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Optimizing Laboratory Mixture Design as It Relates to Field Compaction to Improve Asphalt Mixture Durability

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16. Abstract

Most departments of transportation, including Indiana, currently use the Superpave mixture design method to design asphalt mixtures. This method specifies that the optimum asphalt content for a given gradation be selected at 4 percent air voids. During construction, these mixtures are typically compacted to 7-8 percent air voids. If mixtures were designed to be more compactable in the field they could be compacted to the same density as the laboratory mixture design, which would increase pavement durability by decreasing the in-place air voids. The objective of this research was to optimize the asphalt mixture design in order to increase in-place asphalt pavement durability without sacrificing the permanent deformation characteristics of the mixture.

Three asphalt mixtures were designed using the standard Superpave design method at 100 gyrations of the Superpave Gyratory Compactor, suitable for traffic levels of 3 to 30 million Equivalent Single Axle Loads. Each mixture was then used as a starting point to design three additional mixtures using 70, 50, and 30 gyrations, with optimum binder content chosen at 5 percent air voids, rather than the currently specified 4 percent. The effective asphalt content was held constant for the original and redesigned mixtures. Permanent deformation characteristics of the sets of four mixtures were determined by measuring the dynamic modulus and flow number. The results suggest that the mixture designs produced using 70, 50, and 30 gyrations had permanent deformation characteristics equal to or better than the original 100-gyration mixtures.

Based on the laboratory test results, two field trials were placed evaluate the design method, ease of construction and to compare the construction results of the re-designed and original mixtures. Samples from both projects were collected during construction, test specimens compacted, and additional physical testing completed. The field trial results suggest that it is possible to place a mixture at 5 percent air voids and that mixtures designed at 5 percent air voids should have equivalent performance to those designed at the conventional 4 percent air voids.

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EXECUTIVE SUMMARY

OPTIMIZING LABORATORY MIXTURE DESIGN AS IT RELATES TO FIELD COMPACTION TO IMPROVE ASPHALT MIXTURE DURABILITY

INTRODUCTION

Most departments of transportation, including Indiana, currently use the Superpave mixture design method to design asphalt mixtures. This method specifies that the optimum asphalt content for a given gradation be selected at 4 percent air voids. During construction, these mixtures are typically compacted to 7–8 percent air voids. If mixtures were designed to be more compactable in the field they could be compacted to the same density as the laboratory mixture design, which would increase pavement durability by decreasing the in-place air voids. The objective of this research was to optimize the asphalt mixture design in order to increase in-place asphalt pavement durability without sacrificing the permanent deformation characteristics of the mixture.

FINDINGS

- It is possible to design asphalt mixtures with 5 percent air voids without lowering the effective binder content; this can be accomplished by varying the aggregate gradation of the mixture.
- Asphalt mixtures designed using lower compaction levels than currently specified by the Superpave method can be designed to have |E*| and FN values as high as, or higher than, conventionally designed mixtures, without lowering the effective binder content
- On average, the mixture data appeared to indicate that 53, 52, and 42 were the optimum numbers of gyrations for the three re-designed mixtures developed in the laboratory.
- Asphalt mixtures designed in the laboratory at 5 percent air voids can be compacted to 5 percent air voids in the field. The field test indicates this can be done without additional compaction effort.
- Results of testing re-heated, plant-produced mixtures are somewhat variable, as indicated in the post-trial test results. Some test results seem to indicate the standard mixtures may perform better, while other results favor the re-designed mixtures. The performance of the test sections will help confirm whether the re-designed mixtures are or are not better performers than the standard mixtures.

IMPLEMENTATION

While the findings of this study are encouraging, the amount of testing was limited, making it difficult to recommend the exact manner in which INDOT should proceed. The following recommendations are offered as a starting point:

- For medium to high traffic levels, 50 appears to be the correct number of gyrations to be used in designing asphalt mixtures with optimum binder content chosen at 5 percent air voids.
- For lower volume roads, 30 gyration mixture designs might be more appropriate. Further laboratory and field testing should be conducted to verify this number.
- Low-temperature and moisture susceptibility testing should be completed for both standard and re-designed mixtures to help quantify the enhanced durability of the re-designed mixtures.
- Additional laboratory work should be done so as to include the effects of more traffic levels; mixtures containing RAP, RAS, or both; and additional binder grades and aggregate types.
- The performance of the trial projects placed as part of this study should be monitored for performance.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

Asphalt pavements in Indiana currently have nominal service lives of 15–20 years. Generally, these pavements reach the end of their service lives based on durability issues associated with asphalt binder aging. As the asphalt binder ages, stiffness and embrittlement increase. In terms of surface condition, the results are typically displayed as cracking, which is usually non-load associated (thermal or reflective) but may also include load associated (fatigue) cracking. Asphalt binder aging is predominantly related to oxidation, which is controlled by access of air into the asphalt pavement. Reducing the air permeability of compacted asphalt mixtures decreases the rate of asphalt binder aging, and thus allows for a longer pavement life.

The current Indiana Department of Transportation (INDOT) method of design and construction (quality assurance) of asphalt pavements targets a 4 percent air voids (V_a) content in the laboratory compacted specimens. On the road, the density acceptance criterion for INDOT Section 401 mixtures is based on 90 percent of field core densities being within statistical limits with a lower threshold limit of 91 percent of the mixture maximum theoretical specific gravity (G_{mm}). The average density needed to comply with this specification is approximately 93 percent (7 percent V_a). The result is that when the in-place density specification is met, 10 percent of the pavement area may have a density of less than 91 percent, meaning a V_a content of more than 9 percent. Test data has shown that when in-place densities begin to rise above 8 percent, air permeability begins to increase dramatically (Cooley, Prowell, & Brown, 2002).

Increasing average in-place asphalt mixture densities to 95 percent (V_a of 5 percent) would significantly decrease asphalt binder aging. Conservatively, the estimated pavement life increase would be two to three years, representing a 12 to 20 percent increase in pavement life. The key to achieving this in-place density is to optimize the relationship between laboratory and field compaction.

2. BACKGROUND

Since its adoption in the 1940s, the Marshall mixture design method has been widely used throughout world. In the 1960s and 70s, the Laboratoire Central des Ponts et Chaussées (LCPC) developed a mixture design method still used in France. Today, most agencies in the United States make use of the Superpave mixture design method to design asphalt mixtures. This mixture design method contains similarities to both the Marshall and LCPC methods. An understanding of the Marshall and LCPC asphalt mixture design methods is helpful in determining how the Superpave design method might be altered to optimize the relationship between laboratory and field compaction.

2.1 Marshall Mixture Design Method

The Marshall mixture design philosophy involves selecting an aggregate gradation and a compaction level. Aggregate is mixed with varying percentages of asphalt binder and then compacted using a drop hammer. The air voids in the compacted samples are then determined and compared to the specification values. Normally, four percent air voids is desired with a voids in the mineral aggregate (VMA) requirement that is based on the nominal maximum size of the aggregate blend. If the specified air voids cannot be achieved by merely varying the asphalt content, a new aggregate gradation, or even new aggregate materials, must be examined. A design is selected when the gradation is within a specified range and the volumetrics meet specified criteria (Asphalt Institute, 1989).

The philosophy of Marshall design compaction is that the density of specimens in the mold should match the ultimate density that will be obtained after the mixture has been in service for some period of time. Typically air voids in the Marshall mold were designed to be from 3 to 5 percent (Griffith, 1949; McFadden & Ricketts, 1948). The mixture was to be compacted to 8 to 10 percent air voids on the road. After being trafficked–some believe for two to four years, others believe at the end of service life (15 years)–the density would increase, leaving only 3 to 5 percent air voids in the pavement.

Early developmental research for the method sought to correlate the density of field test sections to the number of blows of a Marshall hammer. As a result, the design compaction level is related to the expected amount of traffic. Three levels of compaction were recommended, namely 35, 50 and 75 blows on each side of a specimen to be used for light, medium and heavy traffic, respectively.

2.2 LCPC Mixture Design Method

In the 1960s and 70s, LCPC developed a new method of mixture design. The design method is based on the principle that an asphalt mixture should be designed and, during construction, compacted to its ultimate density. Therefore, no post-construction densification is anticipated. The density at the end of the mixture's in-service life is the same as the density immediately after construction.

Developmental research for the LCPC method sought to establish a design compactive effort to match the compaction that occurs during construction. LCPC defined a standard rolling train as 16 passes of a pneumatic-tired roller weighing 3 tonnes (6,600 lbs) and having a tire pressure of 600 kPa (87 psi).

A full-scale laboratory compaction bench was built and a range of different mixtures was compacted on it. The compaction bench was 2.1 m (7 ft) wide and 4.2 m (14 ft) long with one pneumatic tire. It was discovered that for a specific mixture the density achieved under a standard rolling train was related to the lift thickness.

When lift thickness was too thin, density was lower (Moutier, 1977).

As a result, a standard lift thickness was established based on maximum aggregate size. The required lift thickness is five times the maximum aggregate size. This is equivalent to six times the nominal maximum aggregate size as defined by the Superpave mixture design method. For a 0/14 mixture, the maximum aggregate size is 14 mm (0.55 in.) and the lift thickness is 80 mm (3 in.), about six times the nominal maximum aggregate size of 12.5 mm (0.5 in.). The method results in the use of thick lifts in France. In the United States lifts tend to be thinner; about four times the nominal maximum aggregate size.

For laboratory compaction, LCPC chose a gyratory compactor. They had some experience with the Marshall hammer and with static compaction. (The Duriez method of mixture design, used before the LCPC method, used static compaction.) In 1959, a French delegation visited Texas and witnessed the Texas gyratory compactor. Studies were done, and by 1968 a gyratory compaction protocol was developed in France.

The Texas compactor used an angle of nearly six degrees and applied gyrations at the rate of one per second in bursts of three gyrations. After each set of gyrations the specimen was checked for resistance to further compaction. If the specimen was not yet resistant enough, an additional set of gyrations was applied. Compaction was considered complete when the specimen was sufficiently resistant to compaction (Ortolani & Sandberg, 1952).

LCPC designed and constructed a gyratory compactor that applied gyrations continuously instead of in bursts. Studies were done to investigate the effect of gyratory angle and vertical pressure for different types of mixtures. Studies were also done to select specimen size. As a result of these studies LCPC standardized the gyratory angle at 1 degree with a vertical pressure of 600 kPa (87 psi). LCPC also monitored the rate of increase in density and discovered a nearly linear relationship between density and the log of the number of gyrations (Moutier, 1974).

After correlating compaction curves in the gyratory to densification curves from the laboratory compaction bench, LCPC set the design number of gyrations to equal the lift thickness in millimeters. Therefore, a 14 mm maximum size mixture (12.5-mm nominal maximum size) would be constructed 80 mm (3 in.) thick (five times maximum size) and the design compaction would be 80 gyrations.

In the LCPC method, the design asphalt binder content is fixed for each mixture type; there are adjustment factors for asphalt absorption, aggregate specific gravity, and gradation or surface area. Since the effective asphalt binder volume is fixed, the design method becomes one of selecting an aggregate structure to provide the correct range of air voids at the design compaction. The range of allowable air voids is 4 to 8 percent. Most designers target 5 percent air voids. In the field, the required density is 95 percent of maximum theoretical gravity. LCPC has documented that there is little or no increase in density under traffic during the pavement service life.

2.3 Superpave Mixture Design Method

The Superpave mixture design method and procedures were developed as part of the Strategic Highway Research Program (SHRP) to be a comprehensive system for the design and modeling of asphalt materials. Asphalt binder testing was implemented to relate the performance of the binder to the climate and traffic level. Aggregate quality specifications were established in an effort to improve the performance of the resulting mixtures. The Superpave gyratory compactor (SGC) was developed as a laboratory tool that more closely simulates field compaction of asphalt mixtures.

During SHRP, developmental studies focused on relating gyratory compactive effort to the density of pavements at the end of their service lives. The underlying principle in SHRP was carried over from the Marshall method. The design air voids were to be selected at a low level (air voids were 4 percent instead of the range of 3 to 5 percent) and construction air voids were expected to be around 8 percent. The N-design experiment during SHRP determined the number of gyrations required to match in-place density for pavements that were at least 12 years old and had been subjected to different levels of traffic as measured by Equivalent Single Axle Loads (ESAL; Huber, Blankenship, & Mahboub, 1994).

