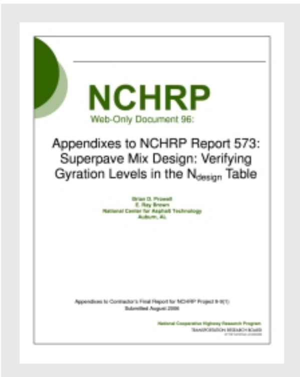
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DETAILS

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Appendixes to NCHRP Report 573: Superpave Mix Design: Verifying Gyration Levels in the N_{design} Table

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Appendixes to Contractor's Final Report for NCHRP Project 9-9(1)
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National Cooperative Highway Research Program
TRANSPORTATION RESEARCH BOARD
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APPENDIX A

LITERATURE REVIEW

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

A.1 A BRIEF HISTORY OF HMA MIX DESIGN PRIOR TO SUPERPAVE

A.1.1 Proprietary Mixes

The first asphalt pavement constructed in the United States (U. S.) was built in Newark, New Jersey in 1870 (*1, 2*). This pavement was constructed with asphalt binder and rock asphalt imported from Europe (*2*). In 1876, President Grant appointed a commission of the U. S. Army Engineers to recommend paving materials for Washington, D. C (*1*). 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 (*2*). 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 (*1*).

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 (*1*) 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 (*1, 2*). 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 (*3*). Richardson proposed the following specification for sheet asphalt (*4*):

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 (*1, 4*). 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 (*5*).

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 (*1, 2, 6*). A patent for "Warrenite" pavement, which incorporated a thin layer of sheet asphalt laid on top of hot Bitulithic pavement soon followed (*1, 5*). 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 (*1, 2, 6*). Davis (*6*) 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 (*6*) 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 (*2*):

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).

A.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 (*1, 2, 5, 7*).

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.

A.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 Sacramento, 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 (*1, 8*). 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 (*9*) [later this was reduced to 200 times gravity (*7*)], 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 (*9*). 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 (*1*, *9*).

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 (*9*). 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 (*8*) 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 (*7*).

Vallerga and Lovering (*8*) 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 A.1.

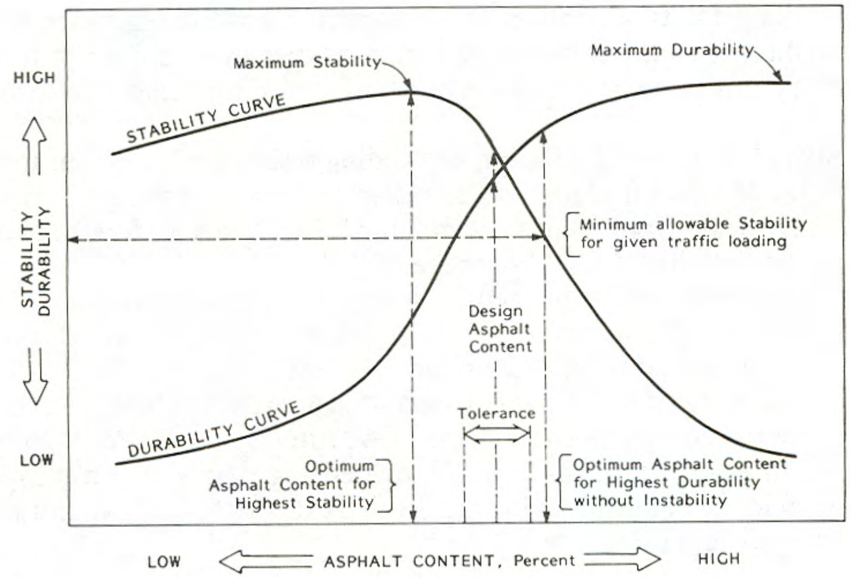

Figure A.1. Stability and Durability as a Function of Asphalt Content (*8*).

A.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 (*7*). 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 (*2*). 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 (*10*).

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 (*11*).

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 (*10*). At this time the Marshall method had been used by some southern states for up to four years (*11*).

In the first phase of the study begun in 1943 (*10)*, 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 (*10*):

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 A.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.

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.

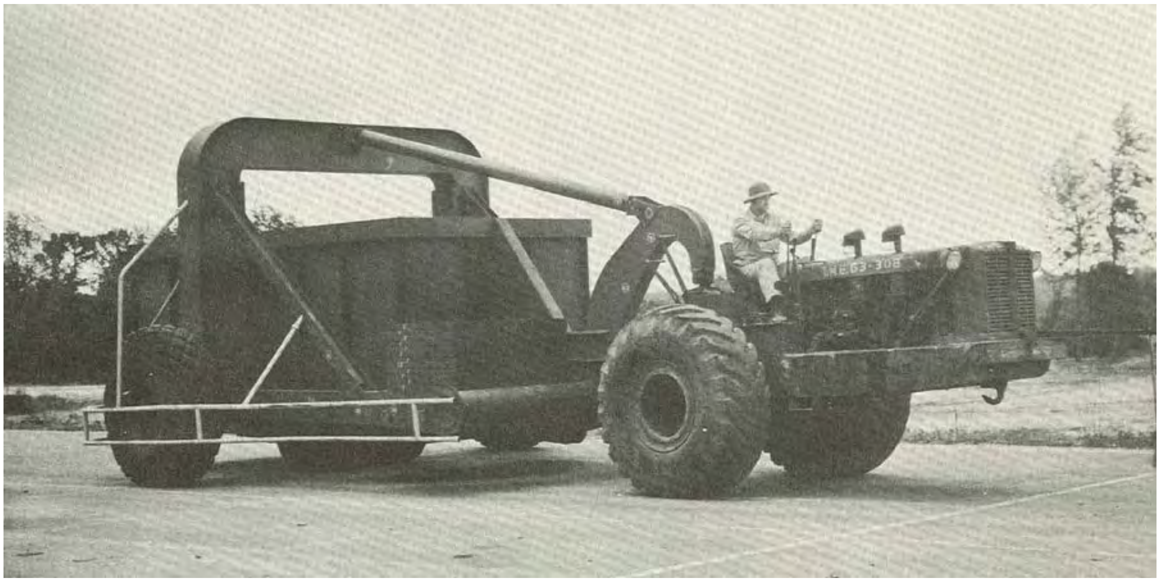

Figure A.2. Model C Tournpull with Specially Built Loading Cart (*10*).

The following is a summary of the conclusions from the Corps study which relate to this current study (*10*):

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 A.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.

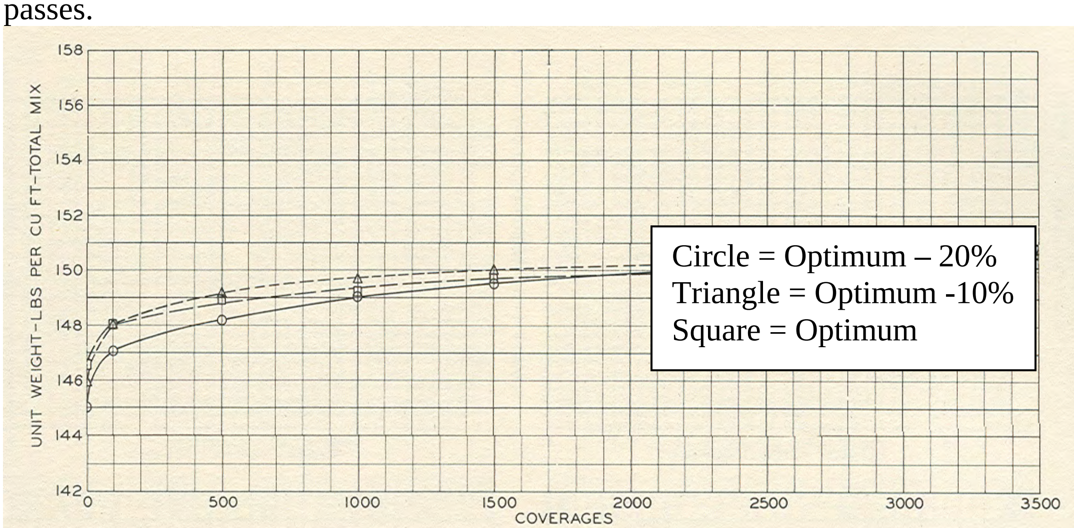

Figure A.3. Traffic Compaction Data for Mix 11, Crushed Limestone with Medium Filler Content (*10*).

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 (*10*) 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 (*11*), 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 (*12*). 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 (*13*). 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 (*14*).

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 (*15*). 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 (*16*). By 1984, 38 out of 50 states were using the Marshall mix design procedure to design HMA.

Leahy and McGennis (*2*) 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."

A.1.5 Texas Gyratory Method

In 1939, the Texas Highway Department initiated a research program into the design and field control of HMA (*17*). 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 (*17*) 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 A.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.

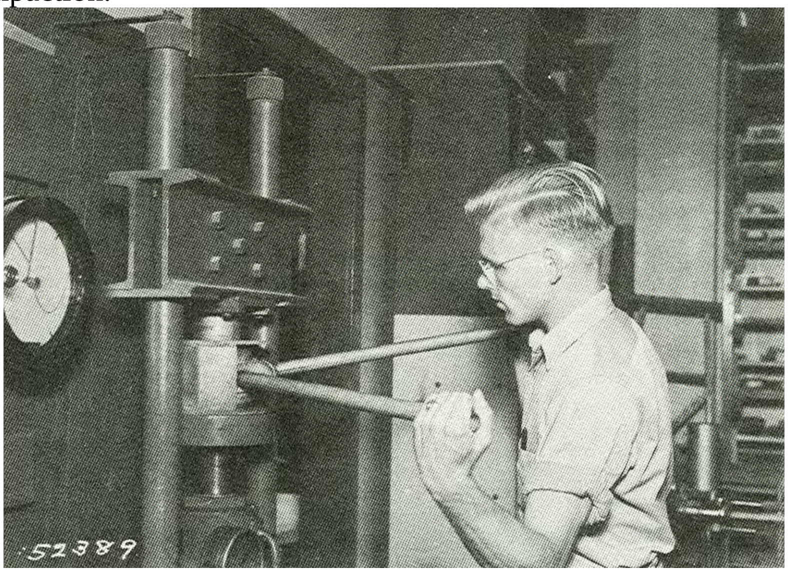

Figure A.4. Manual Texas Gyratory Molding Machine (*17*).

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 (*17*, *18*).

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 (*19*)." 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 (*20*).

2.1.6 Corps of Engineers Gyratory Compactor

McRae (*21*) 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 A.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

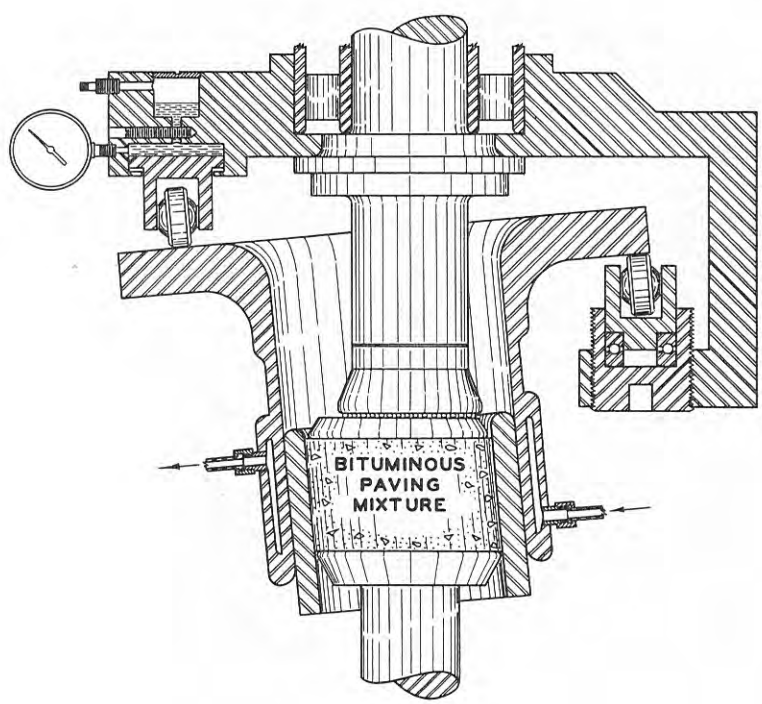

Figure A.5. Schematic of Compaction Head for Corps of Engineers Gyratory Compactor (*21*).

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 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 A.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 A.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 A.6).

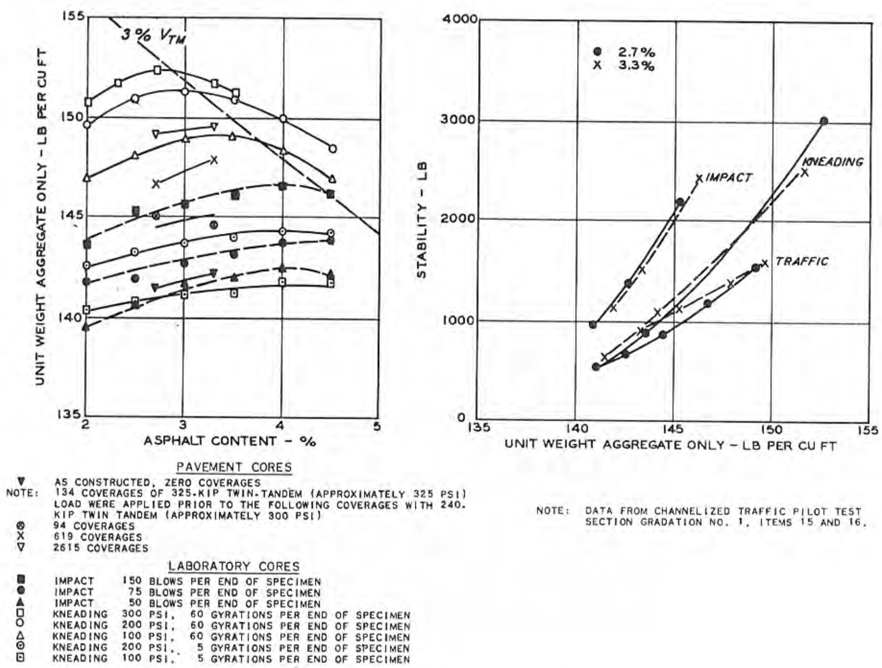

Figure A.6. Comparison of Laboratory and Field Density and Stability Values (*21*).

