.								90.2	92.6	91.3	92.6	93.0	93.7
Std.								1.73	1.05	0.27	0.73	0.75	0.65
2-1	2.638	2.416	2.471	2.460	2.465	2.471	2.481	91.6	93.7	93.3	93.4	93.7	94.0
2-2	2.638	2.350	2.435	2.429	2.437	2.465	2.484	89.1	92.3	92.1	92.4	93.4	94.2
2-3	2.638	2.363	2.431	2.397	2.443	2.449	2.458	89.6	92.2	90.9	92.6	92.8	93.2
Avg.								90.1	92.7	92.1	92.8	93.3	93.8
Std.								1.33	0.84	1.19	0.56	0.43	0.54
3-1	2.649	2.374	2.460	2.418	2.473	2.489	2.498	89.6	92.9	91.3	93.4	94.0	94.3
3-2	2.649	2.381	2.463	2.443	2.486	2.489	2.498	89.9	93.0	92.2	93.8	94.0	94.3
3-3	2.649	2.401	2.466	2.445	2.479	2.480	2.484	90.6	93.1	92.3	93.6	93.6	93.8
Avg.								90.0	93.0	91.9	93.6	93.8	94.1
Std.								0.53	0.11	0.57	0.25	0.20	0.31

TABLE A.74 Core Data for Project NE-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.414	0.000	2.318	2.327	2.327	2.325	2.311	0.0	96.0	96.4	96.4	96.3	95.7
1-2	2.414	0.000	2.329	2.280	2.323	2.322	2.327	0.0	96.5	94.4	96.2	96.2	96.4
1-3	2.414	2.234	2.317	2.287	2.328	2.337	2.318	92.5	96.0	94.7	96.4	96.8	96.0
Avg.								92.5	96.2	95.2	96.4	96.4	96.1
Std.								0.00	0.28	1.05	0.11	0.33	0.33
2-1	2.405	2.251	2.274	2.269	2.260	2.280	2.278	93.6	94.6	94.3	94.0	94.8	94.7
2-2	2.405	2.205	2.271	2.326	2.280	2.286	2.283	91.7	94.4	96.7	94.8	95.1	94.9
2-3	2.405	2.227	2.281	2.324	2.261	2.286	2.291	92.6	94.8	96.6	94.0	95.1	95.3
Avg.								92.6	94.6	95.9	94.3	95.0	95.0
Std.								0.96	0.21	1.34	0.47	0.14	0.27
3-1													
3-2													
3-3													
Avg.													
Std.													

TABLE A.75 Core Data for Project NE-2

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.437	2.256	2.309	2.326	2.326	2.341	2.348	92.6	94.7	95.4	95.4	96.1	96.3
1-2	2.437	2.282	2.319	2.310	2.303	2.351	2.338	93.6	95.2	94.8	94.5	96.5	95.9
1-3	2.437	2.285	2.317	2.310	2.328	2.326	2.335	93.8	95.1	94.8	95.5	95.4	95.8
Avg.								93.3	95.0	95.0	95.2	96.0	96.0
Std.								0.65	0.22	0.38	0.57	0.52	0.28
2-1	2.437	2.262	2.320	2.322	2.329	2.334	2.337	92.8	95.2	95.3	95.6	95.8	95.9
2-2	2.437	2.261	2.317	2.325	2.335	2.339	2.340	92.8	95.1	95.4	95.8	96.0	96.0
2-3	2.437	2.256	2.318	2.329	2.324	2.334	2.334	92.6	95.1	95.6	95.4	95.8	95.8
Avg.								92.7	95.1	95.4	95.6	95.8	95.9
Std.								0.13	0.06	0.14	0.23	0.12	0.12
3-1	2.443	2.270	2.338	2.300	2.336	2.332	2.340	92.9	95.7	94.1	95.6	95.5	95.8
3-2	2.443	2.279	2.340	2.325	2.343	2.318	2.332	93.3	95.8	95.2	95.9	94.9	95.5
3-3	2.443	2.263	2.326	2.311	2.299	2.336	2.340	92.6	95.2	94.6	94.1	95.6	95.8
Avg.								92.9	95.6	94.6	95.2	95.3	95.7
Std.								0.33	0.31	0.51	0.97	0.39	0.19