The decision to fix design air voids at 4 percent, instead of using the range of 3 to 5 percent came about because of an issue dealing with VMA. In the late 1980s the Asphalt Institute was evaluating issues dealing with VMA. The reason for VMA, since its inclusion in Marshall design in 1962, was to ensure that a mixture had a minimum volume of effective asphalt binder and a minimum volume of air voids. Minimum VMA was fixed according to the nominal maximum aggregate size and was constant regardless of the design air voids. As a result the minimum effective volume of asphalt binder could vary. For example, a 12.5-mm nominal maximum size mixture required 14 percent VMA. The volume of effective asphalt binder could range from 11 to 9 percent depending on whether the design air voids were chosen at 3 or 5 percent. As a result, in 1989 the Asphalt Institute changed the design criteria in MS-2, the Manual *Mix Design Methods for Asphalt Concrete*, from a range of 3 to 5 percent to a single value of 4 percent to ensure a sufficient volume of binder.

During SHRP, consideration was given to discontinuing the use of VMA and instead specifying a minimum effective asphalt volume, for example, specifying a minimum effective asphalt volume of 10 percent for a 12.5-mm nominal maximum mixture. Then, the air void content could be specified as a range of 3 to 5 percent. Instead, the decision was made to keep VMA as a design criterion. To prevent the problem of effective asphalt volume (V_{be}) varying depending on design air voids, the decision was made to adopt the Asphalt Institute recommendation of a constant 4 percent design air void criteria.

3. PROBLEM STATEMENT AND OBJECTIVE

The INDOT currently uses the Superpave asphalt mixture design method that specifies a design V_a content of 4 percent. This can result in lower than desired asphalt pavement densities leading to decreases in pavement service lives due to durability loss as the asphalt prematurely ages. Increasing average asphalt pavement densities to 95 percent (V_a of 5 percent) would significantly decrease pavement aging. Conservatively, the estimated pavement life increase would be two to three years, representing a 12 to 20 percent increase in pavement life. The key to achieving this increased density is to optimize the relationship between laboratory and field compaction.

The objective of the research is therefore to optimize asphalt mixture laboratory design compaction as it relates to field compaction in order to increase asphalt pavement durability without sacrificing the permanent deformation characteristics of the mixtures. The mixtures designed according to the optimized design method are expected to be able to be compacted in the field to the laboratory design level and to withstand additional densification under traffic.

4. FINDINGS AND DELIVERABLES

This chapter describes the approach taken in this research project to address the project objectives. The results and findings are summarized, conclusions drawn and recommendations for future work and possible implementation are outlined.

4.1 Research Approach

In order to complete the study objective to optimize asphalt mixture laboratory design compaction as it relates to field compaction without sacrificing mixture permanent deformation characteristics, three appropriate asphalt mixtures that had been used on INDOT projects were chosen for testing. This involved considering various combinations of factors such as pavement categories, aggregate types, mixture types (gradations), and asphalt binder types. These three existing mixture designs were used as the basis for the mixtures evaluated in this study.

Each of the three existing mixtures used reclaimed asphalt pavement (RAP) and/or recycled asphalt shingles (RAS), which were difficult to control tightly in the laboratory. Therefore, a standard mixture design meeting all applicable INDOT specifications was completed using the current INDOT mixture design method with the same materials and design gyrations as the existing mixtures but excluding RAP and RAS. The three resulting asphalt mixture designs are referred to in this report as the “standard” mixture designs.

For each standard mixture design, three additional mixture designs were completed using lower N_{design} levels (with one exception as explained later). The optimum binder content for each of these “re-designed” mixtures was selected at 5 percent air voids; the effective binder volume (V_{be}) and VMA were the same as the corresponding standard design. It was important to keep the V_{be} as constant as possible for each set of mixtures (standard and re-designed) because it is the effective binder that promotes asphalt mixture durability. In order to increase the design air voids by one percent, but keep the V_{be} constant, the aggregate proportions were varied to meet the design criteria.

With the job-mix formula (JMF) for standard and re-designed mixtures established, test specimens were prepared for dynamic modulus and flow number testing according to AASHTO T 342-11 (2015) and AASHTO TP 79-10 (2010), respectively. Specimens for the standard designs were produced at 7 percent air voids according to current test method standards. Specimens for the re-designed mixtures were produced at 5 percent air voids; this is the anticipated in-service air voids level for the re-designed mixtures.

The dynamic modulus (|E*|) and the flow number (FN) were determined for the standard and re-designed mixtures. The |E*| is commonly referred to as the asphalt mixture stiffness, and FN is believed to denote the onset of tertiary (plastic) flow in an asphalt mixture. A higher value of |E*| indicates a stiffer mixture. The FN is believed to be a reliable indicator of in-service rutting; the higher the FN, the more loading an asphalt mixture can tolerate without rutting. The resulting data were analyzed to determine how the dynamic moduli and flow number values for the re-designed mixtures compared to those of the standard mixtures.

In order to keep the experiment to a manageable level, the three asphalt mixtures were chosen to represent two traffic categories (Categories 3 and 4, as shown in Table 4.1), both requiring an N_{design} of 100 gyrations. These two traffic levels account for approximately 50 percent of the asphalt mixture designs used in Indiana. The mixtures are of two sizes, 9.5 and 19.0 mm.

TABLE 4.1
Asphalt pavement categories.

Category	Equivalent Single Axle Loads	N_{design}
1	< 300,000	50
2	300,000 to < 3,000,000	75
3	3,000,000 to < 10,000,000	100
4	10,000,000 to < 30,000,000	100
5	≥ 30,000,000	125

Following conventional practice, 100 gyrations of the SGC were used when designing the standard mixtures. The re-designed mixtures were designed using 70, 50 and 30 gyrations of the SGC. Again, optimum binder content for these mixtures was selected at 5 percent air voids, not 4 percent as the standard mixtures. The experimental design is shown in Table 4.2.

In consultation with the Study Advisory Committee (SAC) the following additional experimental factors and factor levels were chosen:

- One asphalt binder grade, PG 64-22.
- One aggregate gradation type, coarse-graded, defined as a mixture with a gradation that passes below the primary control sieve (PCS) control point.
- Three coarse aggregate types, limestone, dolomite, and air-cooled blast furnace (ACBF) slag.

Fine aggregates used in the mixtures include limestone, dolomite, and natural sands. Baghouse fines from an asphalt plant were also incorporated to ensure adequate levels of fines (minus 0.075-mm material) and proper dust ratios. The gradations and bulk specific gravities (G_{sb}) of the aggregates are shown in Table 4.3.

TABLE 4.2
Experimental design.

Pavement Category	Number of Gyrations	Mixture Type: 9.5-mm	Mixture Type: 19.0-mm
3	30	X	
	50	X	
	70	X	
4	30	X	X
	50	X	X
	70	X	X

4.2 Laboratory Mixture Designs

The Category 4, 19.0-mm asphalt mixtures consisted of limestone coarse aggregate, limestone and natural sand, and baghouse fines as filler. Several trial gradations were tested before the final mixture gradations were determined. The gradations for the standard and re-designed mixtures are given in Table 4.4 and plotted in Figure 4.1. The volumetric properties of the mixtures at optimum binder contents are shown in Table 4.5.

The Category 3, 9.5-mm asphalt mixtures were produced from limestone coarse aggregate, limestone sand, and baghouse fines as filler. The gradations of the four mixtures are shown in Table 4.4 and plotted in Figure 4.2; the volumetric properties are shown in Table 4.5.

Finally, the Category 4, 9.5-mm mixtures were made with dolomite and slag coarse aggregates, dolomite and natural sands, and baghouse fines. The gradations of the mixtures are shown in Table 4.4 and plotted in Figure 4.3. The volumetric properties are given in Table 4.5. Only three mixture designs were completed for this mixture, the standard design with an N_{design} of 100, and re-designed mixtures compacted with N_{design} values of 50 and 30. Mixture design data from both the Category 4, 19.0-mm and Category 3, 9.5-mm mixtures indicated little difference between the 100-gyration and 70-gyration mixtures when the optimum binder content was chosen at 4 and 5 percent air voids respectively. Lowering the number of design gyrations by 30 effectively raised the mixture air voids by one percent. Thus the gradation was either not changed or changed only slightly from the original; therefore the 70-gyration mix was eliminated for the third mixture.

4.3 Results and Discussion

The |E*| test (AASHTO T 342) was performed and master curves developed according to the second-order

TABLE 4.3
Aggregate gradations (percent passing) and specific gravities.

Materials Sieve Size, mm	Limestone #8	Limestone #11	Limestone #12	Limestone #24 Sand	Blast Furnace Slag #11	Dolomite #11	Dolomite #12	Dolomite #24 Sand	Natural Sand	Bag House Fines
25.0	100.0									
19.0	90.3									
12.5	49.7	100.0	100.0		100.0	100.0	100.0			
9.5	25.8	83.3	99.6	100.0	84.4	89.1	99.9	100.0	100.0	
6.3	13.0	44.2	63.8	99.6	41.5	43.3	79.9	99.7	98.6	
4.75	6.8	25.2	46.4	99.4	21.7	22.1	70.7	99.6	97.9	
2.36	2.6	4.7	11.1	90.6	6.0	6.8	20.8	94.9	90.8	
1.18	2.0	2.2	4.1	56.0	4.4	2.9	6.3	63.4	75.3	
0.600	1.7	1.9	2.5	33.4	3.7	1.8	2.6	36.7	57.3	100.0
0.300	1.4	1.7	2.1	16.1	3.2	1.4	1.6	20.1	23.4	99.4
0.150	1.2	1.5	1.9	7.8	2.6	1.2	1.2	8.9	2.4	97.8
0.075	1.0	1.4	1.7	5.0	2.0	0.9	1.0	4.4	1.1	94.6
G_{sb}	2.664	2.664	2.655	2.671	2.442	2.681	2.688	2.748	2.612	2.700

TABLE 4.4
Asphalt mixture gradations and combined aggregate specific gravities.

Mixture Sieve Size, mm	Category 4, 19.0-mm				Category 3, 9.5-mm				Category 4, 9.5-mm		
	N100	N70	N50	N30	N100	N70	N50	N30	N100	N50	N30
25.0	100.0	100.0	100.0	100.0							
19.0	97.4	97.4	96.1	95.3							
12.5	86.4	86.4	79.9	75.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0
9.5	77.0	77.4	68.4	62.1	97.4	97.3	95.6	95.4	95.3	94.7	94.1
6.3	59.6	60.7	55.1	51.8	75.8	76.0	71.6	72.6	72.8	72.2	71.4
4.75	51.2	52.7	48.6	46.8	65.4	65.7	60.0	61.6	61.9	61.3	60.5
2.36	36.5	38.2	37.8	39.3	33.0	34.7	38.1	43.7	34.0	38.2	42.4
1.18	22.8	23.9	23.9	25.5	19.1	20.4	24.6	29.5	20.4	24.9	29.2
0.600	14.6	15.3	15.3	16.6	12.4	12.9	16.1	19.4	12.8	16.4	19.7
0.300	8.5	8.8	8.8	9.7	7.8	7.7	9.5	11.3	7.6	9.9	11.5
0.150	5.6	5.7	5.6	6.4	4.9	4.4	5.3	6.0	4.2	5.7	6.1
0.075	4.4	4.5	4.4	5.1	4.0	3.4	4.0	4.5	3.0	4.3	4.6
G_{sb}	2.665	2.665	2.650	2.651	2.692	2.692	2.692	2.694	2.631	2.630	2.626