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 A.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 (*22*), 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.

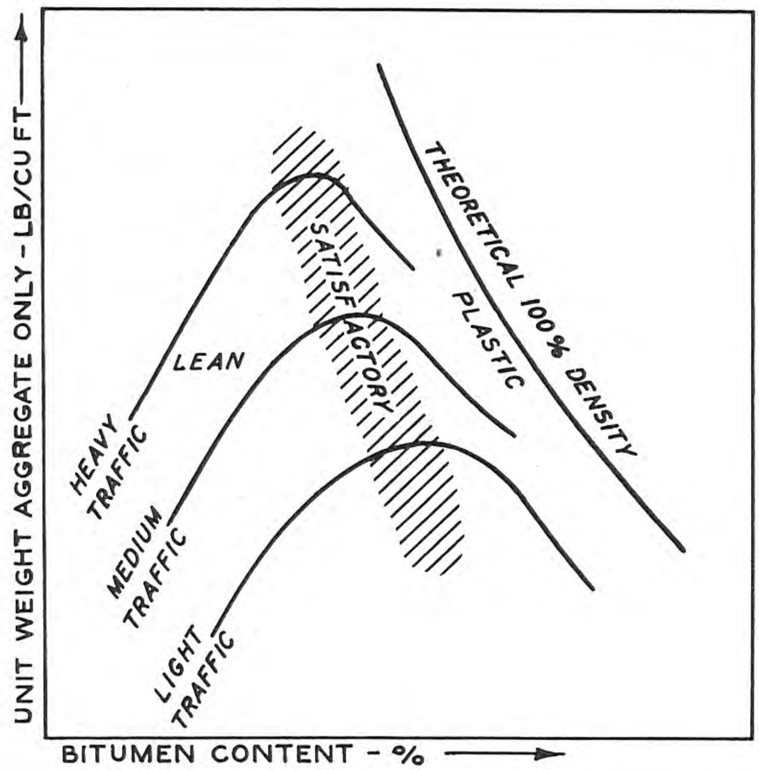

Figure A.7. Aggregate Density as a Function of Asphalt Content and Compaction Level (*21*).

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) (*19*)". 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" (*23*).

2.1.7 French Design Procedure

Bonnot (*24*) 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 A.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.

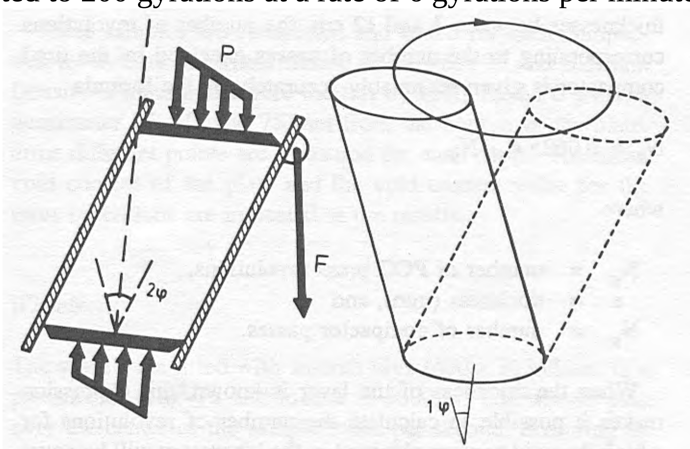

Figure A.8. Compaction Principle of the PCG (*24*).

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.

A.2 SUPERPAVE GYRATORY COMPACTOR

A.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. (*25*) note, "compaction is considered the single most important factor affecting the performance of asphalt pavements. Hughes (*26*) 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. (*27*) 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. (*27*) 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 (*28*). 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. (*27*). 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 (*27*):

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. (*28*) 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 (*28*) 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 (*29*), 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 (*25*). 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 (*25*) 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 (*25, 30*).

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 (*25*). 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 (*30*).

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 (*30*).

Based on the AAMAS study, the research conducted by Button et al. (*30*) 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 (*25*). 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 (*22*) 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 (*25*). 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.

Harman et al. (*31*) provide a concise overview of the evolution of gyratory compaction of HMA. The development of the design compaction level, N_{design} will be discussed later in the report.

A.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 (*25*). 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 (*32*) in reference (*25*), three parameters 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 A.9. Changes in sample asphalt content are expected to affect the compaction curve as illustrated in Figure A.10.

The results of the experiment to compare the three gyratory compactors are shown in Table A.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.

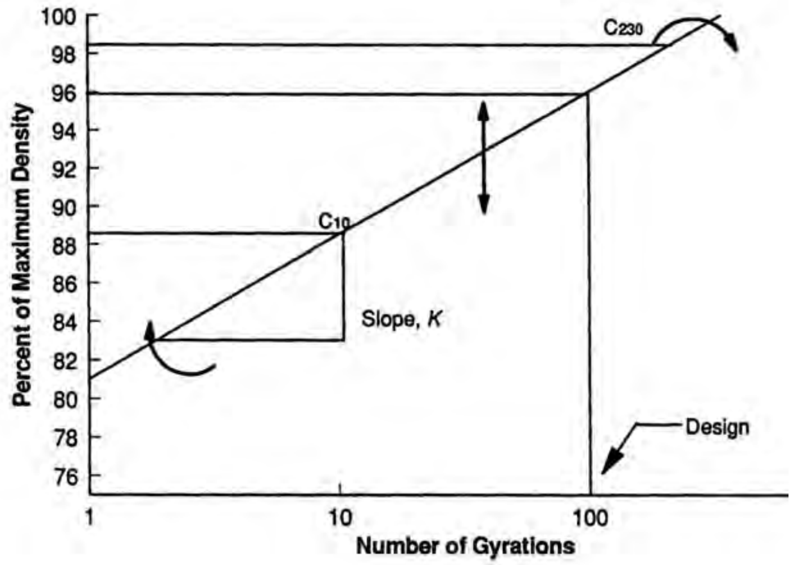

Figure A.9. Typical Gyratory Compaction Curve (*25*).

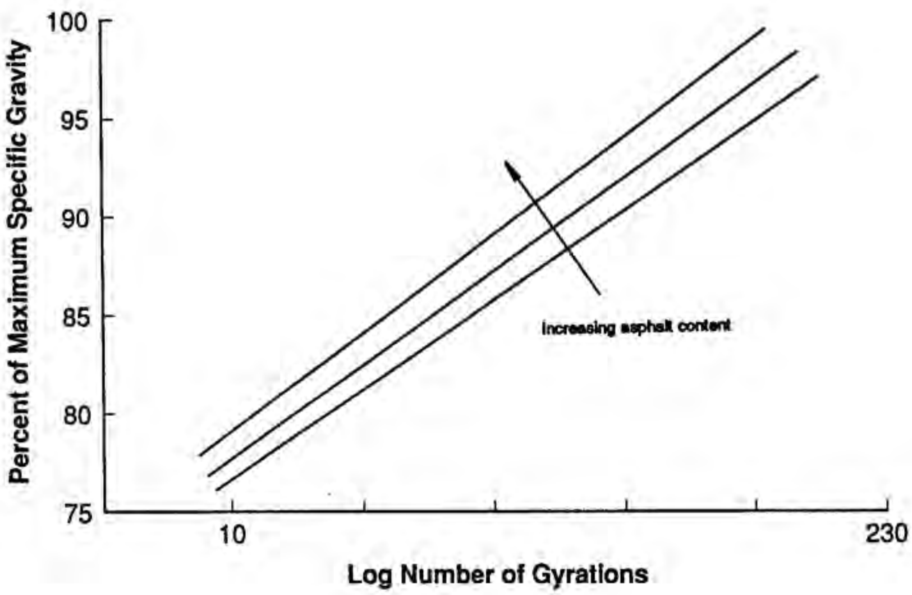

Figure A.10. Effect of Asphalt Content on Compaction (*25*).

TABLE A.1 Comparison of Densification Parameters from Gyratory Compactors (*25*)

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. (*25*) concluded, “A variation in the angle of compaction of ± 0.02 degrees resulted in an air voids variation of ± 0.22 percent 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 degrees.

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 (*33, 34*). A study conducted by the Asphalt Institute (*35*) 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 N_{design}, 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 (*36*). 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 N_{design} experiment (*25*). 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 (*36*).

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 (*36*). 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 (*36*).

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 (*33*). 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)" (*33, 34, 37*). 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 (*33*). When evaluating new models, both Pine and Troxler performed the AASHTO PP35 comparisons against their respective original compactor (*34, 38*).

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 (*39*) 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 $N_{initial}$. 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 N_{design}. The Interlaken SGC produced the largest differences from the average density of all of the compactors.

A.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 (*25*). 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 (*40*). A draft procedure (*37*) 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 A.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 (*40*). Figure A.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 (*40, 41*). Therefore, δ_T and δ_B, as measured by the DAVK, should be averaged to determine an effective internal angle of gyration (δ_{AVG}) (*40 - 42*). 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 (*40*).

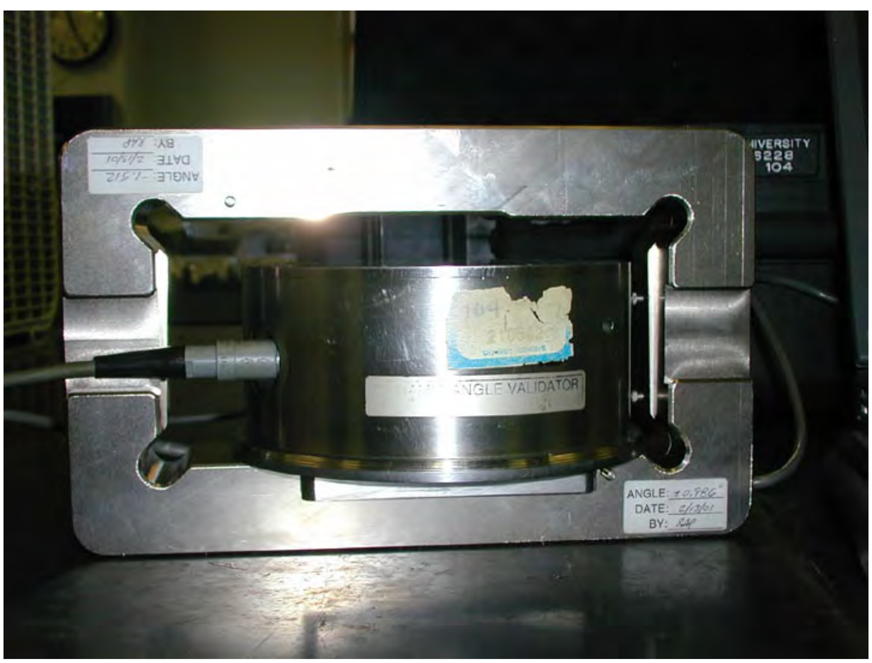

Figure A.11. DAVK and Calibration Block.

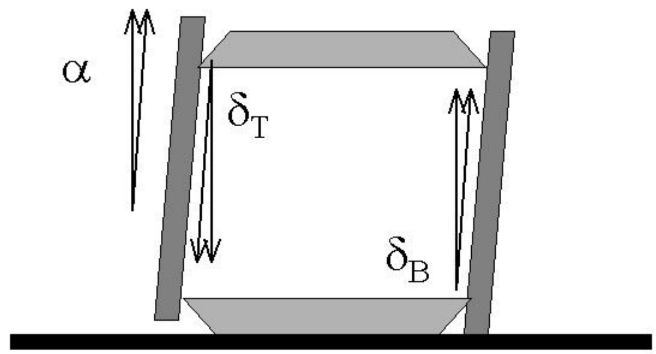

Figure A.12. Definition of Internal and External Angle of Gyration.

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 (*41, 43*) 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 (*41*) 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 G_{mb} 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 G_{mb} 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 (*44*) 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. (*45*) 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 G_{mb} 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. (*46*) 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 A.13 shows the average internal angle of gyration versus the average G_{mb} 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 G_{mb} excluding 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 A.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.

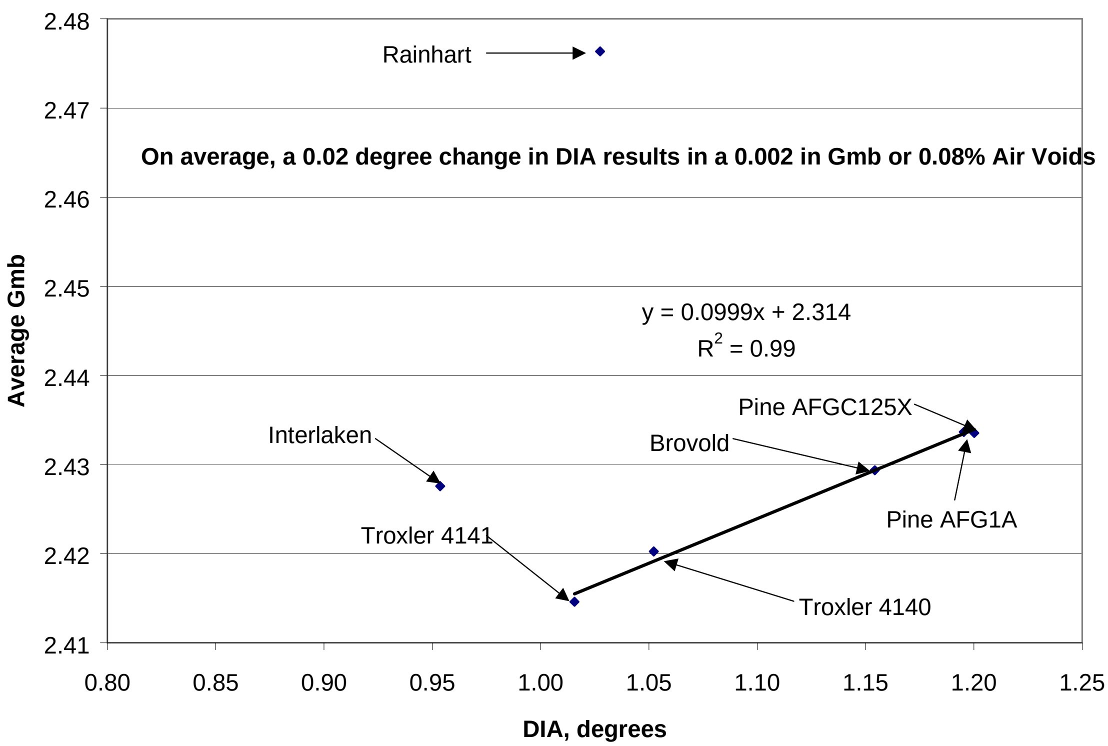

Figure A.13. G_{mb} versus Average Internal Angle of Gyration (*46*).