TABLE A.76 Core Data for Project NE-3

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.405	2.191	2.300	2.289	2.300	2.292	2.314	91.1	95.6	95.2	95.6	95.3	96.2
1-2	2.405	2.171	2.274	2.288	2.296	2.301	2.239	90.3	94.6	95.1	95.5	95.7	93.1
1-3	2.405	2.174	2.253	2.268	2.269	2.288	2.292	90.4	93.7	94.3	94.3	95.1	95.3
Avg.								90.6	94.6	94.9	95.1	95.4	94.9
Std.								0.45	0.98	0.49	0.70	0.28	1.60
2-1	2.390	2.219	2.280	2.285	2.281	2.291	2.298	92.8	95.4	95.6	95.4	95.9	96.2
2-2	2.390	2.165	2.274	2.281	2.285	2.289	2.293	90.6	95.1	95.4	95.6	95.8	95.9
2-3	2.390	2.158	2.270	2.281	2.269	2.286	2.259	90.3	95.0	95.4	94.9	95.6	94.5
Avg.								91.2	95.2	95.5	95.3	95.8	95.5
Std.								1.40	0.21	0.10	0.35	0.11	0.89
3-1	2.398	2.188	2.268	2.283	2.270	2.288	2.291	91.2	94.6	95.2	94.7	95.4	95.5
3-2	2.398	2.201	2.268	2.268	2.262	2.275	2.274	91.8	94.6	94.6	94.3	94.9	94.8
3-3	2.398	2.164	2.267	2.271	2.260	2.267	2.279	90.2	94.5	94.7	94.2	94.5	95.0
Avg.								91.1	94.6	94.8	94.4	94.9	95.1
Std.								0.78	0.02	0.33	0.22	0.44	0.36

TABLE A.77 Core Data for Project NE-4

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.444	2.251	2.310	2.314	2.355	2.373	2.384	92.1	94.5	94.7	96.4	97.1	97.5
1-2	2.444	2.253	2.317	2.319	2.366	2.364	2.371	92.2	94.8	94.9	96.8	96.7	97.0
1-3	2.444	2.260	2.334	2.335	2.374	2.381	2.384	92.5	95.5	95.5	97.1	97.4	97.5
Avg.								92.3	94.9	95.0	96.8	97.1	97.4
Std.								0.19	0.51	0.45	0.39	0.35	0.31
2-1	2.438	2.240	2.311	2.325	2.356	2.375	2.377	91.9	94.8	95.4	96.6	97.4	97.5
2-2	2.438	2.256	2.319	2.317	2.359	2.379	2.381	92.5	95.1	95.0	96.8	97.6	97.7
2-3	2.438	2.264	2.332	2.341	2.371	2.372	2.389	92.9	95.7	96.0	97.3	97.3	98.0
Avg.								92.4	95.2	95.5	96.9	97.4	97.7
Std.								0.50	0.43	0.50	0.33	0.14	0.25
3-1	2.449	2.243	2.311	2.324	2.361	2.383	2.383	91.6	94.4	94.9	96.4	97.3	97.3
3-2	2.449	2.240	2.318	2.329	2.358	2.381	2.380	91.5	94.7	95.1	96.3	97.2	97.2
3-3	2.449	2.265	2.315	2.324	2.367	2.366	2.372	92.5	94.5	94.9	96.7	96.6	96.9
Avg.								91.8	94.5	95.0	96.4	97.0	97.1
Std.								0.56	0.14	0.12	0.19	0.38	0.23

TABLE A.78 Core Data for Project TN-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.459	2.231	2.289	2.278	2.305	2.316	2.298	90.7	93.1	92.6	93.7	94.2	93.5
1-2	2.459	2.212	2.284	2.282	2.320	2.312	2.293	90.0	92.9	92.8	94.3	94.0	93.2
1-3	2.459	2.189	2.276	2.274	2.303	2.308	2.303	89.0	92.6	92.5	93.7	93.9	93.7
Avg.								89.9	92.8	92.6	93.9	94.0	93.5
Std.								0.86	0.27	0.16	0.38	0.16	0.20
2-1	2.467	2.221	2.272	2.266	2.305	2.297	2.304	90.0	92.1	91.9	93.4	93.1	93.4
2-2	2.467	2.222	2.285	2.301	2.315	2.330	2.288	90.1	92.6	93.3	93.8	94.4	92.7
2-3	2.467	2.267	2.293	2.298	2.311	2.318	2.290	91.9	92.9	93.1	93.7	94.0	92.8
Avg.								90.7	92.6	92.8	93.6	93.8	93.0
Std.								1.07	0.43	0.79	0.20	0.68	0.35
3-1	2.464	2.295	2.312	2.306	2.327	2.351	2.327	93.1	93.8	93.6	94.4	95.4	94.4
3-2	2.464	2.294	2.318	2.323	2.335	2.355	2.330	93.1	94.1	94.3	94.8	95.6	94.6
3-3	2.464	2.263	2.310	2.312	2.335	2.317	2.309	91.8	93.8	93.8	94.8	94.0	93.7
Avg.								92.7	93.9	93.9	94.7	95.0	94.2
Std.								0.74	0.17	0.35	0.19	0.85	0.46