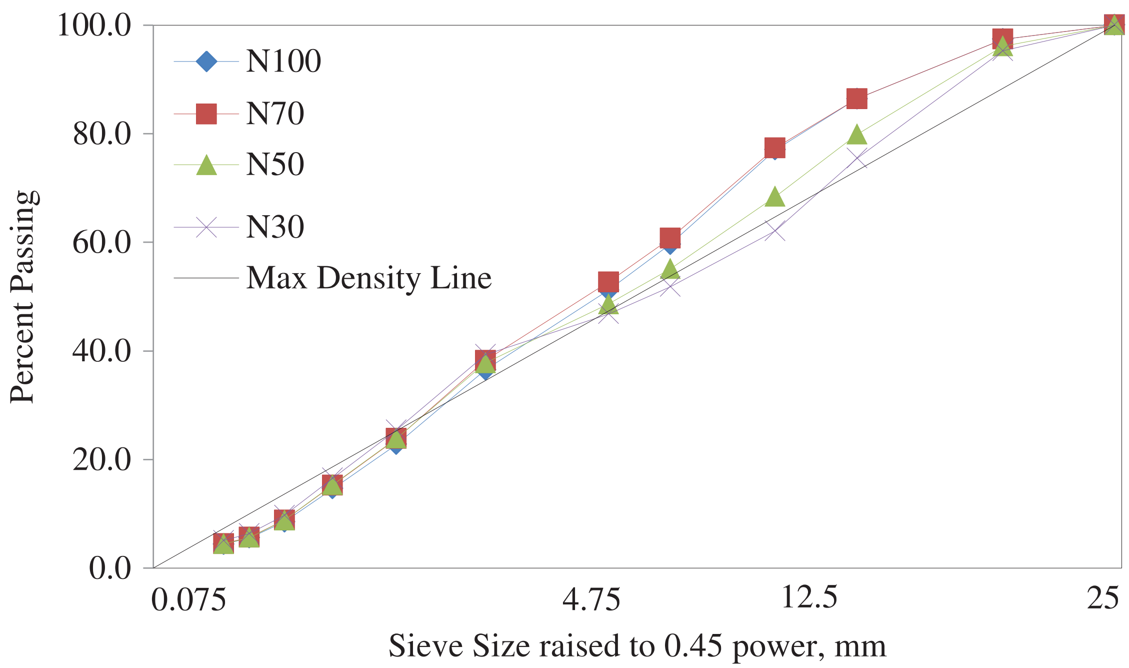

Figure 4.1 Category 4, 19.0-mm mixture gradations.

polynomial fit found in AASHTO PP 62 (2009). The test was performed at six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz) and four temperatures (4, 21, 37, and 50°C). The reference temperature was 30°C.

The FN test was completed using the AMPT (unconfined), according to AASHTO TP 79 with a deviator stress (σ_d) of 600 kPa and contact stress of 30 kPa (5 percent of σ_d) at a temperature of 50.5°C. This temperature was selected based on the LTTPBind database (Bonaquist, 2008). The Francken Model was used for curve fitting.

Once the mixture designs were complete, four test specimens of each mixture were produced to a specific air void content corresponding to the expected field compaction level. Samples of the standard mixtures (N100) were compacted to 7 percent air voids (after coring and sawing) to simulate field compaction at the start of the pavement life. Test specimens for the re-designed mixtures were compacted, cored and sawed to produce 5 percent air voids in the test specimens, again to simulate anticipated field compaction at the start of the pavement life. Once the test specimens were fabricated and air void contents checked, AASHTO T 342 and TP 79 tests were performed. Table 4.6 shows the air void contents for the |E*| specimens prior to testing. The cells in Table 4.6 with no data are where no samples could be tested due to lack of material.

TABLE 4.5
Mixture design volumetrics.

N_{design}	Avg. V_a, %	Avg. G_{sb}	Avg. VMA, %	Avg. VFA, %	P_{be}, %	Avg. Dust Ratio
Category 4, 19.0-mm Mixture						
100	4.0	2.665	13.6	70.6	4.1	1.1
70	4.9	2.665	14.5	66.3	4.1	1.1
50	4.9	2.650	14.4	66.0	4.1	1.1
30	4.9	2.651	14.9	67.2	4.3	1.2
Category 3, 9.5-mm Mixture						
N_{design}	Avg. V_a, %	Avg. G_{sb}	Avg. VMA, %	Avg. VFA, %	P_{be}, %	Avg. Dust Ratio
100	4.1	2.692	15.0	72.9	4.6	0.9
70	5.1	2.692	16.0	67.9	4.6	0.7
50	4.9	2.692	15.8	68.9	4.6	0.9
30	5.3	2.694	16.3	67.6	4.7	0.9
Category 4, 9.5-mm Mixture						
N_{design}	Avg. V_a, %	Avg. G_{sb}	Avg. VMA, %	Avg. VFA, %	P_{be}, %	Avg. Dust Ratio
100	3.8	2.631	15.0	74.9	4.8	0.6
50	4.9	2.630	16.4	70.0	5.0	0.9
30	5.0	2.626	16.4	69.6	5.0	0.9

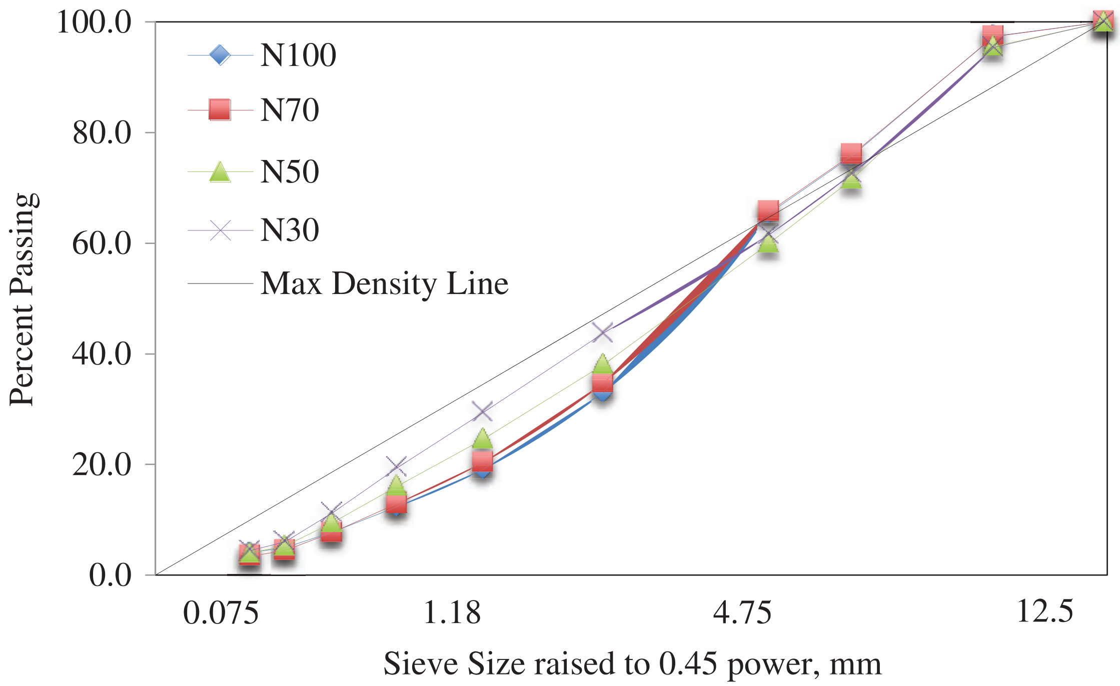

Figure 4.2 Category 3, 9.5-mm mixture gradations.

4.3.1 Category 4, 19.0-mm Mixture

The Category 4, 19.0-mm mixture |E*| master curves for all four gyration levels are plotted on a log-log graph in Figure 4.4. For better clarity at the high temperature frequencies, the data is shown on a semi-log plot in Figure 4.5. The results show that the 70-gyration (N70) mixture is the stiffest of the four mixtures, the N100 mixture the least stiff. The 30- and 50-gyration (N30, N50) mixtures are between the two and appear to be about the same. The fact that the N30, N50, and N70 mixtures all have a higher |E*| values than the N100 mixture is not surprising considering the N100 specimens were compacted to 7 percent air voids and the N30, N50, and N70 specimens to 5 percent air voids. In general, the denser

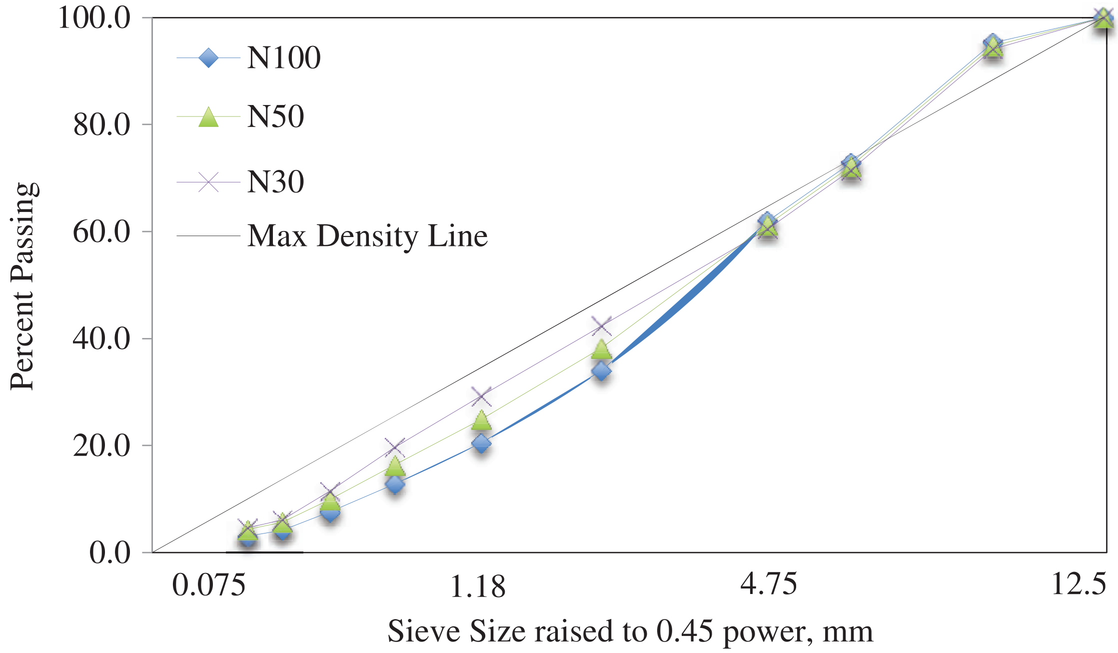

Figure 4.3 Category 4, 9.5-mm mixture gradations.

TABLE 4.6
Dynamic modulus specimen air voids data.

Category 4, 19.0-mm Mixture						
N_{design}	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 4**	**Average**	**SD**
100	7.1	7.5	7.1	–[1]	7.2	0.23
70	5.5	5.3	4.7	–[1]	5.2	0.42
50	4.7	4.9	5.2	4.7	4.9	0.24
30	4.6	4.7	5.2	5.4	5.0	0.39
Category 3, 9.5-mm Mixture						
N_{design}	Sample 1	Sample 2	Sample 3	Sample 4	Average	SD
100	7.7	7.2	6.5	6.7	7.0	0.54
70	4.8	4.7	5.0	5.4	5.0	0.31
50	4.7	4.6	4.8	4.9	4.8	0.13
30	5.1	4.9	5.1	4.9	5.0	0.12
Category 4, 9.5-mm Mixture						
N_{design}	Sample 1	Sample 2	Sample 3	Sample 4	Average	SD
100	6.9	6.9	7.1	7.0	7.0	0.10
50	4.8	5.1	4.8	4.8	4.9	0.15
30	5.1	5.1	4.9	4.9	5.0	0.12

[1]No specimens were available for testing.

a mixture, the stiffer it will be, as long as it is not so dense that it becomes susceptible to plastic flow. Regardless of the specimen air voids, it is important to note that all three of the re-designed mixtures have |E*| values higher than the standard design. In theory this implies the re-designed mixtures should have better rutting performance than the standard mixture when compacted to the anticipated field density.