A.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 (*10*). 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 (*47*).

Dillard (*48*) 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 A.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 A.14).

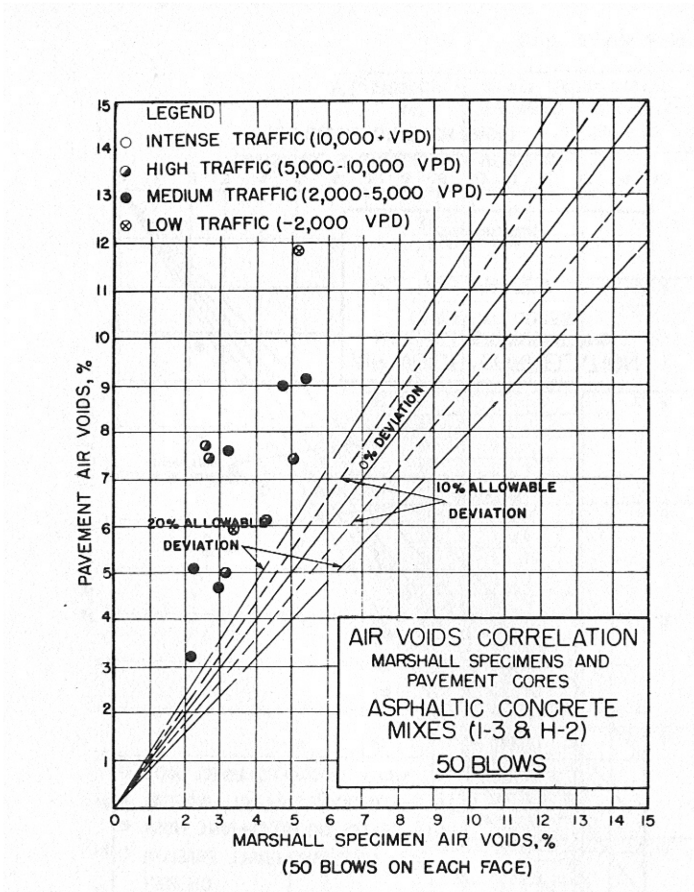

Figure A.14. 50-Blow Marshall versus In-Place Densities (*48*)

Campen et al. (*49*) 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 (*49*), “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. (*50*) 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 (*51*) 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. (*52*) 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. (*53*) 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. (*54*) reported on a continuation of the study conducted by Graham et al. (*50*) 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. (*55*) 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 (*55*):

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 (*55*), "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 A.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 A.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.

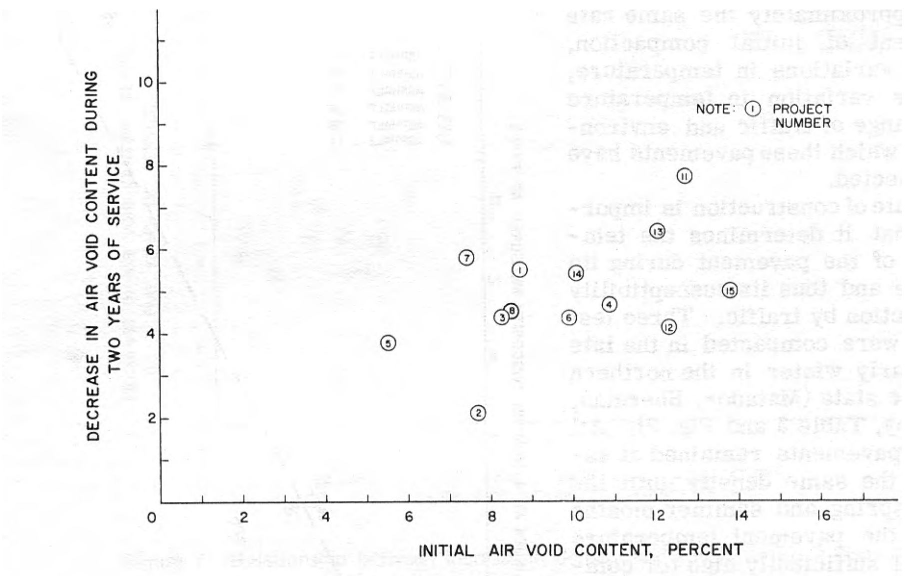

Figure A.15. Densification as a Function of Initial Density (*55*).

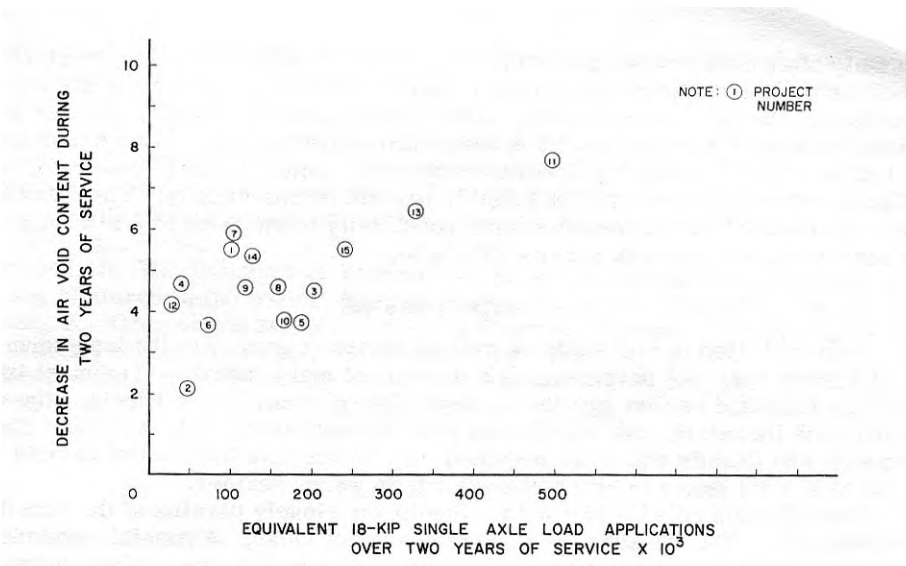

Figure A.16. Densification versus ESALs (*55*).

Kandhal and Wenger (*56*) 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 (*57*) 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. (*58*) revisited 5-pavement sections that were included in the Asphalt-Aggregate Mixture Analysis System study (*28*), 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. (*59*) 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 (*60*) 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.

A.4 STUDIES RELATED TO N_{design}

A.4.1 Development of the Original N_{design} Table

The original N_{design} experiment was conducted by the Asphalt Institute as Task F of SHRP contract A001 (*61*). 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 (N_{design}), and 2) match the as-constructed density, targeted as 92 percent density ($N_{construction}$). The specifications for the SHRP Gyratory Compactor were discussed previously (*25*). Sections from the Long-Term Pavement Performance (LTPP) Studies General Paving Sections (GPS) were selected to determine N_{design} and $N_{construction}$. 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 $N_{construction}$ gyrations since only approximately 30 gyrations would be required to obtain 92 percent density.

Three hypotheses were identified for the experiment (*61*):

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 (*61*):

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 N_{design} values.

The experimental matrix is shown in Table A.2. Two replicates (different pavements) were desired for each cell. The selected pavements were to be at least 12 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. The maximum design traffic included in the experiment was 32.1 million ESALs.

TABLE 2.2 Experimental Matrix for Original N_{design} Experiment (*61*)

Lift	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

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

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 (*61*) 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: $N_{construction}$ = 92 percent density and N_{design} = 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 N_{design} for the lower lifts was not reported. Figure A.17 shows a comparison between the N_{design} levels determined at an angle of 1 and 1.3 degrees.

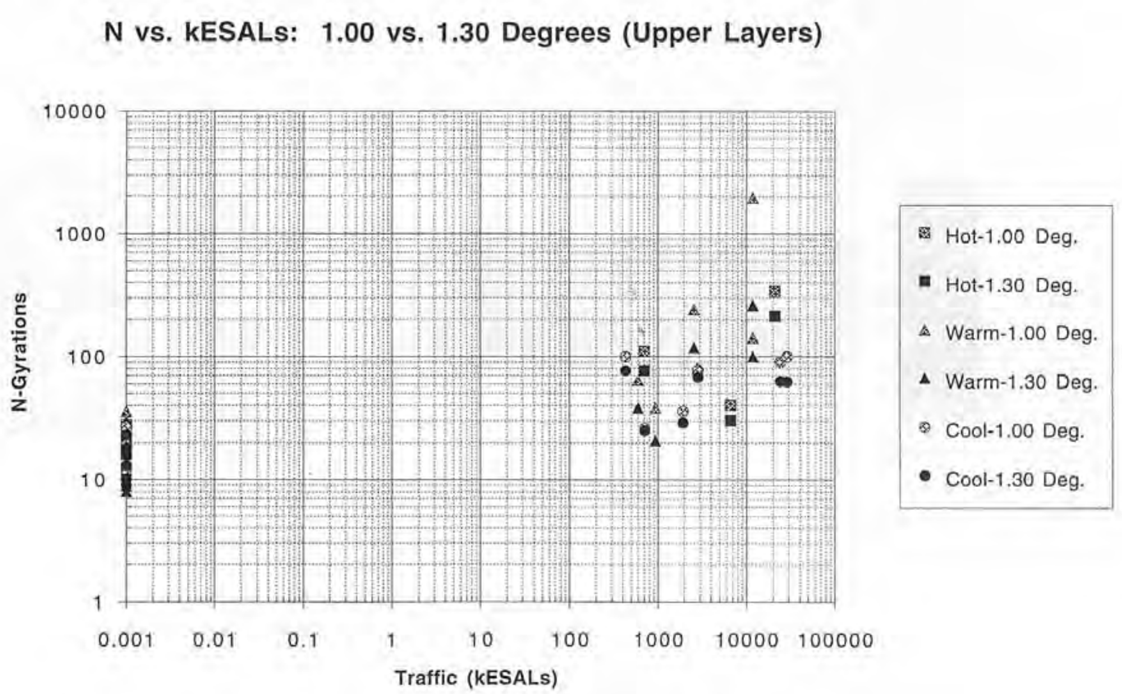

Figure A.17. Comparison of N_{design} from Angles of 1 and 1.3 Degrees (*64*)

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 A.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 A.3 N_{design} Models (*61*)

Climate	Model	R^2	ANOVA *P-value*
Hot	$N_{design} = 10^{1.34276+0.10850\times \text{Log (Traffic, ESALs)}}$	0.66	0.05
Warm	$N_{design} = 10^{1.26454+0.11206\times \text{Log (Traffic, ESALs)}}$	0.69	0.00
Cool	$N_{design} = 10^{1.21211+0.09148\times \text{Log (Traffic, ESALs)}}$	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 (*61*).

The next step in the development of the original Superpave N_{design} table was the determination of the numbers of gyrations for $N_{initial}$ (then termed N_{89}) and $N_{maximum}$ (then termed N_{98}) for each of the traffic levels and climatic zones (*25*). This was accomplished by translating the original compaction curves horizontally until the density at N_{design} corresponded to 96 percent (Figure A.17). This translation is based on some of the principles investigated by Moultier (*32*). The ratio of Log ($N_{maximum}$) to Log (N_{design}) and the ratio of Log ($N_{initial}$) to Log (N_{design}) was determined for each compaction curve. The average ratios were 0.47 and 1.22 for $N_{initial}$ and $N_{maximum}$, respectively. Based on this work, SHRP recommended the following equations (*25*):

$$Log\ Ninitial = 0.45 \times Log\ Ndesign \quad (4)$$

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

The density at $N_{initial}$ was specified as less than 89 percent to prevent tenderness during compaction and the density at $N_{maximum}$ 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 (*25*). 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. (*25*) 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 N_{design} 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 N_{design} 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 A.4 presents the original N_{design} table. The authors have never seen documentation of the decision to go from the three climatic levels presented by Blankenship (*61*) to the four levels provided in the original table. The N_{design} gyration levels for the 43 to 45 °C climate match the gyrations levels for the hot climate determined by Blankenship (*61*). The remaining levels appear to be interpolated.

TABLE 2.4 Original N_{design} Table (*62*)

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 $N_{maximum}$ and the density at N_{design} and $N_{initial}$ back calculated using the sample heights recorded by the SGC (Equation 6).

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

This is a simplified version of Equations 3-6, 3-7, and 3-8 presented by Cominski (*62*), produced by combining terms.

A.4.2 Research Related to N_{design} 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 N_{design} levels. Sousa et al. (*63*) 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. (*25*), 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 N_{design} 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. (These authors note 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. (*64*) 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. N_{design} 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 N_{design} of 126 gyrations and 50-blow Marshall and comparison between N_{design} of 113 and 75-blow Marshall samples.

Gowda et al. (*65*) conducted a study to evaluate the sensitivity of volumetric properties and optimum asphalt content to the Superpave N_{design} levels resulting from variations in design traffic and climate. The authors were concerned by the small differences in N_{design} between some traffic and climate levels (Table A.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 ($N_{maximum}$ for > 100 million ESALs with a 7-day maximum air temperature of 43 to 45 °C). The volumetric properties at the 27 N_{design} levels were back calculated from these samples. Figure A.18 shows the calculated VMA as a function of N_{design}. Note that for a given gradation, VMA changes by approximately 0.3 percent for a change in N_{design} of 10 gyrations.