TABLE A.79 Core Data for Project UT-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.470	2.269	2.331	2.327		2.317	2.313	91.9	94.4	94.2		93.8	93.6
1-2	2.470	2.287	2.339	2.336		2.331	2.323	92.6	94.7	94.6		94.4	94.0
1-3	2.470	2.246	2.314	2.320		2.310	2.295	90.9	93.7	93.9		93.5	92.9
Avg.								91.8	94.3	94.2		93.9	93.5
Std.								0.83	0.52	0.32		0.43	0.57
2-1	2.458	2.310	2.310	2.316		2.297	2.302	94.0	94.0	94.2		93.4	93.7
2-2	2.458	2.313	2.319	2.323		2.318	2.296	94.1	94.3	94.5		94.3	93.4
2-3	2.458	2.270	2.323	2.296		2.309	2.328	92.4	94.5	93.4		93.9	94.7
Avg.								93.5	94.3	94.0		93.9	93.9
Std.								0.98	0.27	0.57		0.43	0.69
3-1	2.465	2.220	2.211	2.224		2.315	2.302	90.1	89.7	90.2		93.9	93.4
3-2	2.465	2.220	2.300	2.238		2.249	2.292	90.1	93.3	90.8		91.2	93.0
3-3	2.465	2.244	2.297	2.287		2.326	2.298	91.0	93.2	92.8		94.4	93.2
Avg.								90.4	92.1	91.3		93.2	93.2
Std.								0.56	2.05	1.34		1.69	0.20

TABLE A.80 Core Data for Project WI-1

Sample	Gmm	Roadway Core - Gmb						Roadway Core - %Gmm					
		In-Place	3-Month	6-Month	1-Year	2-Year	4-Year	In-Place	3-Month	6-Month	1-Year	2-Year	4-Year
1-1	2.563	2.409	2.406	2.412	2.421	2.425	2.407	94.0	93.9	94.1	94.5	94.6	93.9
1-2	2.563	2.320	2.368	2.392	2.402	2.408	2.410	90.5	92.4	93.3	93.7	94.0	94.0
1-3	2.563	2.338	2.391	2.377	2.389	2.386	2.383	91.2	93.3	92.7	93.2	93.1	93.0
Avg.								91.9	93.2	93.4	93.8	93.9	93.6
Std.								1.84	0.75	0.69	0.63	0.76	0.58
2-1	2.558	2.408	2.427	2.448	2.434	2.420	2.417	94.1	94.9	95.7	95.2	94.6	94.5
2-2	2.558	2.397	2.434	2.426	2.437	2.432	2.430	93.7	95.2	94.8	95.3	95.1	95.0
2-3	2.558	2.367	2.416	2.393	2.423	2.405	2.421	92.5	94.4	93.5	94.7	94.0	94.6
Avg.								93.5	94.8	94.7	95.0	94.6	94.7
Std.								0.83	0.35	1.08	0.29	0.53	0.26
3-1	2.546	2.351	2.403	2.398	2.414	2.413	2.408	92.3	94.4	94.2	94.8	94.8	94.6
3-2	2.546	2.326	2.370	2.363	2.403	2.396	2.390	91.4	93.1	92.8	94.4	94.1	93.9
3-3	2.546	2.339	2.362	2.359	2.393	2.397	2.403	91.9	92.8	92.7	94.0	94.1	94.4
Avg.								91.9	93.4	93.2	94.4	94.3	94.3
Std.								0.49	0.85	0.84	0.41	0.37	0.36