Table 4.7 shows the FN results for Category 4, 19.0-mm mixture designs. Due to a lack of materials, it was not possible to make a separate set of specimens for FN testing. Instead, the |E*| specimens were used for FN testing. This resulted in the low air voids as shown in Table 4.7; the specimens were consolidated in the |E*| tests. While reusing the |E*| specimens for FN testing is not strictly correct according to standard test methods,

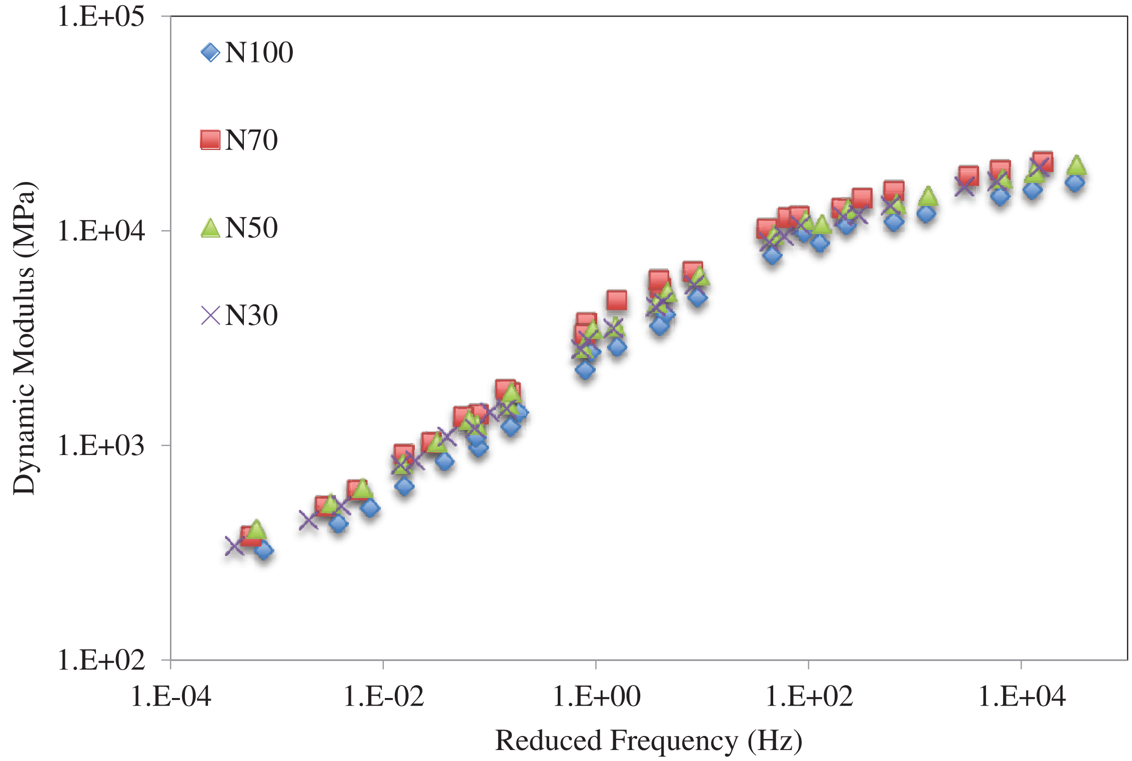

Figure 4.4 Category 4, 19.0-mm master curves (log-log).

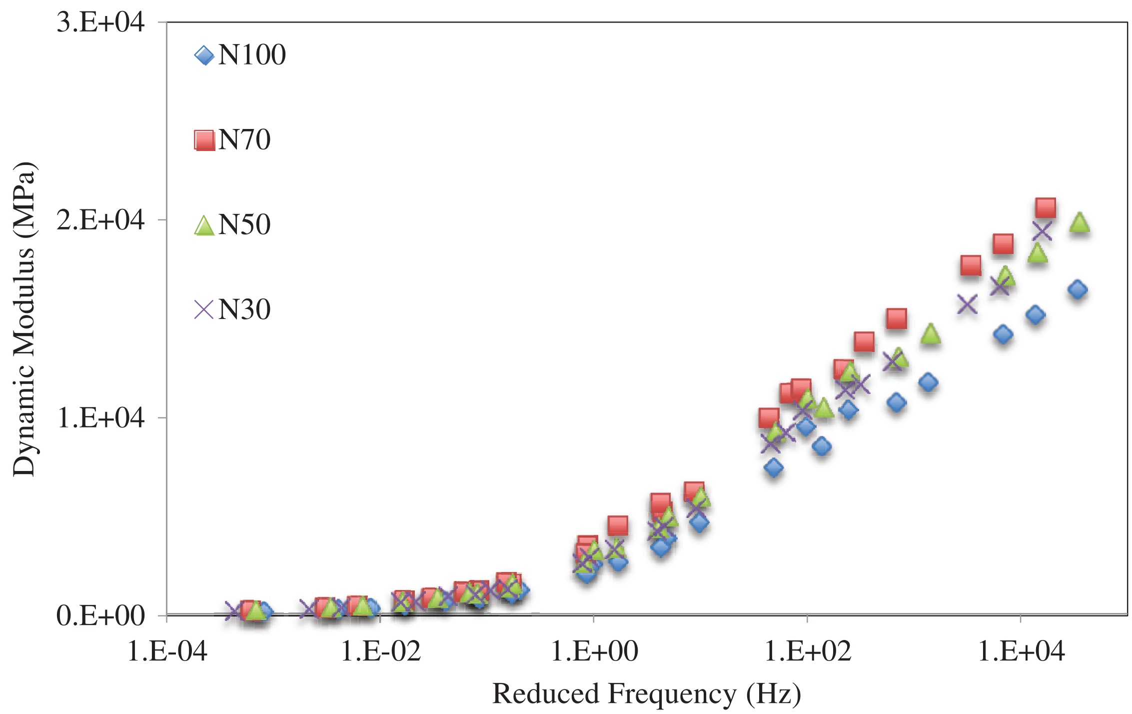

Figure 4.5 Category 4, 19.0-mm master curves (semi-log).

it does allow for a relative comparison. Some of the |E*| specimens were not reusable and thus the lack of data in some data cells.

The FN results indicate that all three of the re-designed mixtures have higher FN values than does the standard mixture. The N50 and N70 mixtures have FN values over twice as large as the standard design. Also, the strain at FN is lower for the re-designed mixtures than for the standard mixture. Both these results suggest the re-designed mixtures should be able to withstand higher levels of traffic than the standard design before the onset of rutting when compacted to the anticipated field density.

The |E*| and FN data can be plotted as a function of the number of gyrations used to compact the mixture design specimens. An example of this is shown in Figure 4.6.

TABLE 4.7
Category 4, 19.0-mm mixture flow number data.

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Average	SD
	N100						
Air Voids, %	6.7	7.0	6.7	–[1]	–[1]	6.8	0.17
Flow Number	191	132	164	–[1]	–[1]	162	30
Strain @ FN, με	21,881	27,129	22,938	–[1]	–[1]	23,983	2,776
	N70						
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Sample 5**	**Average**	**SD**
Air Voids, %	5.1	4.9	4.1	–[1]	–[1]	4.7	0.53
Flow Number	371	360	427	–[1]	–[1]	386	36
Strain @ FN, με	20,230	18,245	16,331	–[1]	–[1]	18,269	1,950
	N50						
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Sample 5**	**Average**	**SD**
Air Voids, %	4.3	4.2	4.0	4.0	–[1]	4.1	0.15
Flow Number	220	337	424	410	–[1]	348	93
Strain @ FN, με	20,179	20,722	18,973	19,654	–[1]	19,882	747
	N30						
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Sample 5**	**Average**	**SD**
Air Voids, %	4.0	4.1	4.8	5.3	5.0	4.6	0.60
Flow Number	210	221	162	140	194	185	34
Strain @ FN, με	23,862	23,171	20,811	21,542	21,066	22,090	1350

[1]No specimens were available for testing.

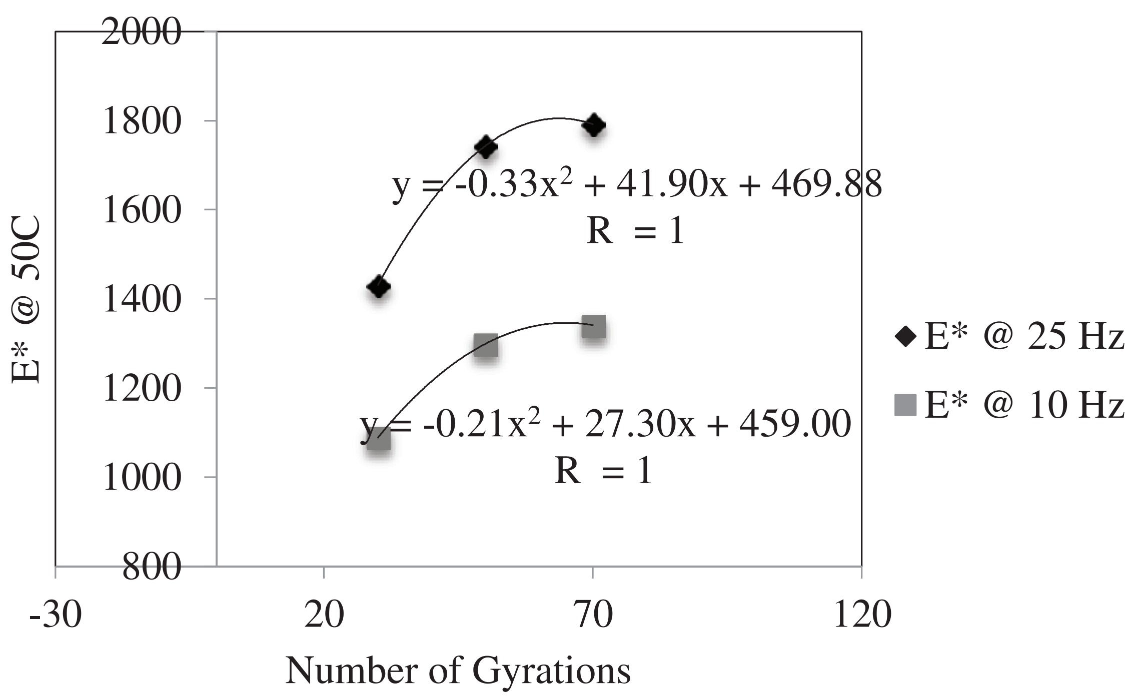

Figure 4.6 Dynamic modulus as a function of gyrations.

The figure shows the |E*| at 50°C for 10 and 25 Hz. A polynomial trend line has been used to fit both the 10 and 25 Hz data. The equations can each be solved to determine the number of gyrations that results in the peak |E*| values. The peak |E*| value for the 10 Hz data occurs at 65 gyrations; the peak |E*| for the 25 Hz data at 63 gyrations. When this technique is used for the |E*| data at the 6 and 50°C data, and the FN, the overall average optimum number of gyrations is 53.

4.3.2 Category 3, 9.5-mm Mixture

For the Category 3, 9.5-mm mixture, specimens were prepared and tested in a similar fashion to the 19.0-mm mixture. In addition to the specimens for the standard and three re-designed mixtures, an additional set of N100 test specimens was produced. This second set of N100 mixture test specimens was compacted to 5 percent air voids, as were the re-designed mixture specimens. This was done in order to compare the |E*| and FN data for standard mixture to the re-designed mixtures when all of the test specimens have similar air voids contents. The |E*| master curves for the five groups of specimens are shown in Figure 4.7. For better clarity of the high frequency data, the semi-log plot is shown in Figure 4.8.

The |E*| results indicate that all the mixtures have approximately same stiffness, although at higher frequency, the N30 mixture shows a slightly higher stiffness compared to the other mixtures. In general, the standard N100 mixture specimens compacted to 5 percent air voids have |E*| values approximately equivalent to the re-designed mixtures and |E*| values higher than the standard N100 mixture specimens compacted to 7 percent air voids. This would seem to validate the theory that, for a given mixture, the more densely compacted the mixture, the higher the stiffness. Thus, producing asphalt mixtures with in-place air voids of 5 percent should yield better rutting performance than compacting mixtures to in-place air voids of 7–8 percent as is done currently. However, overcompacting mixtures will cause the air voids to become low enough to make them susceptible to plastic flow, thus causing them to rut under traffic loads.

Table 4.8 shows the FN results for the Category 3, 9.5-mm mixture. Rather than re-using the |E*| for FN testing of these mixtures, new specimens were made for FN testing. However, due to limitations of materials quantities, it was not possible to produce and test four specimens for each mixture. Also, the N100 standard design specimens compacted to 5 percent air voids were not tested for FN, again due to materials limitations. The data in the table indicate that all three re-designed mixture have higher FN values than does the standard mixture. The strain at FN data is somewhat conflicting with the N30 and N50 re-designed mixtures having higher strains at FN than the standard mixture; the N70 re-designed mixture has a strain at FN lower than does the standard mixture.