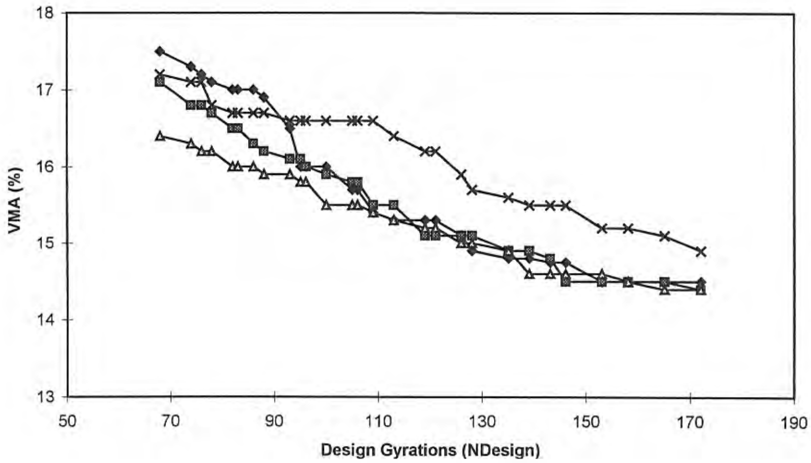

Figure A.18. Variation in VMA with N_{design} for PG 64-22 (*65*).

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 N_{design} levels for differing design traffic which differ by 1 to 2 gyrations do not result in significantly different mix properties and that N_{design} levels from differing climates do not result in significantly different mix properties for a given traffic level.

Habib et al. (*66*) 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 $N_{maximum}$ = 104 gyrations. Volumetric properties were back calculated at N_{design} = 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 N_{design} levels for low volume pavements are approximately 20 percent too high.

Mallick et al. (*67*) 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 $N_{maximum}$ and the volumetric properties back calculated at N_{design}. The back calculation uses a correction factor which is the ratio of the measured G_{mb} using AASHTO T166 to the G_{mb} 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 A.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 N_{design} level, particularly for coarse graded mixes. This resulted in a slight reduction in optimum asphalt content for samples compacted to N_{design} as opposed to those compacted to $N_{maximum}$ where volumetric properties were back calculated at N_{design}.

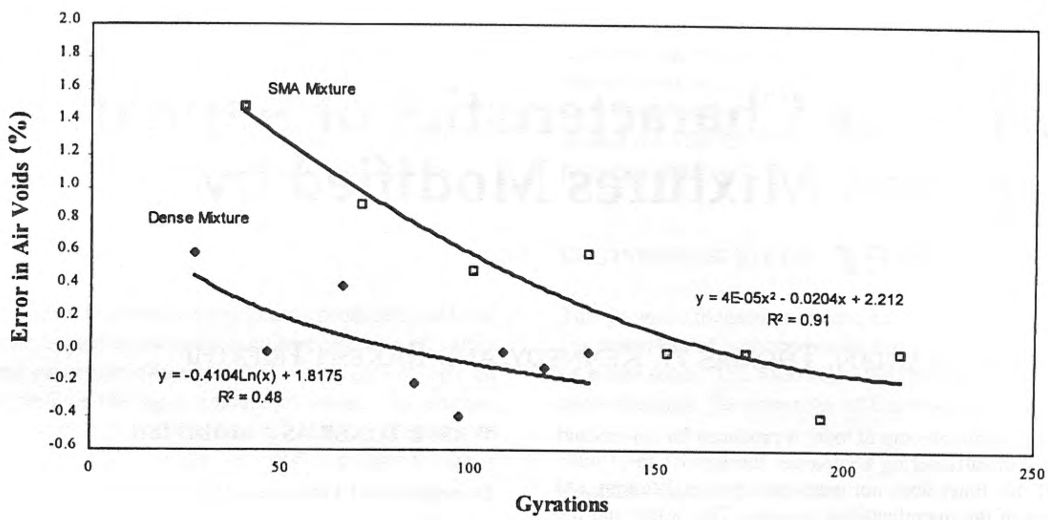

Figure A.19. Error in Back Calculated Air Voids Versus Gyration Level (*67*).

Brown and Mallick (*60*) reported on a preliminary study to evaluate the N_{design} 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 N_{design} for each project. The results from one project, I-90 in Idaho, were discarded since it began to rut after two years. The N_{design} values from this study predicted to match the in-place density after three years were approximately 30 gyrations less than those determined during SHRP. (These authors note 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 (*68*) performed an initial evaluation of N_{design} for the Arizona DOT. The authors note three concerns about the SHRP N_{design} experiment:

1. The angle of gyration used to develop the original N_{design} table was 1 degree, but an angle of gyration of 1.25 degrees was later selected by SHRP without modifying the N_{design} table,
2. The original N_{design} experiment was performed using 100 mm diameter specimens whereas SHRP later specified 150 mm diameter samples,
3. The mixes used in the original N_{design} study were predominately fine graded whereas coarse graded mixes were more predominant when Superpave was first implemented,
4. The N_{design} study was based on only two cores per project (actually one (*61*), there were two cores per cell except for the hot climate).

The authors present a comparison of the N_{design} levels determined in the original N_{design} experiment (Table A.5) based on Reference (*61*) for angles of gyration of 1 and 1.3 degrees. Notice that N_{design} is between 27 and 46 gyrations less at an angle of gyration of 1.3 degrees.

TABLE A.5 Comparison of N_{design} Levels for Hot Climate for 1 and 1.3 Degrees (*68*)

Design Traffic (Million ESALs)	Predicted N_{design}	
	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 $N_{maximum}$ using a Troxler SGC after which the sample densities were back calculated at N_{design}. The G_{mb} 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 N_{design} value for 1.3 degrees. This reduced the difference between the laboratory compacted samples to 0.012 G_{mb} 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 (*69*) 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 N_{design} compaction matrix could be consolidated from the original 28 levels determined during SHRP, and secondly to evaluate the back calculation of N_{design} from $N_{maximum}$. The first objective was evaluated by examining the effect of N_{design} 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 N_{design} 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 N_{design} 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 N_{design}. The samples were compacted to N_{design} (not $N_{maximum}$). Separate samples were compacted to $N_{maximum}$ for three N_{design} 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 N_{design}; 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., N_{design}, 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 N_{design} levels. The analyses were conducted separately for the coarse-graded and the fine-graded mixes. For both gradations, the differing N_{design} 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 N_{design} 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 N_{design} 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 N_{design} levels as a function of climate were eliminated from the N_{design} table, collapsing the table from 28 to 7 levels.

Since the coarse-graded mixes were more sensitive to N_{design} than the fine graded mixes were, the VMA results for the coarse-graded mixes were evaluated to further consolidate the N_{design} table. The average difference in VMA between N_{design} 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 N_{design} 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 N_{design} were proposed 70, 100 and, 130 gyrations. A fourth N_{design} level, 50 gyrations, was proposed for low volume roads.

None of the mixes included in this study failed the $N_{maximum}$ criteria. Further, it was determined that compacting samples to $N_{maximum}$ and back calculating the volumetric properties at N_{design} can result in errors of up to 0.8 percent air voids. Therefore, it was recommended that samples be compacted to N_{design} for the determination of volumetric properties. Separate samples could be compacted to $N_{maximum}$ after the optimum asphalt content is determined. Table A.6 presents the revised N_{design} table recommended by Buchanan (69).

TABLE A.6 Revised N_{design} Table Proposed by Buchanan (*69*)

Design Traffic Level (million ESALs)	Gyration Levels			% G_{mm} @ $N_{initial}$	% G_{mm} at $N_{maximum}$
	$N_{initial}$	N_{design}	$N_{maximum}$		
<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. (*70*) conducted an evaluation N_{design} based on the sensitivity of engineering properties to changes in N_{design}. 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 N_{design} table,
2. Select a validated performance test for rutting,
3. Determine the sensitivity of the performance test to changes in N_{design},
4. Recommend a new N_{design} table.

Six Superpave mix designs were developed using two aggregate types, crushed limestone and crushed gravel, and three N_{design} 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 N_{design} 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. Based on data reported in the paper, the limestone mixes were 65, 60 and 36 percent stiffer than the gravel mixes when designed at 130, 100 and 70 gyrations, respectively. 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 lower, 18 percent for the limestone mixes and 3 percent for the gravel mixes, for both aggregate mixtures designed with N_{design} = 70 gyrations as compared to N_{design} = 100 gyrations. There was a general trend of decreasing shear stiffness with decreasing N_{design}. It was believed that this trend is related to changes in the aggregate skeleton. [Alternatively, these authors believe 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. (*27*) 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 N_{design} levels.

The second part of the study looked was conducted to examine the sensitivity of VMA to N_{design}. In this phase, the mixes which were designed at one gyration level were compacted at the other gyrations levels without adjusting the asphalt content or gradation. This resulted in varying VMA and consequently air voids. Similar results to NCHRP 9-9 were noted. Good correlations were found between air voids and G*. The mixes were most sensitive in the range of 3 to 6 percent air voids with an increase in air voids from 4 to 5 percent resulting in an average decrease in stiffness of 20 percent. 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 N_{design}. Based on that meeting, a new N_{design} table was recommended and adopted by AASHTO in 2001. The revised N_{design} Table from AASHTO PP28 is shown below (Table A.7) (*33*). In 2004, AASHTO PP28 was adopted as AASHTO M323 (*71*).

TABLE 2.7 Superpave Gyratory Compaction Effort (*33*)

Design ESALs (millions)	Compaction Parameter $N_{initial}$	N_{design}	$N_{maximum}$
< 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 N_{design} and one level above and one level below the specified N_{design}. 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 A.20 shows a comparison between the laboratory compacted air voids at N_{design} 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 N_{design} 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.

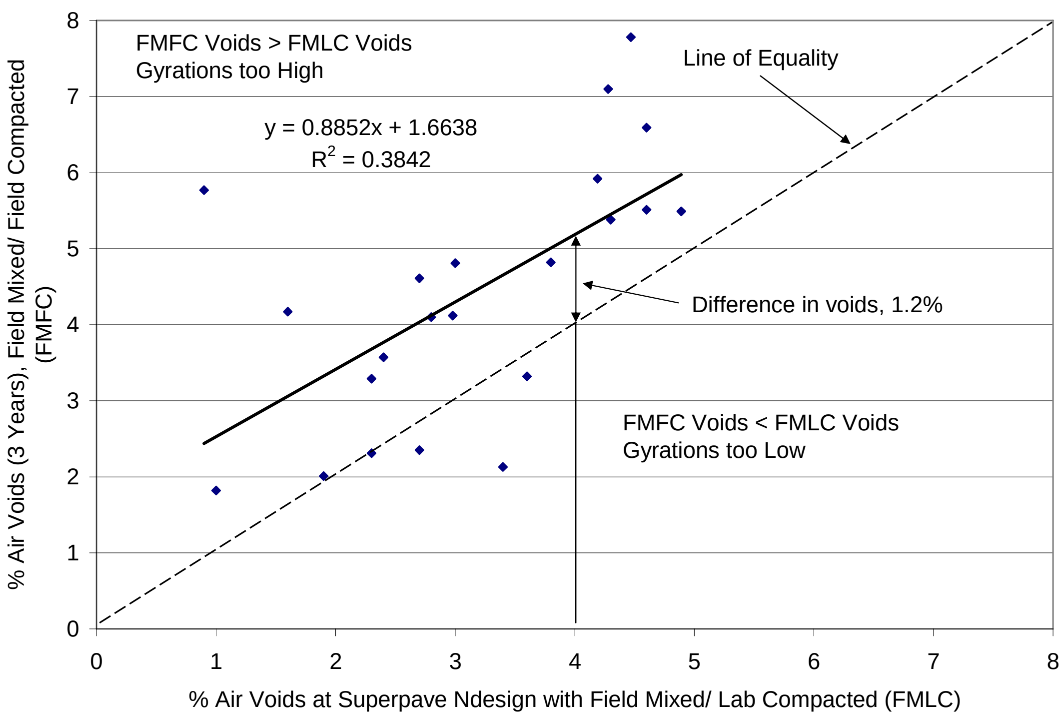

Figure A.20. Comparison of N_{design} and In-Place Air Voids after 3 Years (*72*)

Watson et al. (*73*) conducted a study to verify the N_{design} 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 A.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 A.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

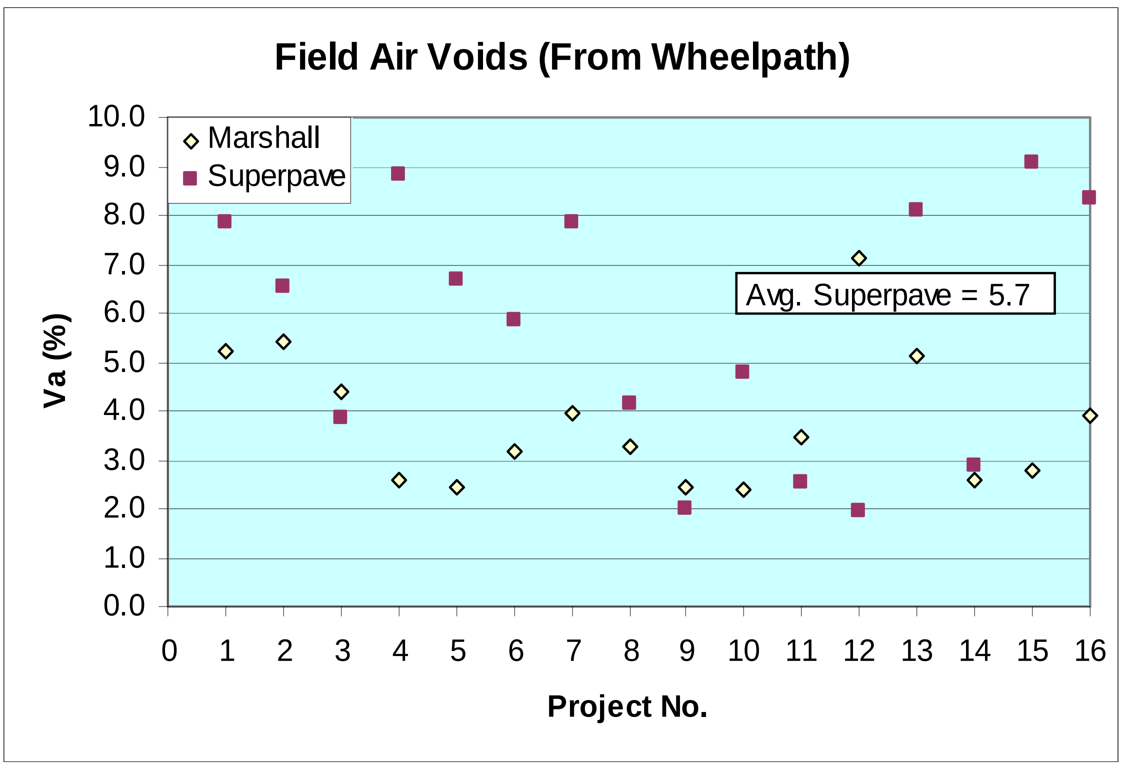

Figure A.21. Comparison of Superpave and Marshall in-place Air Voids (*73*)