Overall, the |E*| and FN data for the Category 3, 9.5-mm mixture appear to indicate that, were these mixtures placed in the field, the re-designed mixtures would be expected to perform as well as the standard mixture in terms of rutting resistance.

The |E*| and FN data for the Category 3, 9.5-mm mixture was plotted as a function of the number of gyrations used to compact the mixture specimens, as

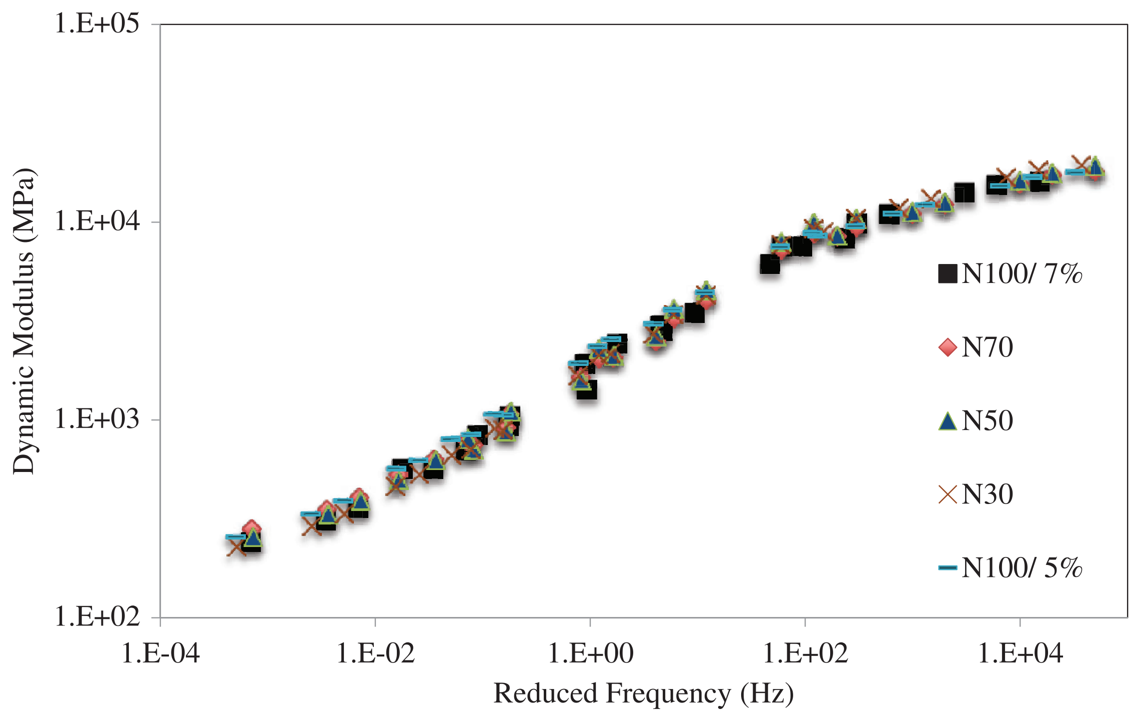

Figure 4.7 Category 3, 9.5-mm master curves (log-log).

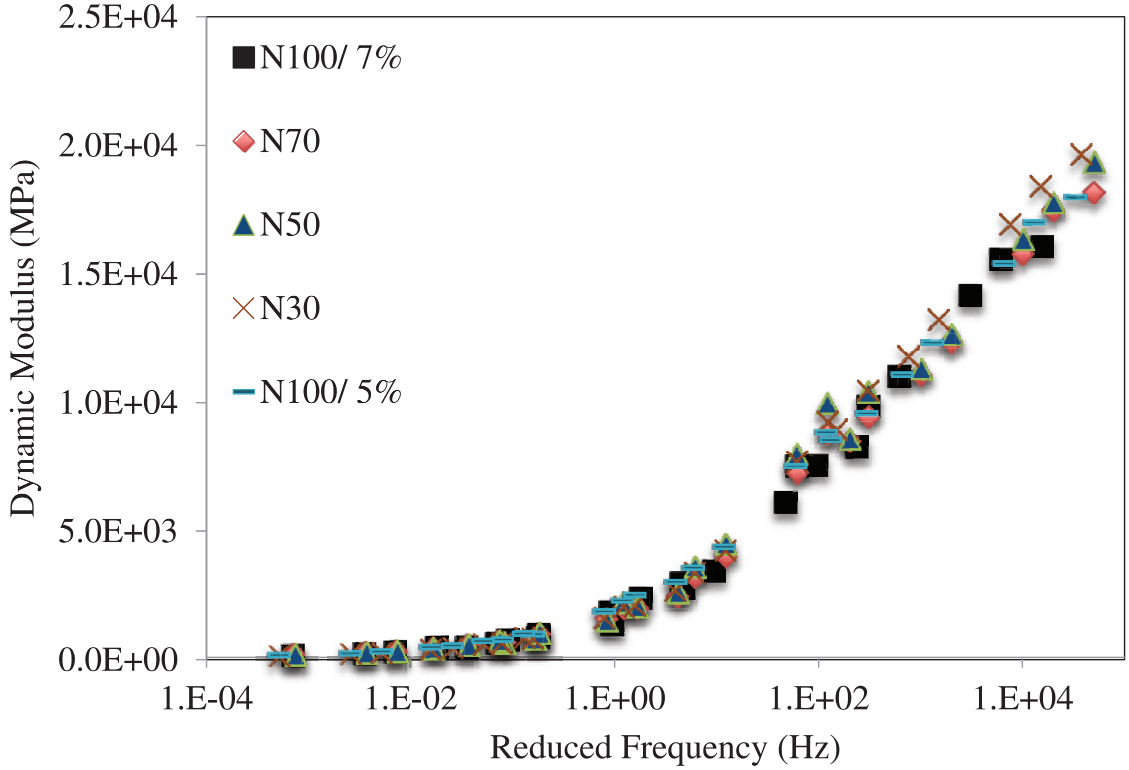

Figure 4.8 Category 3, 9.5-mm master curves (semi-log).

TABLE 4.8
Category 3, 9.5-mm mixture flow number results.

	N100					
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Average**	**SD**
Air Voids, %	8.0	7.0	7.3	7.0	7.3	0.5
Flow Number	92	101	61	109	91	21
Strain @ FN, με	18,366	18,436	16,886	18,769	18,114	837
	N70					
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Average**	**SD**
Air Voids, %	5.5	4.8	4.8	–[1]	5.0	0.40
Flow Number	161	176	165	–[1]	167	8
Strain @ FN, με	17,650	18,296	17,166	–[1]	17,704	567
	N50					
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Average**	**SD**
Air Voids, %	4.5	4.5	4.6	–[1]	4.5	0.15
Flow Number	183	156	151	–[1]	163	17
Strain @ FN, με	18,008	18,861	24,030	–[1]	20,300	3,259
	N30					
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Average**	**SD**
Air Voids, %	5.3	5.0	4.9	–[1]	5.1	0.21
Flow Number	141	162	164	–[1]	156	13
Strain @ FN, με	19,000	19,419	19,193	–[1]	19,204	210

[1]No specimens were available for testing.

was done for the Category 4, 19.0-mm mixture. However, for the Category 3, 9.5-mm mixture, the low temperature (4°C) |E*| and FN data produced no peak values. Instead, for these two sets of data, the number of gyrations producing the highest values was chosen as the optimum number of gyrations and included in the overall average. The overall average optimum number of gyrations for this mixture appears to be 53.

4.3.3 Category 4, 9.5-mm Mixture

For the Category 4, 9.5-mm mixture, specimens were prepared and tested in a similar fashion to the previous mixtures. However, since the results from the two previously tested mixtures indicated little difference between the standard N100 mixtures and the N70 re-designed mixtures, no N70 mixture was designed and tested for the Category 4, 9.5-mm mixture. Only three sets of mixture specimens were tested for this mixture - the standard N100 mixture and the N30 and N50 re-designed mixtures.

Figures 4.9 and 4.10 show the |E*| results for Category 4, 9.5mm mixtures. The |E*| data indicate that all the mixtures have approximately the same stiffness, although at higher frequency, the N50 mixture shows a slightly higher stiffness than does the N30 mixture. The N100 standard mixture has the lowest stiffness of the three mixtures.

The FN results for the Category 4, 9.5-mm mixture are shown in Table 4.9. The data show that the N30 and N50 re-designed mixtures have FN over twice as high as the standard N100 design. Additionally, both the re-designed mixtures have lower strain at FN values than does the standard design. As noted earlier, both these results suggest the re-designed mixtures should be able to withstand higher levels of traffic than the standard design before the onset of rutting when compacted to the anticipated field density.

Determining an optimum number of gyrations for the Category 4, 9.5-mm mixture is somewhat more problematic than for the prior two cases. For this mixture there are only two data points (N30, N50) for the |E*| and FN data. No curve fit can be established for only two points. Therefore, the gyration level that produced the highest |E*| or FN was chosen for use in calculating the optimum number of gyrations. Using this approach, the overall average optimum number of gyrations for this mixture appears to be 42. However, since no curve fit could be established, one might consider assigning lesser importance to the 42 gyrations than to the 53 and 52 gyrations determined for the mixtures with more data.

4.4 Statistical Analysis

Statistical summaries and analyses of the |E*| data were completed for each of the three mixtures. Within each mixture type, the Bonferroni method was used to compare the re-designed and standard mixtures to determine if the mixtures were statistically significantly different. This method allows for a direct statistical comparison of the mean values of a variable. In this case, the Bonferroni method null hypothesis is that there are no differences between the average |E*| values of the mixtures ($\mu_{N30} = \mu_{N50} = \mu_{N70} = \mu_{N100}$).

4.4.1 Category 4, 19.0-mm Mixture

The statistical summary of the |E*| data is shown in Table 4.10; the summary includes the average |E*|

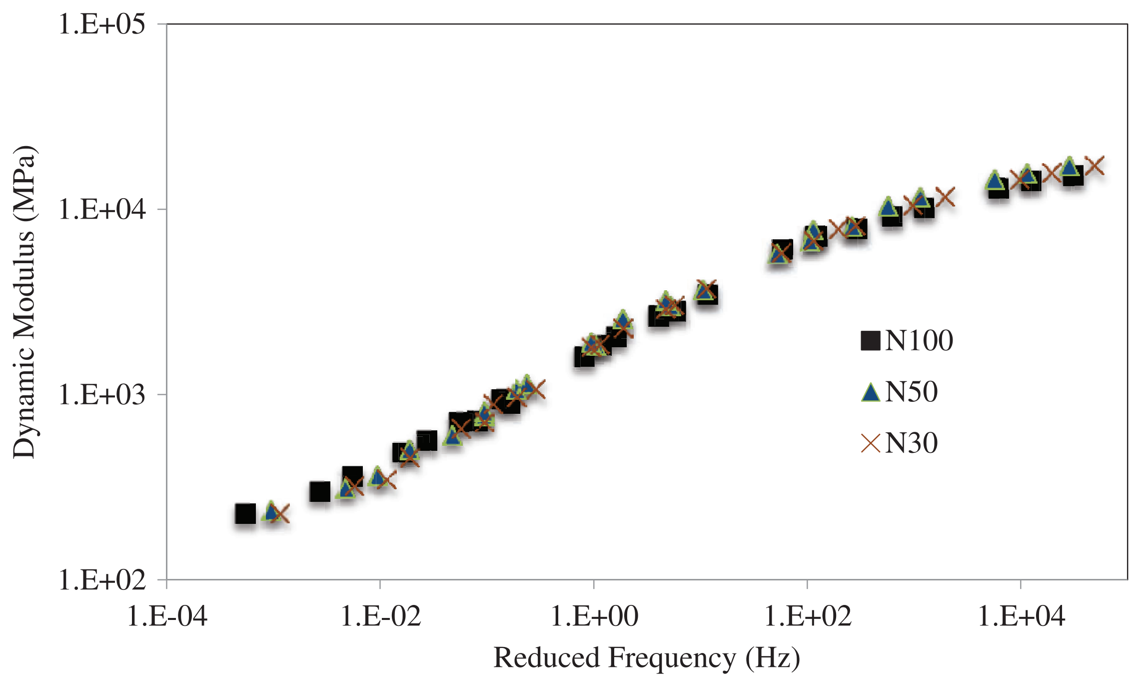

Figure 4.9 Category 4, 9.5-mm master curves (log-log).

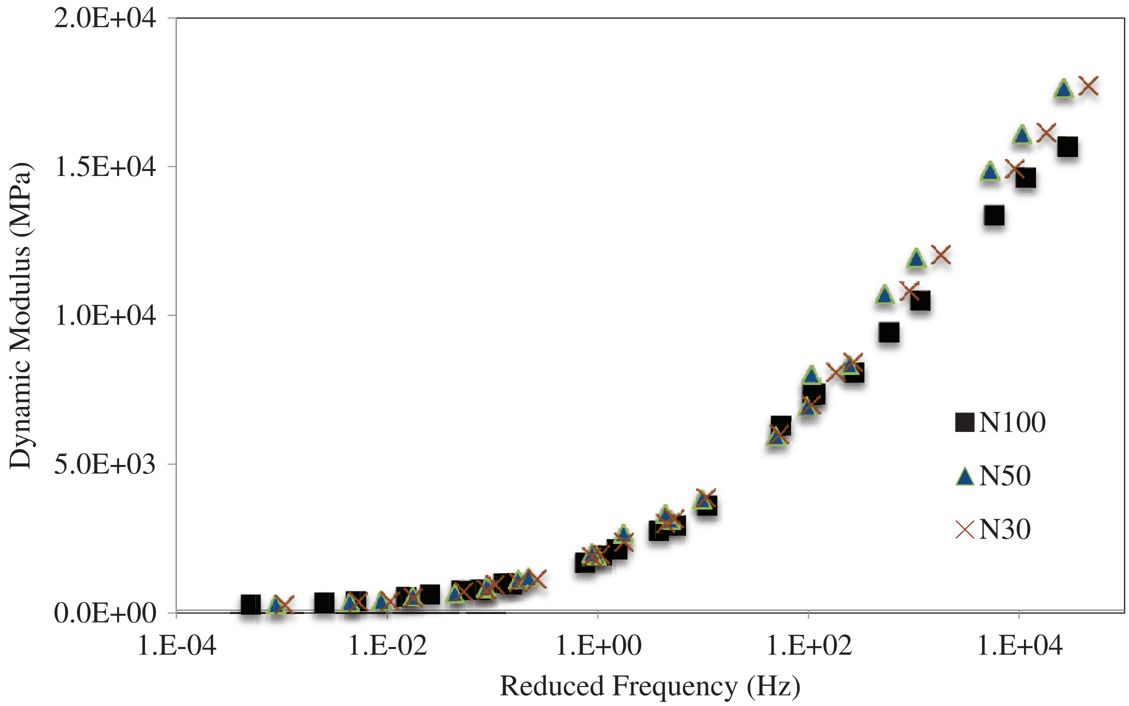

Figure 4.10 Category 4, 9.5-mm master curves (semi-log).

TABLE 4.9
Category 4, 9.5-mm mixture flow number results.

	N100						
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Sample 5**	**Average**	**SD**
Air Voids, %	6.5	6.5	6.7	6.7	–[1]	6.6	0.12
Flow Number	157	173	162	147	–[1]	160	11
Strain @ FN, με	17,269	18,657	23,497	21,448	–[1]	23,983	2,793
	N50						
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Sample 5**	**Average**	**SD**
Air Voids, %	5.2	5.5	5.3	5.4	5.4	5.3	0.11
Flow Number	226	207	303	318	212	253	53
Strain @ FN, με	20,600	19,756	22,556	21,751	20,013	20,935	1,188
	N30						
	Sample 1	**Sample 2**	**Sample 3**	**Sample 4**	**Sample 5**	**Average**	**SD**
Air Voids, %	5.3	5.3	5.1	5.3	–[1]	5.3	0.10
Flow Number	209	218	231	184	–[1]	211	20
Strain @ FN, με	19,654	22,812	19,702	21,973	–[1]	21,033	1,607

[1]No specimens were available for testing.

values at each temperature and frequency, along with the coefficient of variation (CV) for the averages. The summary statistics seem reasonable, except perhaps for the CV for the N30 value at 6°C and 25 Hz. The CV tends to increase as the test temperature increases. This is to be expected; as the specimens become less stiff, the measurement of |E*| becomes less precise. In the case of the N30 CV previously mentioned, the value appears to be higher than it should be. In reviewing the individual |E*| data values, one of the specimens has an |E*| that is much different than the others. This is the likely reason for the high CV. If this one data point were removed, the average |E*| of the N30, 6°C, 25 Hz data would be slightly over 17,000 MPa with a CV of 1.6%, a value much more consistent with the other data.

Table 4.11 contains the statistical groupings of the data using the Bonferroni method. There is some variation in the data, but overall the results indicate that statistically there are only minor differences in the |E*| values of the standard and re-designed mixtures. This suggests that the re-designed mixtures should perform at least as well in the field as the standard mixture.

4.4.2 Category 3, 9.5-mm Mixture

The statistical summary of the |E*| data for the Category 3, 9.5-mm mixture is shown in Table 4.12. The summary includes the average |E*| values at each temperature and frequency, along with the CV for the averages. The averages and CV are consistent for the data, although it should be noted that the CV for the 50°C data is lower than for the 37°C data. The CV values for the 50°C data appear to be in fairly good agreement with the 4 and 21°C data; this might suggest some error in the 37°C testing though

TABLE 4.10
Category 4, 19.0-mm statistical summary.

	N30		**N50**		**N70**		**N100**	
25 Hz	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
6°C	19,554	15.2	20,041	6.4	20,145	9.6	16,166	9.0
22°C	11,276	3.7	12,465	7.4	12,132	9.3	10,088	11.0
37°C	4,157	6.4	4,552	9.4	5,299	15.0	3,561	7.7
50°C	1,431	7.9	1,743	6.3	1,792	13.1	1,366	0.6
10 Hz	**N30**		**N50**		**N70**		**N100**	
	Average |E*|, MPa	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
6°C	16,770	1.8	18,477	5.2	18,277	7.2	14,898	5.2
22°C	10,328	3.3	11,097	4.0	11,219	5.4	9,207	11.3
37°C	3,236	7.6	3,547	6.4	4,251	10.7	2,841	6.7
50°C	1,089	7.9	1,299	9.1	1,341	10.8	1,052	2.1

TABLE 4.11
Category 4, 19.0-mm Bonferroni groupings.

Test Frequency	25 Hz		10 Hz	
Test Temperature	p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
6°C	0.0933	(N70=N50=N30=N100)	0.0028	(N50=N70=N30); (N30=N100)
22°C	0.0295	(N50=N70=N30); (N70=N30=N100)	0.0080	(N70=N50=N30); (N30=N100)
37°C	0.0065	(N70=N50=N30); (N50=N30=N100)	0.0008	(N70=N50=N30); (N50=N30=N100)
50°C	0.0042	(N70=N50); (N50=N30); (N30=N100)	0.0096	(N70=N50=N30); (N50=N30=N100)

TABLE 4.12
Category 3, 9.5-mm statistical summary.

	N30		N50		N70		N100/7%		N100/5%	
25 Hz	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %
4°C	19,715	4.4	19,413	4.0	18,226	8.2	16,144	13.1	18,055	6.2
21°C	10,529	6.3	10,480	3.7	9,504	8.8	8,351	10.7	9,660	10.7
37°C	2,756	8.1	2,678	13.4	2,529	11.2	3,046	20.2	3,109	5.7
50°C	919	7.5	1,114	5.3	1,081	8.1	944	6.2	1,085	5.8
10 Hz	**N30**		**N50**		**N70**		**N100/ 7%**		**N100/5%**	
	Average |E*|, MPa	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
4°C	18,492	6.1	17,843	3.3	17,543	6.7	15,639	6.4	17,089	5.5
21°C	9,331	7.1	10,042	5.6	8,864	9.2	7,628	11.5	8,921	9.2
37°C	2,191	10.7	2,136	15.4	2,092	14.0	2,461	20.4	2,588	6.3
50°C	671	7.9	811	4.4	789	7.0	707	6.1	810	4.2

none was detected during testing or subsequent data analyses.

In general, the standard N100 mixture design with test specimens compacted to 7 percent air voids has the lowest |E*| value. The only exception is at 37°C when the N100 standard mixture (7 percent air voids) has the second highest |E*| value at both 10 and 25 Hz; the N100 mixture with specimens compacted to 5 percent air voids has the highest |E*| value at both frequencies.

Results of the Bonferroni method are shown in Table 4.13. This data indicates that at a test temperature of 37°C, there are no differences in the |E*| values of the five mixtures for both test frequencies (10 and 25 Hz). At 50°C, for both frequencies, the N30 and N100/5% mixtures are not equivalent to any other mixtures. At the two higher temperatures, the results are somewhat contradictory. This suggests that the re-designed mixtures may or may not perform as well as the standard mixture.

4.4.3 Category 4, 9.5-mm Mixture

The statistical summary of the |E*| data for the Category 4, 9.5-mm mixture is shown in Table 4.14. The summary includes the average |E*| values at each temperature and frequency, along with the CV for the averages. The averages and CV are consistent. The N30 and N50 re-designed mixtures tend to have higher |E*| values than the standard N100 mixture.

The Bonferroni method results are shown in Table 4.15. This data indicates that at a test temperature of 21°C and a frequency of 25 Hz, there is no statistically significant difference between the N30, N50, and N100 mixtures. Also, at 37°C, the N100 mixture is not equivalent to the other mixtures at 25 Hz; however, at 10 Hz, it is the N50 mixture that appears to be unequal to the others. Overall it would appear that the re-designed mixtures are at least as stiff as the standard mixture and thus equivalent rutting performance would be expected.

4.5 Field Trials

The SAC suggested two trial sections be placed using the modified asphalt mixture design method developed in the study to investigate if the laboratory findings were applicable to the field. The Indiana Department of Transportation (INDOT) complied with this request; a search was made for asphalt paving contracts (Summer 2013 and Winter 2014) that would be suitable for such a field trial. Several locations were identified and then eliminated for various reasons. Eventually, INDOT settled on two projects and the first trial section was built on SR-13 near Fort Wayne, Indiana

TABLE 4.13
Category 3, 9.5-mm Bonferroni groupings.

Test Frequency	25 Hz		10 Hz	
Test Temperature	p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
4°C	0.03729	(N50=N70=N100/7%); (N30=N50=N100/5%)	0.01992	(N70=N100/7%); (N30=N50=N70=N100/5%)
21°C	0.02429	(N70=N100/7%); (N30=N50 =N100/5%=N70)	0.01885	(N70=N100/7%); (N30=N50=N100/5%=N70)
37°C	0.32682	(N100/5%= N100/7%=N30=N50=N70)	0.35636	(N100/5%= N100/7%=N30=N50=N70)
50°C	0.00439	(N50=N100/5%=N70= N100/7%=N30)	0.0021	(N50=N100/5%=N70= N100/7%=N30)

TABLE 4.14
Category 4, 9.5-mm statistical summary.

	N30		**N50**		**N100**	
25 Hz	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
4°C	17,682	2.8	17,610	0.9	15,632	1.9
21°C	8,392	3.7	8,308	2.5	8,025	3.7
37°C	2,970	4.8	3,297	2.8	2,721	4.4
50°C	1,093	8.8	1,171	4.1	971	11.7
10 Hz	**N30**		**N50**		**N100**	
	Average |E*|, MPa	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
4°C	16,107	3.0	16,055	1.4	14,591	2.3
21°C	6,957	4.0	6,897	2.6	7,364	1.1
37°C	2,334	7.4	2,632	2.7	2,105	6.8
50°C	910	8.9	829	3.1	731	12.1

TABLE 4.15
Category 4, 9.5-mm Bonferroni groupings.