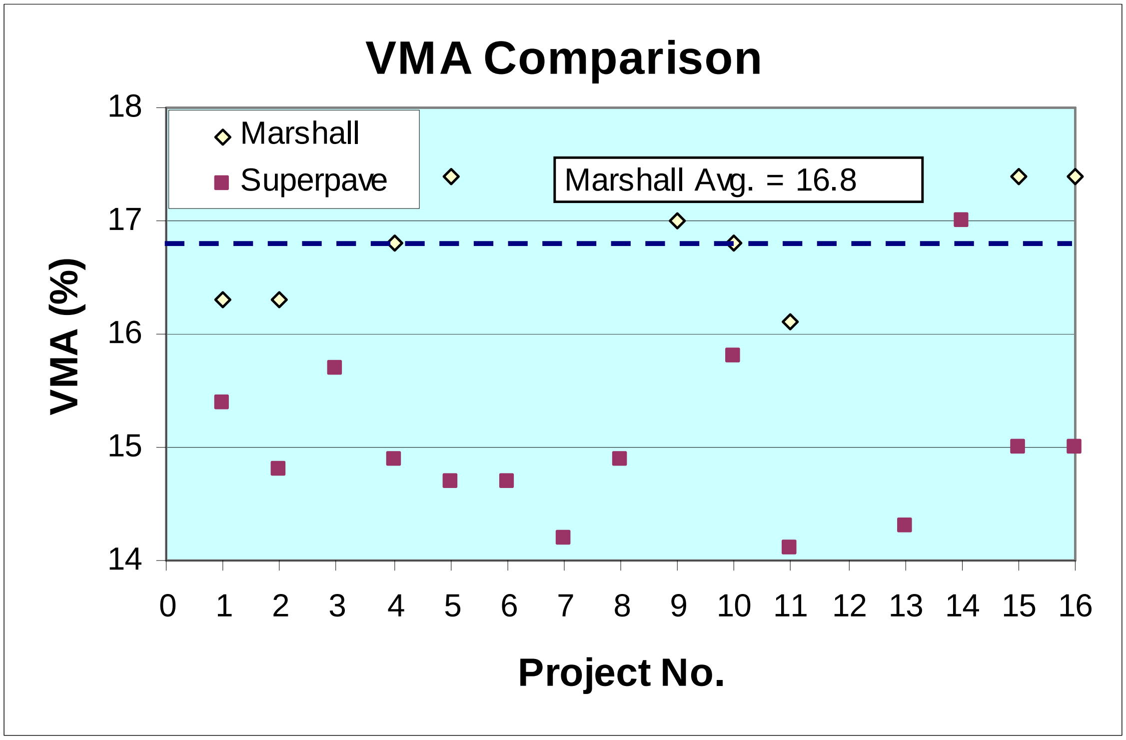

Figure A.22. Comparison of Superpave and Marshall Design VMA (*73*)

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.

Huber and Anderson (*74*) provide a concise history of asphalt mix design through early post SHRP attempts to verify the Superpave N_{design} table using density at the end of service life. The authors state that density at the end of service life cannot be used to define N_{design}. Two examples are given to prove this point. In the first example, a small experiment to verify the N_{design} levels is described. Cores were taken from eight projects at the time of construction. The sites were cored a second time at a later date. The mix was reconstituted and compacted in the gyratory compactor. The number of gyrations to match the in-place density was determined for both coring times. The number of gyrations to produce 96 percent of G_{mm} was calculated or extrapolated. Assuming one ESAL for the time of construction and knowing the traffic estimate for the second coring interval, the field densification was extrapolated to determine the number of ESALs to produce 96 percent density in situ. A plot was produced relating the number of gyrations to reach 96 percent density (N_{design}) and the number of ESALs required producing 96 percent density in situ. The author's state (*74*), "The data is clustered at two extremes. Three of the sites yielded reasonable design gyrations of 80 to 160 gyrations but unreasonable design ESALs of 10^9 to 10^{18}. At the other end of the spectrum, five of the sites yielded design ESALs of 500,000 to 2,000,000 but very unreasonable design gyrations of 9 to 30." There are a number of potential fallacies in this analysis. First due in part to the aging of the binder in-situ, a pavement may reach its ultimate density before reaching 96 percent density and hence produce the extremely high estimates of the design ESALs. Secondly, if the as-constructed density is low, the predicted N_{design} gyrations may also be low, particularly if the mix is stiff and resistant to densification under traffic. In addition, number of summers the pavements were exposed to under traffic would affect both the in-place density and the predicted ESALs. If these intervals were not held constant (not reported in the paper) it could result in errors in the calculations.

In the second example, the author's demonstrate the effect of binder stiffness on field densification. Stiffer binders densify at a slower rate. Data is presented from a project near San Antonio, Texas and from the 2000 NCAT Test Track (*74*). This differential rate of densification is not evident in the SGC since mixes are compacted at equiviscous compaction temperatures. The author's also summarize data from the N_{design} II experiment conducted by the Asphalt Institute. This study was discussed previously in reference (*70*). Again it is worth noting that a change in N_{design} of 25 gyrations altered the mixture stiffness by 30 percent. Similarly, an increase of one high temperature binder grade, say from a PG 64-22 to a PG 70-22, increased mixture stiffness by 30 percent.

Huber and Anderson (*74*) provide a good discussion on constant strain versus constant stress compaction. At the time of construction, a roller applies a constant stress to the pavement. This stress can be increased, within limits, by increasing ballast, increasing tire pressure on a rubber tire roller, or by vibration. The resulting strain will vary with the temperature of the mixture and other factors. Densification under traffic is also constant stress. Heavy vehicles apply a relatively constant stress to the pavement. The resulting strain will vary with temperature (for a given pavement section), with higher strains in the summer and very low strains in the winter. The SGC is a constant strain compaction device and operates at higher strains than those typically observed in the field. These differences certainly complicate comparisons between laboratory densification in the SGC and field densification.

Finally, Huber and Anderson (*74*) provide a discussion on the influence of N_{design} on mixture properties. Table A.8 summarizes the effects. The author's point out that decreasing N_{design} may not increase the design asphalt content. This is because asphalt content is truly governed by VMA. If N_{design} is decreased, the measured VMA for a given mix (aggregate and gradation) will increase. However, the designer will most likely adjust the mix to reduce the VMA closer to the minimum in order to reduce cost.

TABLE A.8 Effect of Design Compaction on Mixture Properties (*74*)

Property	Increased N_{design}	Decreased N_{design}
Coarse Aggregate Angularity	Increased demand for crushed aggregate	Reduced demand for crushed aggregate or no change
Fine aggregate angularity	Reduce natural sand	Reduced need for manufactured sand or no change
Gradation	Change to increase VMA	Change to reduce VMA or no change
Air Voids	No effect	No effect
Voids in Mineral Aggregate	No effect after mix adjustment	No effect after mix adjustment
Voids filled with asphalt	Little or no change	Little or no change
Compaction on road	More difficult	Less difficult
Mixture stiffness	Increased stiffness	Decreased stiffness

A5. Locking Point

Pine (*75*) 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 (*76*) discuss errors in the back calculated density from samples compacted to $N_{maximum}$. 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 A.9.

TABLE A.9 Sample Gyratory Height Data Illustrating Locking Point Determination (*76*)

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

A.6 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 (*1, 2*).

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 (*5*). Hveem (*1*) 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 (*1, 8, 10, 11, 17, 18*).

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 (*10, 11*).

Hveem (*1*) 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 (*8, 21, 22*).

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 (*2*).

Numerous studies were conducted to monitor the densification of pavements, in situ (*10, 28, 47 – 60*). 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 (*62*). 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 (*25, 27, 30*). 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 (*24*).

An experiment was conducted during SHRP to determine N_{design} (*25, 61*). 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 N_{design} 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 (*61*). 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 N_{design}. 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 N_{design} levels for three climates and 7 traffic levels was developed (*25, 61*). Later the SHRP researchers expanded this table to 4 climates (*25*). Late in SHRP, the angle of gyration was changed to 1.25 degrees. The N_{design} levels were not altered at this time even though angles had been demonstrated to affect N_{design} (*25*).

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, and therefore lower optimum asphalt contents than the Marshall system did (*60, 64, 66, 68*).

Research indicated that significant differences did not exist between mix properties resulting from many of the N_{design} levels which were close together (*65, 69*). Errors were observed between the density at N_{design} back calculated from $N_{maximum}$, as originally recommended in the Superpave system, and the density of samples compacted to N_{design} (*66, 68*). Significant research was conducted to confirm these findings which resulted in a consolidation of the N_{design} table from 28 to four levels and a change in practice from compacting samples to $N_{maximum}$ and back calculating volumetric properties at N_{design} to simply compacting samples to N_{design} for volumetric property determination.

However, the consolidation of the N_{design} 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 (*68*, *69*, *76*).

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 N_{design} using the SGC. Lower design gyrations or design air void contents were recommended (*71*). 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 (*72*). Studies attempting to relate the density at the end of service life to the density at N_{design} gyrations have been criticized (*73*). Partially this is due to the fact that compaction in the field at the time of construction and under traffic tends to be a constant stress mode where as compaction in the SGC is a constant strain mode. Further, since mixtures are compacted in the SGC at an equiviscous compaction temperature, the SGC does not account for differences in binder stiffness, which have a profound effect in the field (*73*).

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 (*74*).

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

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APPENDIX B

FIELD PROJECT DATA

TABLE B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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	2.352	92.4	96.2	95.1	95.4	95.1
1-2	2.474	2.300	2.388	2.334	2.384	2.396	93.0	96.5	94.3	96.4	96.8
1-3	2.474	2.359	2.381	2.345	2.378	2.397	95.4	96.2	94.8	96.1	96.9
AVG							93.6	96.3	94.8	96.0	96.3
2-1	2.476	2.302	2.410	2.404	2.369	2.418	93.0	97.3	97.1	95.7	97.7
2-2	2.476	2.319	2.386	2.379	2.386	2.399	93.7	96.4	96.1	96.4	96.9
2-3	2.476	2.328	2.396	2.384	2.379	2.391	94.0	96.8	96.3	96.1	96.6
AVG							93.6	96.8	96.5	96.0	97.0
3-1	2.485	2.309	2.392	2.377	2.358	2.387	92.9	96.3	95.7	94.9	96.1
3-2	2.485	2.330	2.389	2.373	2.385	2.387	93.8	96.1	95.5	96.0	96.1
3-3	2.485	2.309	2.378	2.373	2.373	2.400	92.9	95.7	95.5	95.5	96.6
AVG							93.2	96.0	95.5	95.5	96.2

TABLE B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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 B.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

APPENDIX C

ROADWAY CONDITION SURVEYS

AL-1, Highway 157 Southbound, just North of Moulton, Alabama

Two-Year Survey: June 5, 2002

The bridge over Bear Branch Creek, immediately before site 1, was taken out of service 5 weeks before the site was surveyed. Traffic was maintained on sites 2 and 3. The road is a 4-lane divided highway. Access is not limited. Posted speed limit is 65 mph. No rutting was observable. The surface appeared to be dry with numerous popouts. There was also some minor raveling in the form of loss of fines. No cracking was observed. The underlying layer is badly stripped (approximately ¾ inch scab). Little bond strength exists between the overlay and the underlying layer.

Four-Year Survey: July 21, 2004

Site 1 was overlaid due to the bridge replacement observed during the two-year survey. Figure C2 shows an overview of the site. No cracking was observed. The surface exhibited popouts and minor raveling.

Figure C1. Overview of Site 2, AL-1 Highway 157 Moulton, AL.

AL-2, Highway 168 Eastbound, just East of Boaz, Alabama

Two-Year Survey: June 6, 2002

The road is a two-lane road. The posted speed limit was not observed, but is believed to be 55 mph. Site 3 is near a stop sign at the intersection of Double Bridges Road. It is within the area vehicles would be slowing down. No rutting was observable. Some popouts were observed. Some minor raveling in the form of loss of fines was observed. A number of wet spots (Figure C2), typically in the right wheel path or along the longitudinal

Figure C2. Wet Area Indicating Permeability, AL-2 Highway 168 Boaz, AL

joint were observed. No cracking was observed. There appeared to be a good bond with the underlying layer.

Four-Year Survey: July 22, 2004

The pavement condition was similar to the two-year survey. No cracking was observed, some popouts and fractured coarse aggregate were observed and minor raveling was observed.

AL-3, Highway 80, Whitehall, Alabama

Two-Year Survey: July 31, 2002

Highway 80 is a four-lane divided highway, two lanes in each direction. The test sites are located in the westbound travel (right) lane. No rutting was measurable with a four-foot straight edge. Transverse cracks were observed in the shoulder (Figure C3), but not on the mainline pavement. Minor loss of fines were observed.