Test Frequency Test Temperature	25 Hz		10 Hz	
	p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
4°C	0.00002	(N100); (N30=N50)	0.00031	(N50=N100); (N30=N50)
21°C	0.20278	(N30=N50=N100)	0.04165	(N100=N30); (N30=N50)
37°C	0.00039	(N100); (N50=N30)	0.00136	(N50); (N30=N100)
50°C	0.03879	(N30=N100); (N50=N30)	0.03752	(N30=N50); (N50=N100)

and the second on Georgetown Road in Indianapolis, Indiana. During construction, mixture samples from both the conventional and trial pavement sections were obtained. Additionally, in November 2013, approximately four months after construction, INDOT took several cores from the SR-13 trial section and 20 cores per each design were obtained from Georgetown Road.

4.5.1 SR-13 Project

The SR-13 project called for the surface to be milled and an overlay placed. The new asphalt overlay mixture was designed as a Category 4, 9.5-mm surface. This means the design was done using 100 gyrations of the SGC and optimum asphalt binder content chosen at 4% air voids. It was initially decided by the SAC, in consultation with the research team that the re-designed mixture for use on the project should be designed using 30 gyrations of the SGC and selecting the optimum asphalt binder content at 5% air voids. Later, the decision was made to change the 30-gyration design to a 50-gyration design instead. Both the original and re-designed mixtures consisted of steel slag and limestone coarse aggregates, limestone and natural sands, recycled asphalt shingles (RAS), and a PG 70-22 asphalt binder. The aggregate gradations of both mixtures are given in Table 4.16 and shown graphically in Figure 4.11. Relatively small changes in the aggregate proportions were needed to allow the re-designed mixture to achieve the design air voids at reduced gyrations.

4.5.1.1 Pre-Field Trial Laboratory Testing. The SAC, in consultation with the research team decided that the re-designed mixture for the project should be designed using 30 gyrations with optimum asphalt binder content selected at 5 percent air voids. Once the mixture re-design was complete, the contractor produced 150 mm diameter SGC specimens for both the standard (N100) and re-designed (N30) mixtures. These specimens were then cored to produce 100 mm diameter specimens for |E*| and FN testing. The test specimens' air voids are shown in Table 4.17. As seen in the table, the air voids for both specimen groups are low. The N100 specimens should have average air voids of approximately 7 percent; the N30 specimens should have average air voids of approximately 5 percent. The results are 1.2% and 1.3% lower than target values for the N100 and N30 designs respectively.

TABLE 4.16
SR-13, Aggregate gradations and specific gravities.

	Original (N100)	Re-designed (N50)
Sieve Size, mm	Percent Passing	
12.5	100.0	100.0
9.5	92.0	89.6
6.3	71.8	70.4
4.75	62.0	61.1
2.36	46.4	40.1
1.18	32.0	27.7
0.6	22.7	19.5
0.3	13.4	11.0
0.15	7.5	6.0
0.075	4.8	4.4
G_{sb}	2.953	2.972

However, since project construction had already started and time was short, it was decided to test the specimens.

The specimens were tested to determine |E*| at 37°C over a range of frequencies. The |E*| data for the two mixtures are in Table 4.18 and are shown plotted in Figure 4.12. As the data clearly show, the re-designed (N30) mixture has a higher |E*| value at every frequency. However, statistically, there are no differences between the |E*| values for the mixtures (α=0.05). Therefore, it was concluded that the N30 mixture is at least as stiff as the N100 mixture and should therefore perform as well in the field, if it is compacted to 5 percent air voids.

When the |E*| testing was complete, due to time and material availability challenges, the same specimens were reused for FN testing. Again, this is not in compliance with the standard test methods, but it does allow for a relative comparison of the mixtures. The N30 mixture yielded a FN of 1181 (C.V.=33.2%) and the N100 mixture a FN of 841 (C.V.=12.9%). A statistical comparison indicates that there is no significant difference between these values. This suggests that the re-designed N30 mixture will have at least same rutting performance in the field as the N100 standard mixture. However, it is again important to remember that to do so, the re-designed mixture must be compacted to 5 percent air voids in the field. The net effect of laboratory comparison of the two mixture designs is that they have about the same mechanical properties as regards stiffness and rutting resistance.

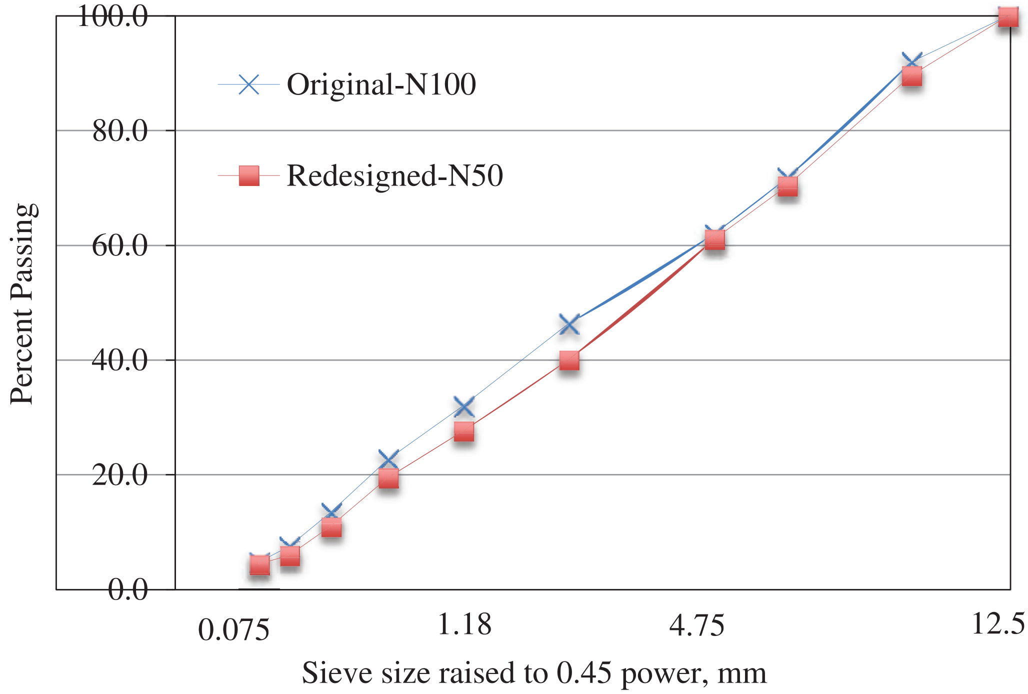

Figure 4.11 SR-13, Field trial mixture gradations.

TABLE 4.17
Test specimen air voids.

N_{design}	Sample 1	Sample 2	Sample 3	Average	SD
100	5.6	6.3	5.6	5.8	0.38
30	4.0	3.9	3.1	3.7	0.53

4.5.1.2 Field Trial Production Testing. Based on the pre-trial laboratory testing of standard (N100) and re-designed (N30) mixtures, two different sections were constructed on SR-13, one with the 100-gyration mixture and the other with the 30-gyration mixture. During the production and placement of the re-designed mixture two companion samples were taken from the truck at 160 tons of production. One sample was extracted using the solvent method; the binder from the second sample was removed using the ignition oven method. No compacted specimens were produced or maximum specific gravities determined for these samples. The aggregate gradations and binder contents for these two samples are shown in Table 4.19.

Given the data results obtained from the 160 tons samples, a decision was made to change the aggregate blend in order to adjust the gradation. This resulting aggregate blend was used for all subsequent mixture production. A sample of the mixture was again taken from a truck at 380 tons and the material used to determine the aggregate gradation, binder content, and volumetric properties. As seen from Table 4.18, the binder content, air voids, and VMA were all higher than the target values.

Since the volumetrics were high when 30 gyrations were used to compact the specimens, additional specimens were produced from the 380 tons sample; these

TABLE 4.18
Dynamic modulus data.

	N30		N100	
Test Frequency, Hz	Average |E*|, MPa	CV, %	Average |E*|, MPa	CV, %
0.1	584	11.4	421	12.3
0.2	737	9.7	540	12.1
0.5	1,002	8.7	751	12.0
1.0	1,258	8.0	954	11.7
2.0	1,578	7.7	1,206	11.5
5.0	2,131	7.7	1,659	10.6
10.0	2,645	7.6	2,072	10.0
20.0	3,257	7.2	2,575	9.6
25.0	3,427	6.2	2,708	9.5

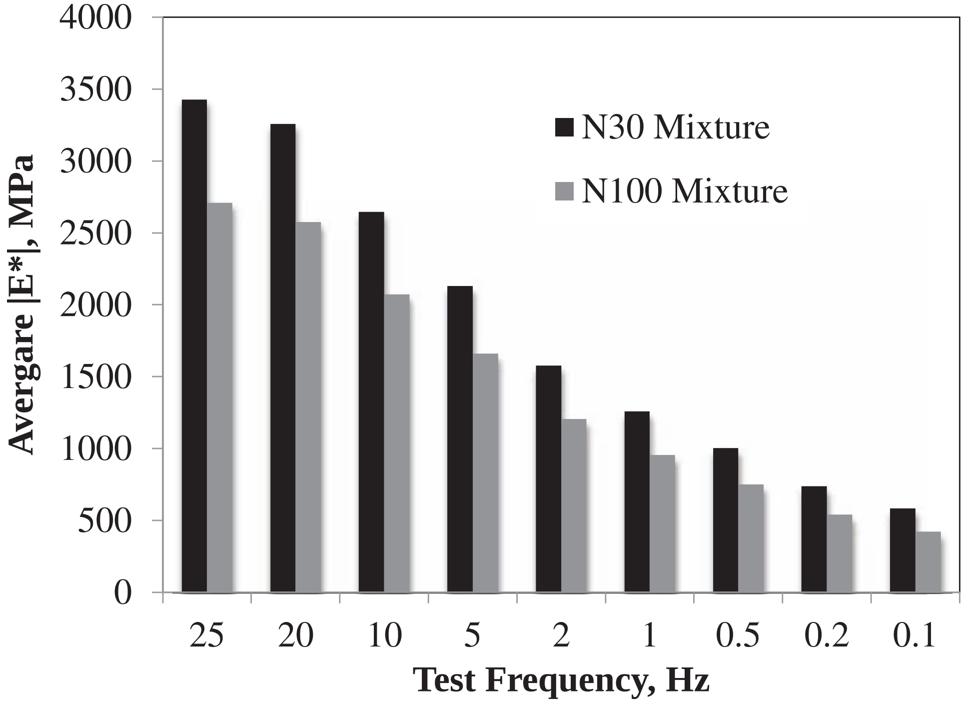

Figure 4.12 Dynamic modulus data for the mixture designs.

TABLE 4.19
Re-designed mixture data (truck samples).

Sample Point, tons		160		380		580
Sieve Size, mm	**Design**	**Solvent**	**Ignition**	**N30**	**N50**	**N50**
12.5	100.0	100.0	100.0	100.0	Same gradation and binder content as N30	100.0
9.5	92.0	89.4	92.5	93.1		89.6
4.75	63.0	56.7	60.2	65.4		61.1
2.36	41.0	35.9	37.5	42.8		40.1
1.18	27.5	24.5	25.3	29.1		27.7
0.600	19.0	17.3	17.7	20.2		19.5
0.300	12.0	10.5	10.5	11.2		11.0
0.150	7.5	6.3	6.2	6.2		6.0
0.075	5.5	4.9	4.7	4.6		4.4
Binder Content, %	5.10	5.38	5.38	5.65		5.32
Air Voids, %	5.0	–[1]	–[1]	6.7	5.5	–[1]
VMA, %	15.3	–[1]	–[1]	19.0	18.0	–[1]

–[1] Not determined.

latter specimens were compacted with 50 gyrations of the SGC. As seen in Table 4.20, using 50 gyrations resulted in volumetrics closer to the target values. Given the results from this testing, the decision was made to change the re-designed mixture from a 30-gyration to 50-gyration mixture design. The VMA at 50 gyrations was still high, but the steel slag was known to have a variable specific gravity, so less confidence was placed on the calculated number.

The final truck sample was taken at 580 tons. The ignition oven was used to determine aggregate gradation and asphalt binder content. No compacted specimens were produced or maximum specific gravities determined for this sample. As can be seen from Table 4.20, the gradation was reasonable and the binder content was slightly high but closer to target value.

Plate sampling from behind the paver was also completed during the production and placement of the re-designed mixture according to the INDOT standard method. The test results obtained from plate samples are shown in Table 4.20. The ignition oven was used to determine binder content and provide aggregate gradations for all plate samples.