A four-year survey was not conducted for site AL-3.

Figure C3. Transverse Crack in Shoulder, AL-3 Whitehall, Alabama

AL-4, Highway 84 Eastbound, just east of Monroeville, Alabama

Two-Year Survey: July 24, 2002

The road is a two-lane road (Figure C4). The posted speed limit was not observed, but is believed to be 55 mph. A significant number of logging trucks were observed. No rutting was apparent when measured with a string line. Low severity reflective cracks were observed on approximately 45-foot spacing. The cracks were not particularly straight (Figure C5). Raveling was observed throughout the section, mainly loss of fines. No evidence of popouts. The underlying layer appears to be a chert gravel.

A four-year survey was not conducted for site AL-4.

Figure C4. Overview of AL-4, Highway 84 Monroeville, AL.

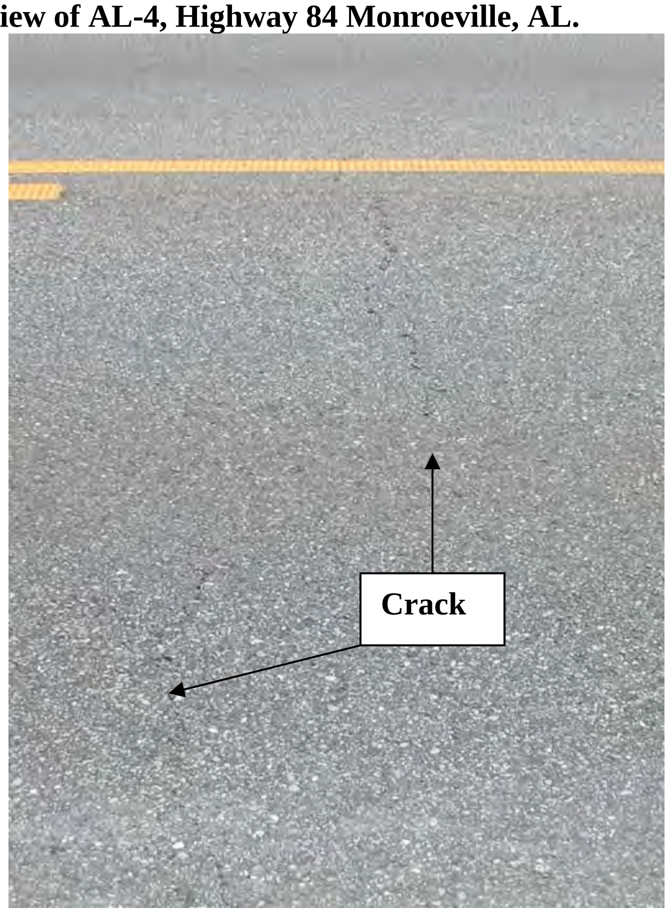

Figure C5. Transverse Reflective Crack, AL-4 Highway 84 Monroeville, AL

AL-5, Highway 167 Southbound, south of Enterprise, Alabama

Two-Year Survey: July 25, 2002

The road is a two-lane road. The posted speed limit is 55 mph. Traffic appeared to be relatively light. It rained during the coring, limiting the number of pictures taken. No rutting was observed with 4-foot level. No cracking or popouts were observed. Examination of the cores indicated very minor raveling. There appeared to be some mica in the aggregate.

A four-year survey was not conducted for site AL-5.

AL-6, Andrew's Road, Opelika, Alabama

Two-Year Survey: September 9, 2002

The road is a two-lane road. Traffic appeared to be relatively light. The road had serves as an access route to a Walmart distribution facility. However, two other more direct routes exist, one a recently completed interchange with Interstate 85.

Four-Year Survey: July 23, 2004

The pavement was in good condition.

AR-1, I-40, Conway, AR

Two-Year Survey: September 23, 2003

The road is a four-lane interstate highway. The test sections are in the westbound lane. The centerline joint was slightly open. Minor loss of fines, some popouts and some fractured coarse aggregate were observed.

Four-Year Survey: June 15, 2005

Figure C6 shows an overview of the section. Note the rust stains occurring around some of the coarse aggregate particles. With the exception of the rust stains, the condition of the pavement had not changed between the two- and four-year survey. Figure C7 is a close-up of some of the minor raveling. Cracking of the asphalt was observed under the thermoplastic line markings.

Figure C6. Rust Spots evident in AR-1, I-40 West of Conway, AR.

Figure C7. Close-up of AR-1 Surface Texture after Four Years.

AR-2, I-55, West Memphis, AR

Two-Year Survey: September 22, 2003

The test section is a four-lane interstate highway. The test sections are in the northbound lane. The longitudinal joints were fairly tight although approximately a 0.25 inch crack was observed at the centerline joint. The surface appears to be open. Diamond grinding had been done to remove bumps.

Four-Year Survey: June 14, 2005

Figure C8 shows an overview of the site, taken in 2005. Occasional reflective cracks were observed. This section of I55 was not rubbilized. Some raveling was observed, primarily loss of fines. The longitudinal joints remain fairly tight (Figure C9)

Figure C8. Overview of AR-2, I-55 North of West Memphis, AR.

Figure C9. Centerline Longitudinal Joint, AR-2 after Four Years.

AR-3, I-40, Brinkley, AR

Two-Year Survey: September 22, 2003

The road is a four-lane interstate highway. The test sections are in the eastbound lanes. Longitudinal joints looked good. Visual stripping was observed in the underlying layer.

Four-Year Survey: June 14, 2005

Figure C10 shows an overview of the site. Moisture damage was observed in the underlying layer and in the bottom of the surface layer. No cracking was evident, except in the thermoplastic line markings. The longitudinal joints were still good. Figure C11 shows a close-up of the pavement surface, including some water spots.

Figure C10. Overview of AR-3, I-40 near Brinkley, AR.

Figure C11. Close-up of AR-3 Surface after Four Years.

AR-4, I-30, Prescott, AR

Two-Year Survey: September 24, 2003

The road is a four-lane interstate highway. The test sections are in the westbound lane. Minor raveling was observed along the centerline longitudinal joint.

Four-Year Survey: June 16, 2005

Figure C12 shows an overview of the site. Minor raveling in the form of loss of fines and some coarse aggregate particles was observed. Figure C13 shows a close-up of the pavement surface. No cracking was observed. The longitudinal joint did not deteriorate any further.

Figure C12. Overview of AR-4, near Prescott, AR

Figure C13. Close-up of AR-4 Surface after Four Year.

CO-1, Highway 9, Frisco, Colorado

Two-Year Survey: August 5, 2002

The road is a four-lane road, two in each direction with additional turning lanes. There are three stoplights throughout the stretch containing the sections. Figure C14 shows an overview of Site 1. The pavement surface appeared to have little macro texture both within and out side of the wheel paths (Figure C15). Popouts were observed throughout the section. Rutting was measured with a four-foot metal straightedge. A stepped ramp device developed by Colorado DOT was used to determine the rut depth. Site 3 was located in the approach to a signalized intersection. A single transverse crack was located at site No. 3.

Four-Year Survey: August 23, 2004

Pop-outs were observed throughout the section. The pavement appeared to have a smooth surface texture. Cracking was observed in conjunction with two drainage structures and is believed to be related to settlement. Cracking was also observed in the thermoplastic line markings.

Figure C14. Site 1, CO-1 Frisco, CO.

Figure C15. Surface Texture, CO-1 Frisco, CO

CO-2, Highway 82, Glenwood Springs, Colorado

Two-Year Survey: August 6, 2002

The site is a four-lane road with two lanes in each direction. The test sections are in the southbound travel (right) lane. The pavement surface appears to be tight. Rut depths were measured with a four-foot metal straightedge. The longitudinal joint was open (Figure C16). A gooey, rubbery black substance was found on all of the cores except the first core taken at site No. 3 (Figure C17).

Four-Year Survey: August 24, 2004

No significant change occurred compared to the two-year survey.

Figure C16. Longitudinal Joint, CO-2 Highway 82 Glenwood Springs

Figure C17. Gooey Black Substance on Core, CO-2 Highway 82

CO-3, Interstate 70 Business Route, Grand Junction, Colorado

Two-Year Survey: August 8, 2002

The sites are located on a city street. The street is three lanes wide and carries one-way (eastbound) traffic. The test sections are located in the travel (right) lane. Figure C18 shows an overview of the first site. The right lane ends just past the second site. Therefore, there appears to be more traffic in the middle lane. No rutting was measurable at Site No. 1 or Site No. 2 with a four-foot straight edge. Minor loss of fines was observed in both sections.

Four-Year Survey: August 26, 2004

Seven transverse cracks were observed in the right lane. Five of the cracks were grouped with approximately 15 foot spacing between the cracks. Figure C19 shows one of the cracks. Minor raveling was noted on the pavement surface. The longitudinal joints were tight.

Figure C18. Overview of Site 1 CO-3, I-70 Business Route, Grand Junction, CO

Figure C19. Transverse Crack CO-3.

CO-4, Highway 13, Meeker, Colorado

Two-Year Survey: August 7, 2002

Highway 13 is a two-lane road. Sites No. 1 and No. 3 are in the southbound lane and Site No. 2 is in the northbound lane. Rut depths were measured with a four-foot metal straightedge. A rut depth of 2.5/32 inch (2 mm) was measured at site No. 1. Rut depths of < 1/32 inch (< 1 mm) were measured at sites No. 2 and No. 3. Minor loss of surface fines was observed throughout the section.

Four-Year Survey: August 25, 2004

Figure C20 shows an overview of Site 1. Raveling (loss of fines) had continued to produce a very coarse surface texture. The longitudinal joint had developed a tight crack.

Figure C20. Overview of CO-4, Highway 13, Meeker, CO

Figure C21. Close-up of CO-4 Surface Texture after Four Years.

CO-5, Highway 82, Glenwood Springs, Colorado

Two-Year Survey: August 6, 2002

Site CO-5 is part of the same paving project that included Site CO-2. Highway 82 is a four-lane road with two lanes in each direction. All of the sites are located in the southbound travel lane. Rut depths were measured in the right wheel path with a four-foot metal straightedge. A rut depth of 1.25/32 inch (1 mm) was measured at sites No. 1 and No. 3. A rut depth of 3/32 inch (2.5 mm) was measured as site No. 2.

Four-Year Survey: August 24, 2004

No noticeable deterioration was noticed at CO-5 after four years of traffic.

FL-1, Davis Highway, Pensacola, Florida

Two-Year Survey: August 13, 2002

The road is a four-lane road, two in each direction with additional turning lanes. The section is in an urban area with signalized intersections, strip malls and other businesses. Site No. 1 is located on an uphill grade (Figure C22); site No. 2 is located on the approach to a signalized intersection. Cores from both sites were taken in the left hand lane. The second and third core locations at site No. 1 (1-2 and 1-3) were taken in the left wheel path (at all coring times). All other cores were taken in the right wheel path.

The pavement did not have very much macro texture, but no flushing was observed. The longitudinal joints were tight. Rut depths were measured with a four-foot metal straightedge. Less than 1/16 inch rutting was measured at all of the sites (the evaluator lost his more accurate ruler on the previous project). A single transverse crack was observed. A bottle, paved into the pavement, created a small pothole (Figure C23).

A four-year survey was not conducted on FL-1.

Figure C22. Overview of Site 1 FL-1, Davis Highway, Pensacola, FL.

Figure C23. Small Pothole Caused By Bottle Paved into Pavement, FL-1

GA-1, Highway 13, Duluth, GA

Two-Year Survey: July 29, 2003

The pavement is five lanes wide with two lanes in each direction and a center turn lane. Minor raveling was observed in the form of loss of fines. The longitudinal joints had cracked (Figure C24). A transverse crack was observed near Site 3.

Four-Year Survey: July 21, 2005

The longitudinal joint was open. Random cracks were observed, particularly near some drainage structures at Site 3 (Figure C25). A longitudinal crack was observed at the edge of the old pavement where the pavement had been widened. Raveling was observed, particularly between the wheel paths (Figure C26).

Figure C24. GA-1 Longitudinal Joint after Two-Years.

Figure C 25. Cracking near Site 3, GA-1.

Figure C26. Close-up of GA-1 Surface Texture after Four-Years.

IL-1, Interstate 57, Gilman, IL

Two-Year Survey: July 8, 2003

The road is a four-lane interstate highway. The test sections are in the southbound lane. The HMA is approximately a four-inch overlay over concrete pavement. No reflective cracks were observed. The centerline longitudinal joint was good. The longitudinal joint with the shoulder corresponded to the edge of concrete pavement and had cracked. Minor loss of fines between wheel paths.

Four-Year Survey: June 21, 2005

Longitudinal cracking along shoulder due to underlying edge of concrete pavement (Figure C27). Thermoplastic line markings cracked resulting in cracks in the HMA. Minor raveling between wheel paths and occasional popouts (Figure C28). Occasional transverse cracks (Figure C29).

Figure C27. Longitudinal Cracking along Shoulder IL-1, I-57, Gilman, IL.

Figure C28. Minor Raveling between Wheel Paths, IL-1.

Figure C29. Transverse Crack IL-1.

IL-2, Interstate 64, Collinsville, IL

Two-Year Survey: July 10, 2003

The tests sections are on a four-lane interstate highway in the eastbound lanes. Figure C30 shows an overview of the site. The HMA is an overlay (approximately 3.25 inches thick) over continuously reinforced concrete pavement. A good centerline joint was observed; the joint between the lane and shoulder showed a fine crack corresponding to the edge of concrete pavement. Moisture damage was observed in the underlying layer (Figure C31). A few minor popouts were observed.

Four-Year Survey: June 23, 2005

The longitudinal centerline joint was still tight. No reflective cracking was observed. Minor raveling was observed. The underlying limestone binder layer exhibited visual striping. Problems were observed throughout the district under bridges where the overlays were thinned to 1.5 inches to maintain bridge clearance.

Figure C30. Overview of IL-2, Collinsville, IL.

Figure C31. Moisture Damage in Underlying Layer.

IL-3, Interstate 70, Effingham, IL

Two-Year Survey: July 9, 2003

There are a total of approximately 6 inches of HMA over concrete pavement. Reflective cracks were observed approximately every 90 feet (Figure C32). The cracks were sealed.

Four-Year Survey: June 22, 2005

Longitudinal joint had apparently cracked and was subsequently sealed. Reflective crack spacing was now every 45 to 60 feet. The cracks were sealed. Very minor raveling was observed.

Figure C32. Sealed Reflective Crack IL-3.

IN-1, Highway 136, Brownsburg, Indiana

Two-Year Survey: September 24, 2002

The road is a two-lane road with an additional center turn lane. Two sites, labeled No. 2 and No. 3 are east of Highway 267, and site No. 1 is west of Highway 267. All of the sites are in the westbound lanes. Paving was conducted against traffic. Site No. 1 is on a downgrade heading out of town. Site No. 2 is 100 yards prior to a red light. The wheel path for site No. 3 is close to the left longitudinal joint due to a turn lane.

The maximum rut depth, measured with a four-foot string line was < 1/16 inch at all three sites. No cracking was observed at any of the sites. Loss of surface fines was observed outside the wheel paths.

Five-Year Survey: September 14, 2005

This survey was not conducted until the pavement had been is-service for five years. There is a paving joint in the middle of the lane at site 3. There is cracking at the joint and in the right wheel path with some evidence of slippage (Figure C33). Site 2 also exhibited cracking in the right wheel path.

Figure C33. Longitudinal Cracking Site 3, IN-1.

IN-2, Interstate 69, Auburn, Indiana

Two-Year Survey: September 23, 2002

The site is a four-lane Interstate highway. All three sites were located in the southbound travel (right) lane. The pavement is an HMA overlay over a Portland cement concrete pavement. The maximum rut depth, measured with a four-foot string line, was 1/16 inch at all three sites. A longitudinal crack was observed one foot into the travel lane from the construction joint with the shoulder. Reflective cracks were observed with spacing between 30 and 90 feet. The cracks were sealed. A single popout was observed. Raveling, in the form of loss of surface fines, was observed throughout the section. Moisture damage was observed in the underlying layers.

Four-Year Survey: September 13, 2005

This survey was not conducted until the test section had received five years of traffic. Transverse reflective cracks were evident with approximately a 36 foot spacing (Figure C34). The cracks had been sealed, but the joint sealant had failed. A longitudinal crack was observed just inside the edge line of the pavement. There was raveling, including loss of coarse aggregate, particularly between the wheel-paths (Figure C35).

Figure C34. Transverse Reflective Crack IN-2.

Figure C35. Pavement Texture between Wheel-paths for IN-2.

KS-1, Interstate 70, Hays, KS

Two-Year Survey: September 10, 2003

The pavement is a four-lane interstate highway. The test sections are located in the eastbound lanes. Some raveling was observed in the form of loss of fines, as well as popouts. A longitudinal crack was observed in the right wheel path between sites 2 and 3. A center of wheel path longitudinal crack was also observed with fines coming to the surface.

Four-Year Survey: July 28, 2005

Water was observed coming to the surface through an edge line crack near site 1 (Figure C36). Fines were observed along the crack. The shoulder mix is a different gradation and appears to be denser than the mainline pavement. The underlying layer exhibited moisture damage, particularly the coarse aggregate (Figure C37). Maintenance forces suggested that the westbound lanes were in worse condition. Raveling was observed including loss of coarse aggregate particles as well as popouts (Figure C38). Some rust stains were also observed. Cracking was observed in the passing lane near site 2 (Figure C39).

Figure C36. Edge line Crack with Evidence of Water and Fines KS-1.

Figure C37. Visual Evidence of Moisture Damage in Underlying Layer KS-1.

Figure C38. Popout in Surface KS-1.

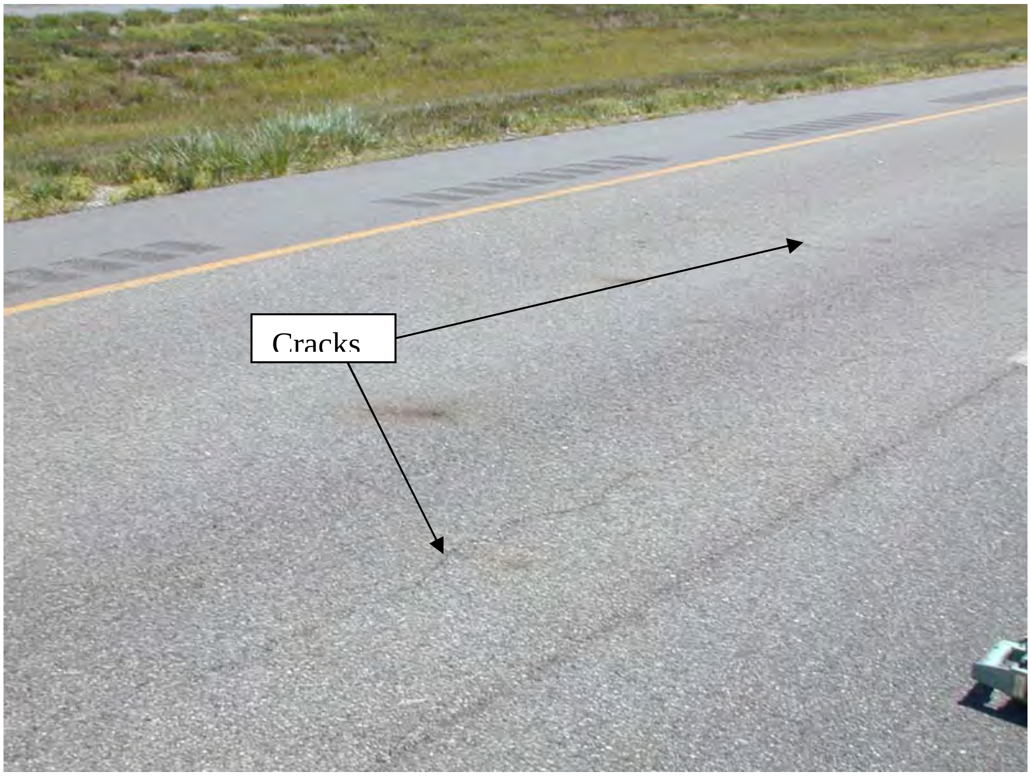

Figure C39. Cracking in Passing Lane and Rust Stains KS-1.

KY-1, County Road (CR) 1796, Lebanon, Kentucky

Two-Year Survey: October 22, 2002

CR1796 is a rural two-lane (no centerline stripe) road approximately 20 feet wide. The geometry of the road and traffic volume is such that the wheel paths are not clearly defined. Site No. 1 is in the lane heading away from KY 555 and Site No. 2 is in the lane heading toward KY 555. No rutting was found when the wheel paths were checked with a string line. Some popouts were observed.

Four-Year Survey: October 14, 2004

Numerous popouts were observed during the four-year survey. Some transverse cracking was observed, particularly near drainage structures. Cracking was also observed at the edge of pavement.

KY-2, Interstate 64, Olive Hill, Kentucky

Two-Year Survey: October 22, 2002

This section of I-64 is a four-lane divided highway with two lanes in each direction. The sites are in the eastbound travel (right) lane. No cracking and only extremely minor loss of fines was observed. The longitudinal joint was tight. Rutting was measured in the right wheel path at each site using a four-foot string line. Site No. 1 had 1/32 inch rutting and Site No. 2 had 1/16 inch rutting. Permeability problems (wet spots) were observed in this section of pavement, but not at the test sites.

Four-Year Survey: October 14, 2004

Figure C40 shows an overview of the project. The longitudinal joint was just starting to crack. Some moisture damage was observed in the underlying layers. Raveling was observed, primarily loss of fines (Figure C41).

Figure C40. Overview of KY-2, Interstate 64.

Figure C41. Surface Texture with Raveling of KY-2.

KY-3, CR 1779, Shelbyville, Kentucky

Two-Year Survey: October 22, 2002

CR 1779 is a two-lane road. Sites No. 1 and 3 are located in the eastbound lane; site No. 2 is located in the westbound lane. Minor popouts were observed. The centerline joint was in good condition. Minor edge cracking and a single transverse crack were observed. No rutting was measurable with a four-foot string line at site No. 2 and No. 3. There was a 3/32 inch rut at site No. 1.

Four-Year Survey: October 14, 2004

Some popouts were observed. Some full-width transverse (Thermal?) cracks were observed (Figure C42).

MI-1, Interstate 75, Detroit (Auburn Hills), Michigan

Two-Year Survey: October 1, 2002

All three sites are located in the northbound travel (right) lane. The Interstate is three lanes in each direction. Significant reflective cracking was observed at 36 to 50 foot intervals (Figure C43). At site No. 2, noticeable deflection in the slabs in the travel lane was observed resulting from traffic passing in the middle lane. The pavement structure consists of 4.5 inches of hot-mix asphalt on 9.0 inches of Portland cement concrete (Figure C44). Very minor loss of fines was visible along with a few popouts (Figure C45). Rutting was measured with a four-foot string line as ¼ inch at sites No. 1 and No. 2 and 3/32 inch at site No. 3. Contrary to expectation, site No. 3 was located on an uphill grade.

Four-Year Survey: November 29, 2004

The reflective cracks noted in the two-year survey had been sealed. However, the cracks reflected through the sealer. Minor faulting was also observed. Some parallel cracks were observed at the longitudinal joints. Some popouts were also observed.

Figure C43. Reflective Cracking MI-1

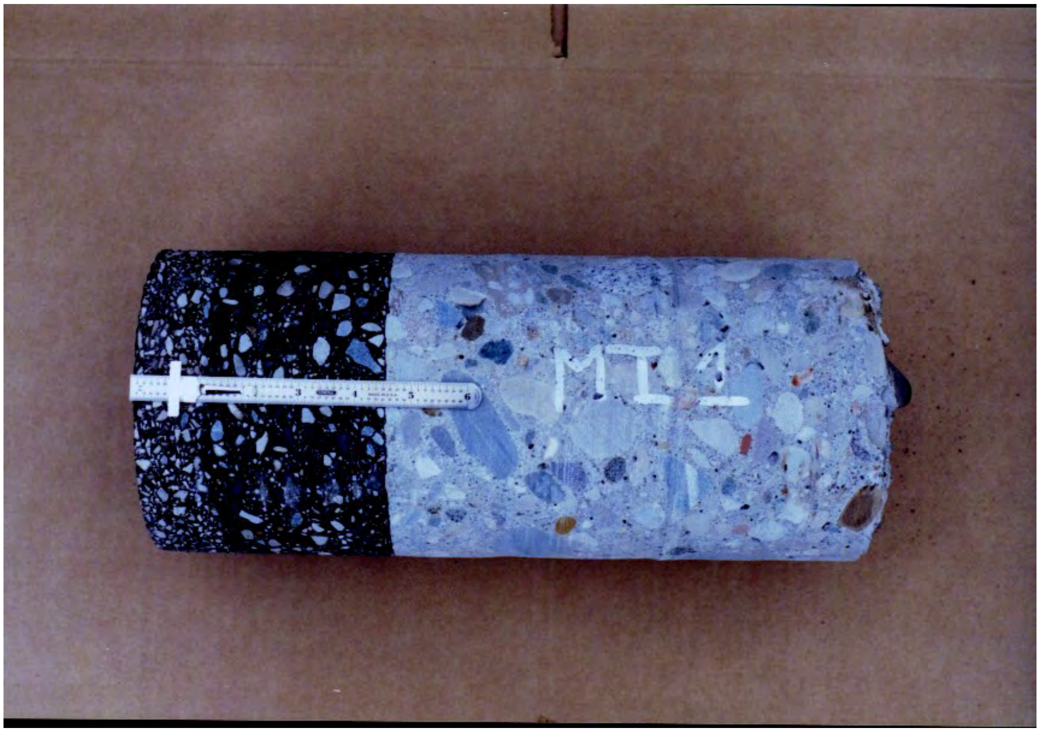

Figure C44. Core of Pavement Structure, MI-1

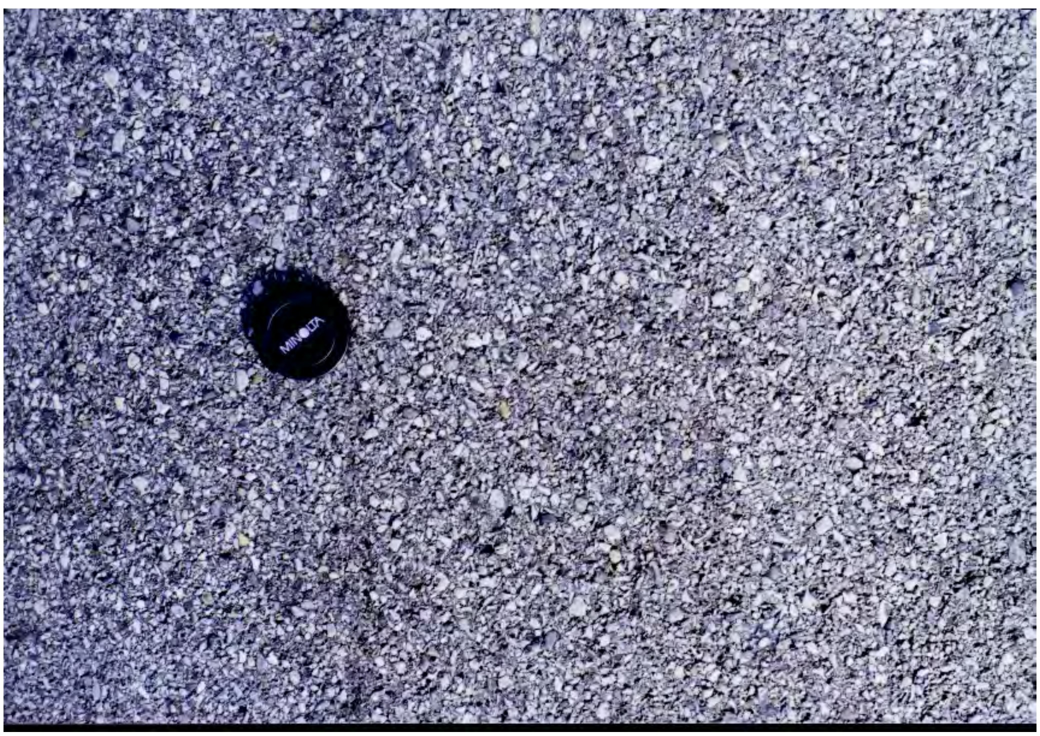

Figure C45. Longitudinal Joint (Right Edge of Lens Cap) and Surface Texture, MI-1

MI-2, State Route M50, Jackson (Brooklyn), Michigan

Two-Year Survey: October 1, 2002

M50 is a two-lane road. The test sites are located in the northbound lane. A few small popouts were observed. Rutting was measured with a four-foot string line. No rutting was observed at sites No.1 and No. 2. A rut depth of 1/32 of an inch was measured at site No. 3.

Four-Year Survey: November 30, 2004

A few small popouts were observed. Transverse cracks were observed with a 12 to 93 foot spacing, averaging 21 to 24 feet apart. The cracks were full width and appeared to be thermal cracks. The transverse cracks were generally sealed.

MI-3, State Route M52, Owosso (east of Lansing), Michigan

Two-Year Survey: October 1, 2002

M52 is a two-lane road. The test sites are located in the southbound lanes. The pavement structure is a composite section with approximately 5.5 inches of hot-mix asphalt on Portland cement concrete (Figure C46). Some reflective cracking, at inconsistent intervals was observed (Figure C47). No rutting was measurable with a four-foot string line.

Four-Year Survey: November 29, 2004

A few popouts were observed. Reflective cracks were also observed.

Figure C46. Core Showing Pavement Structure, MI-3 Note Cracking in Concrete

Figure C47. Reflective Crack, MI-3

MO-1, Interstate 70, Warrenton, MO

Two-Year Survey: September 9, 2003

The pavement is a four-lane interstate highway. The sites are located in the eastbound lanes. Minor raveling was observed along with occasional popouts. There were a limited number of rust stains. The longitudinal joints were tight.

Four-Year Survey: July 26, 2005

A Novachip layer had been placed on the pavement surface due to friction concerns. GPS was used to accurately locate the sites as evidenced by coring through a portion of a previous core. Unfortunately, the wrong layer (the Novachip) was tested for density and the remaining portion of the cores discarded.

MO-2, Highway 65, Springfield, MO

Two-Year Survey: September 11, 2003

Medium severity raveling was observed, particularly between the wheel paths. There were a large number of popouts and the surface aggregate appeared to be shattered. The longitudinal joint near site 3 was raveling. The HMA was an overlay of portland cement concrete. No cracking was observed.

Four-Year Survey: July 27, 2005

Lots of popouts and raveling was observed (Figure C48). No cracking was observed.

Figure C48. Popouts and Raveling MO-2.

MO-3 Interstate 44, Springfield, MO

Two-Year Survey: September 11, 2003

Minor raveling between wheel paths. Transverse settlement between Sites 2 and 3. Longitudinal joints pretty good.

Four-Year Survey: July 27, 2005

Some popouts were observed as well as minor raveling ion the form of loss of fines. The centerline longitudinal joint was slightly open. Some parallel cracks were observed along the edge joint at Site 3 (Figure C49). Slight cracks were observed in the thermoplastic line markings.

Figure C49. Cracking at Edge of Pavement MO-3.

NC-1, Interstate 85, Kings Mountain, NC

Two-Year Survey: June 10, 2003

The test locations are on a four lane interstate pavement. The sites are located in the northbound lanes. Five transverse reflective cracks were observed under the bridge between Sites 1 and 2. Longitudinal crack was observed in the right wheel-path at Site 1 (Figure C50). Minor raveling observed by site 3. A patch was placed just past site 3. Longitudinal joints good. Fragments of wood and rubber were observed in the pavement. Some segregation was observed.

Four-Year Survey: August 9, 2005

Severe raveling had occurred in some of the previously mentioned segregated areas. Transverse cracks were still evident between Sites 1 and 2. Longitudinal crack was observed in right wheel-path at site 1 and between the wheel-paths at Site 3. Extensive cracking was observed in the thermoplastic line markings.

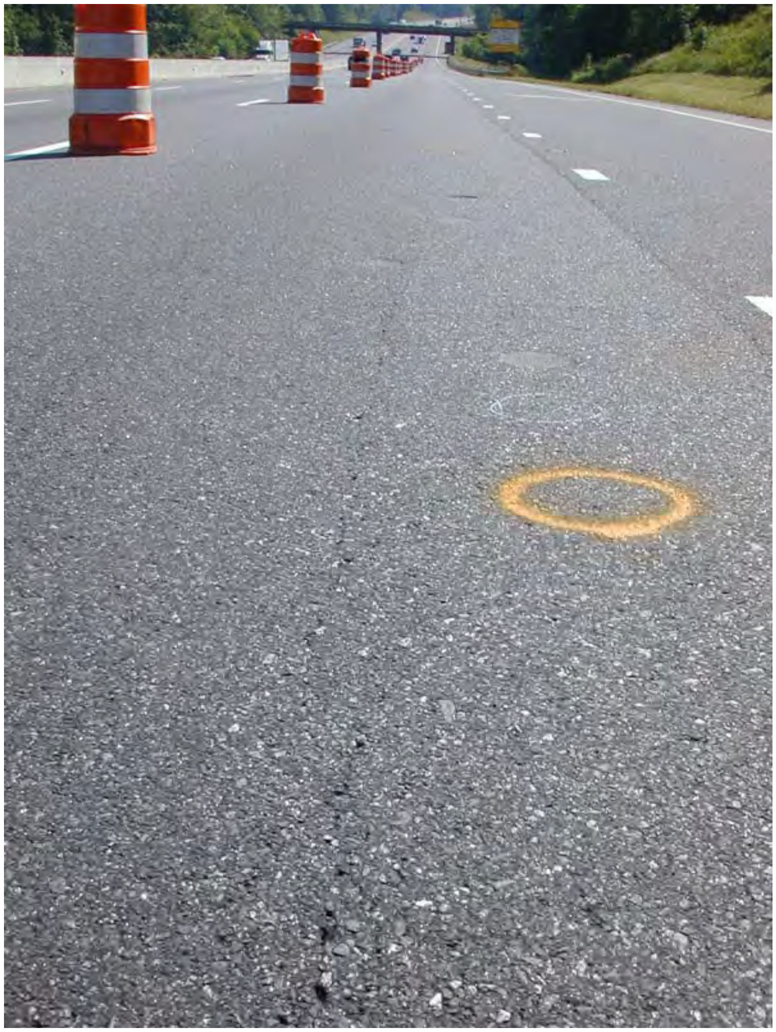

Figure C50. Longitudinal Crack in Right Wheel-Path NC-1.

Figure C51. Raveling in Segregated Area NC-1.

NE-1, Highway 8, Chester, NE

Two-Year Survey: October 29, 2003

Some popouts were observed. Poor bond was observed from cores with lower layer at Site 1. Longitudinal joints appeared good.

Four-Year Survey: August 1, 2005

Popouts were observed throughout the section (Figure C52). No cracking was observed and the longitudinal joint appeared good. Some moisture damage was observed in the underlying layer.

Figure C52. Popouts and Raveling NE-1.

NE-2, Highway 77, Oakland, NE

Two-Year Survey: October 28, 2003

Thermal cracks were observed with a 60 to 70 foot spacing (Figure C53). The cracks were 0.13 to 0.25 inches wide. Longitudinal cracks were observed on both sides of the edge line. The centerline longitudinal joint was cracked in places. A few popouts were observed.

Four-Year Survey: August 2, 2005

Spacing between some of the thermal cracks had decreased to as few as 36 feet. The pavement is reported to be a HMA overlay of PCC. Some reflective cracks had appeared. The cracks were sealed (Figure C54).

Figure C53. Thermal Crack after Two Years, NE-2

Figure C54. Sealed Thermal Crack after Four Years NE-2

NE-3, Highway 8, Superior, NE

Two-Year Survey: October 29, 2003

The test sites are located in the eastbound lanes on a two-lane road. A few shallow popouts were observed.

Four-Year Survey: August 1, 2005

Longitudinal joint was in god condition. No cracking was observed. Some popouts were observed (Figure C55). Moisture damage was observed in the underlying layer.

Figure C55. Popouts, NE-3 after Four Years.

NE-4, Interstate 80, Kimball, NE

Two-Year Survey: October 30, 2003

The centerline longitudinal joint was open. The longitudinal joint with the shoulder was still tight. No evidence of raveling or popouts. The core had a good bond with the underlying layer.

Four-year Survey: August 3, 2005

Transverse cracks observed with approximately a 40 foot spacing (Figure C56). The transverse cracks and centerline longitudinal joints were sealed. The thermoplastic line markings had cracked.

Figure C56. Transverse Crack NE-4 after Four Years.

TN-1, Highway 171, near Nashville, TN

Two-Year Survey: August 5, 2003

The sites are on a two-lane road with a turn lane in some places. No cracking was observed. Very minor raveling was observed between the wheel paths. The longitudinal joints were in good condition.

Four-Year Survey: July 20, 2005

Some raveling was observed as well as fractured coarse aggregate (Figure C57). Longitudinal joints were still in good condition. Cracking was observed in thermoplastic line markings.

Figure C57. Low Severity Raveling between Wheel-path (and elsewhere), TN-1.

UT-1, Highway 150, Kamas, Utah

Two-Year Survey: August 7, 2002

Highway 150 is a two-lane road connecting Kamas and some National Forest lands. The test sites are located in the eastbound lanes. A single layer chip seal was applied to the project prior between the six-month and one-year evaluation. It is standard practice for Utah Department of Transportation to apply a surface seal to all of their hot-mix asphalt pavements within a year of overlay. A plant mix seal coat is applied to high volume roads and a chip seal to low volume roads. Some reflective cracking, or possibly thermal cracking was observed throughout the project (Figure C58). No rutting was measurable with a four-foot string line.

Four-Year Survey: October 24, 2004

Thermal cracks were observed in the westbound lane. The cracks are approximately 10 feet apart at Site 2 (Figure C59). The transverse cracks emanating in the westbound lane propagate into the eastbound lane. The pavement thickness appears to vary with Site 3 being thicker.

Figure C58. Transverse Crack, UT-1.

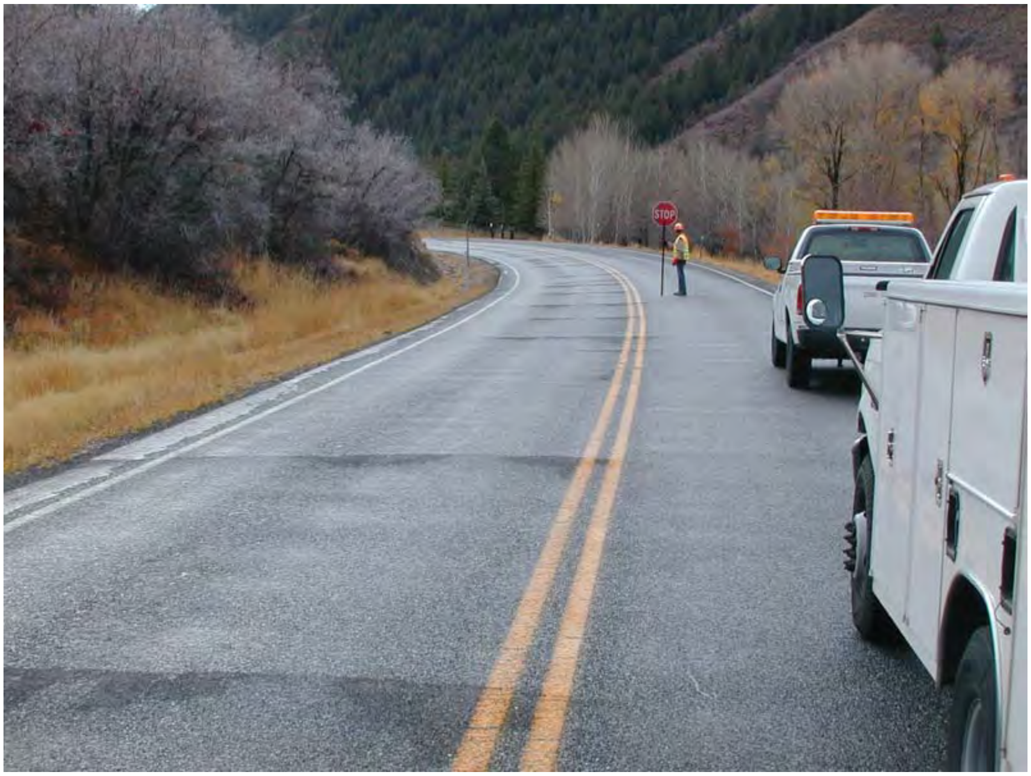

Figure C59. Transverse Cracking in Westbound (non-test) Lane UT-1.

WI-1, U.S. Highway 45, Milwaukee, Wisconsin

Two-Year Survey: October 2, 2002

U. S. Highway 45 is a six-lane road, three lanes in each direction. All of the test sites are in the southbound passing (far left) lane. The pavement structure is a composite section with approximately 4.5 inches of HMA on top of Portland cement concrete. The section has extensive transverse and longitudinal reflective cracking (Figure C60). The majority of the reflective cracks have been sealed. Agency officials stated that no attempt to prevent reflective cracking was considered in the design. There was no measurable rutting with a four-foot string line.

Four-Year Survey: September 26, 2004

Transverse reflective cracks had 60 to 75 foot spacing. Longitudinal joints were cracked as well. Some fat spots were visible on the pavement surface. These may have resulted from tracked crack sealant. The four-year coring was conducted at night and the pictures are not of good quality.

FigureC60. Reflective Cracking WI-1