The first plate sample was obtained at 380 tons and is from the same tonnage as the N30 and N50 truck samples discussed previously (Table 4.19). Subsequent plate samples were taken at 482, 816, 1310 tons. All of the plate sample test data was obtained after the decision was made to change the re-designed mixture to a 50-gyration mixture.

The plate sample data indicate that aggregate gradation control was good. However, the binder content was high and the laboratory air voids low in all of the samples. The mixture VMA was close to the target, but variable, most likely due to issues with the steel slag G_{sb}. For this reason, little emphasis should be placed on VMA values. Finally, the overall average road core density for the three sublots was 94.7%, close to the 95.0% target in-place density. This density was achieved without any changes to the rollers or rolling patterns that were used for the standard (N100) mixture.

Overall the data from the field trial appears to indicate that changing the re-designed mixture's design gyrations from 30 to 50 probably should not have been done. The change was made based on one sample that was taken early in the production process. Using the field data and volumetric relationships established by the Bailey Method, it is estimated that had the re-designed mixture remained a 30-gyration design, the air voids and VMA would have increased by about 1.18%, on average. The estimated air voids and VMA values are shown in Table 4.21. The average laboratory air voids is estimated at 4.9%, almost exactly the target value of 5.0 percent. The estimated VMA is high, but again, this is likely due to issues with the steel slag G_{sb}, and perhaps the VMA should therefore be de-emphasized for this project.

A total of nine sublots of the original mixture and three sublots of the re-designed mixture were placed. For a surface mixture, INDOT specifications define a sublot as 544 tonnes (600 tons) of mixture. As a standard quality assurance measure INDOT extracted two cores from each sublot to establish the in-place mixture densities. The data is summarized in Table 4.22. The overall average in-place density of the original mixture was 91.8%; for the re-designed mixture the average was 94.7%, close to the goal of 95%. The contractor reported that the re-designed mixture field density was achieved without making any changes in the rollers or rolling patterns.

4.5.1.3 Post-Field Trial Testing. The primary reason for modifying the mixture design method is to produce asphalt mixtures that can be compacted to higher

TABLE 4.21
Estimated volumetric properties at 30 gyrations.

	Sublot 1	Sublot 2	Sublot 3	Average
Air Voids, %	5.1	4.8	4.7	4.9
VMA, %	17.2	16.6	17.2	17.0

TABLE 4.22
SR-13, Field core densities.

Mixture	Sublot Number	Density, Percent of G_{mm} Core 1	Core 2	Average
Original (N100)	1	92.3	89.7	91.0
	2	90.3	94.6	92.5
	3	92.6	92.7	92.6
	4	92.4	93.9	93.1
	5	90.5	90.0	90.3
	6	90.2	90.0	90.1
	7	92.4	91.4	91.9
	8	92.6	92.4	92.5
	9	92.3	92.4	92.3
Re-designed (N50)	1	92.3	94.5	93.4
	2	93.6	94.7	94.1
	3	96.3	96.7	96.5

TABLE 4.20
Re-designed mixture data (plate samples).

Sample Point, tons / Sieve Size, mm	Design	380 Ignition	482 Sublot 1	816 Sublot 2	1310 Sublot 3
12.5	100.0	100.0	100.0	100.0	100.0
9.5	92.0	91.3	91.9	90.3	92.7
4.75	63.0	63.1	65.8	61.3	63.6
2.36	41.0	41.8	42.3	40.7	41.0
1.18	27.5	28.7	28.4	28.0	28.3
0.600	19.0	20.2	19.8	19.9	20.1
0.300	12.0	11.5	11.4	11.5	11.5
0.150	7.5	6.5	6.3	6.4	6.5
0.075	5.5	4.9	4.6	4.8	4.9
Binder Content, %	5.10	5.48	5.61	5.47	5.45
Air Voids, %	5.0	3.4	3.9	3.6	3.5
VMA, %	15.3	15.5	16.0	15.4	16.0
Road Core A Density, % G_{mm}	Not Applicable	–[1]	92.30	93.59	96.29
Road Core B Density, % G_{mm}	Not Applicable	–[1]	94.53	94.68	96.69
Average Road Core Density, % G_{mm}	Not Applicable	–[1]	93.42	94.14	96.49

–[1] Not determined.

densities in the field. Typically, an asphalt mixture is compacted to approximately 93 percent density in the field. With the modified mixture design method, the contractor was able to achieve approximately 95 percent density during construction. It is hypothesized that the additional densification will extend the life of an asphalt mixture by slowing the embrittlement of the asphalt binder and increasing the fatigue life of the asphalt mixture. This hypothesis can be tested through binder and mixture testing as outlined in Table 4.23.

The aged conditions must be approximated. The post-construction condition (immediately after construction) was simulated using loose mixture samples taken during the time of construction. To approximate eight years of in-service aging, the mixtures were tested after being conditioned according to AASHTO R30 (2002).

All of the field material used for the post field trial testing was collected after producing 380 tons of mixture on the project, when the change was made to the 50-gyration mixture. Again, specimens for the standard design were produced at 7 percent air voids according to current test method standards. Specimens for the re-designed mixture were produced at 5 percent air voids. In order to produce the test specimens the field-sampled mixtures had to be re-heated, split into appropriate sizes, and the specimens compacted. All re-heating was performed carefully and consistently at the lowest possible temperature. Once the samples were produced they were tested as previously described.

TABLE 4.23
Post field trial testing plan.

	Approximate Mixture Aged	
Test	**Post Construction**	**Condition 8 Years**
Binder Grading	X	X
Dynamic Modulus	X	X
Flow Number	X	X
Fatigue Test	X	X

4.5.1.3.1 Dynamic Modulus and Flow Number Testing. The dynamic modulus and flow number were determined for the post-construction and long-term aged mixtures according to AASHTO TP 79 as previously described. Figures 4.13 and 4.14 show plots of the |E*| data from the two mixtures; Figure 4.13 is the more common log-log plot of the data, while Figure 4.14 shows the data in semi-log form. This latter plot normally allows a better view of the high frequency data. The results of both plots clearly show that the re-designed mixture (N50) has a lower |E*| value over the frequency range than does the standard (N100) mixture. This result is different than the findings from the earlier laboratory study, which indicated that the re-designed mixtures typically had |E*| values at least as high as, and often higher than, the standard design. Testing and analyses of the field mixtures during the mixture design phase of the SR-13 project indicated no differences in |E*| values between the re-designed and standard mixtures.

To decide if there is an actual difference between the |E*| values of the two plant-produced mixtures, a t-Test was used to compare the means. Table 4.24 is a summary of the |E*| data of the field mixtures. The statistical analysis indicates that with 95 percent

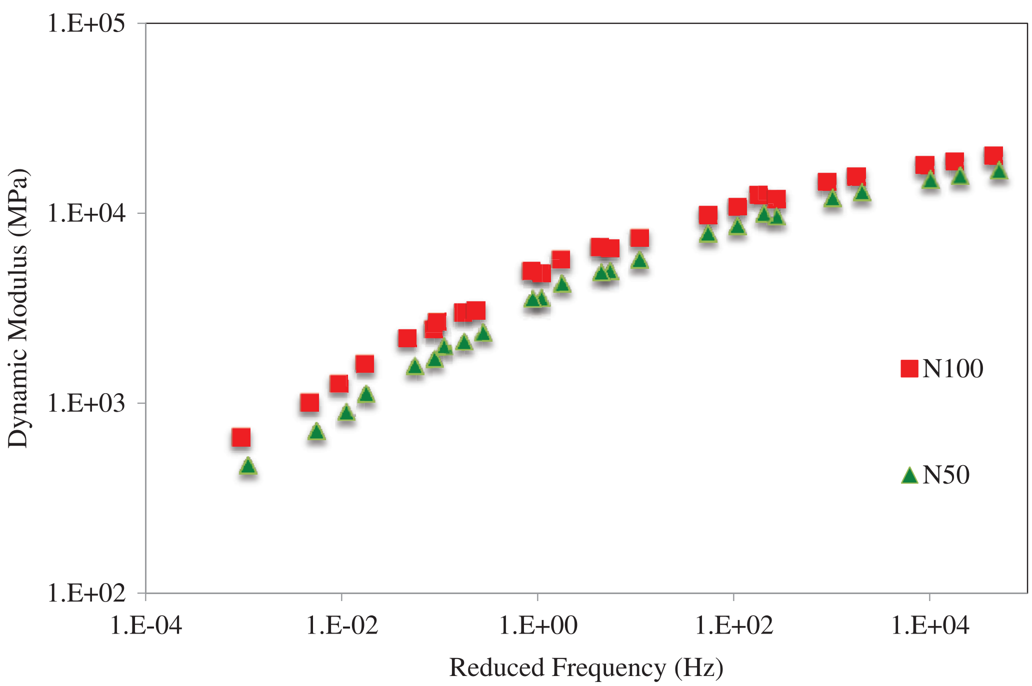

Figure 4.13 Post-construction mixture master curves (log-log).

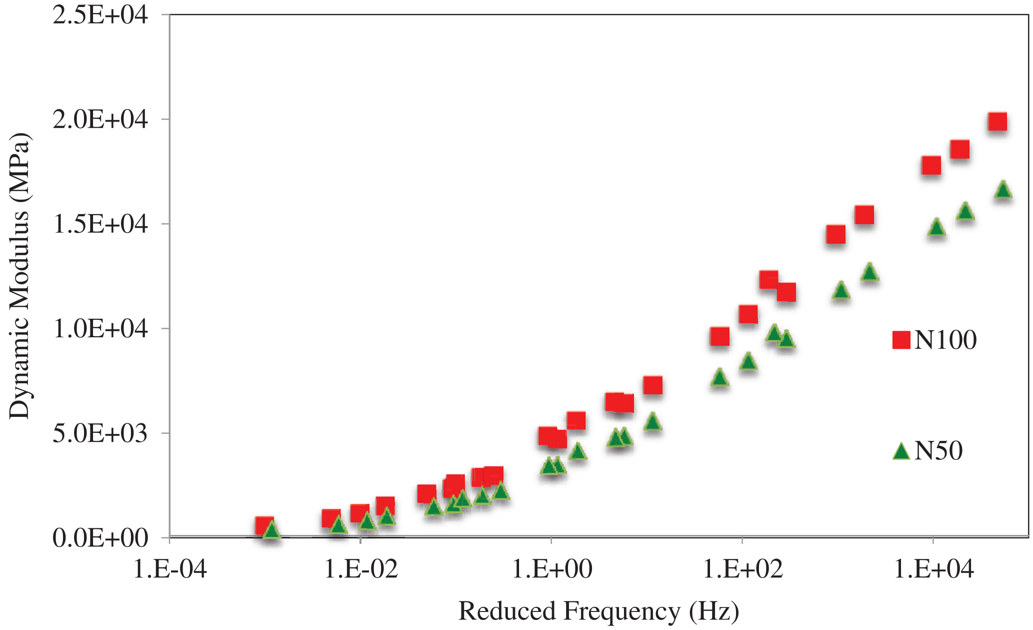

Figure 4.14 Post-construction mixture master curves (semi-log).

TABLE 4.24
Post-construction mixture statistical summary.

25 Hz	N100 Average |E*|, MPa	N100 C.V., %	N50 Average |E*|, MPa	N50 C.V., %
4°C	19,980	4.0	16,754	2.2
21°C	11,819	2.3	9,595	4.9
37°C	6,572	4.0	4,867	8.1
50°C	3,059	8.3	2,343	8.1

10 Hz	N100 Average |E*|, MPa	N100 C.V., %	N50 Average |E*|, MPa	N50 C.V., %
4°C	18,648	3.4	15,713	1.6
21°C	10,764	3.8	8,544	6.5
37°C	5,683	6.1	4,243	6.1
50°C	2,673	8.0	1,977	7.8

TABLE 4.25
Post-construction mixture flow number results.

Design	Air Voids, %	Flow Number	Strain @ Flow Number (μm)
N100-1	7.4	8,224	18,402
N100-2	6.8	5,716	18,793
N100-3	6.6	6,473	18,412
N100-4	6.7	7,447	18,869
N100-5	7.3	6,412	22,594
Average	7.0	6,854	19,414
C.V., %	5.2	14.3	9.2
N50-1	4.5	1,476	23,886
N50-2	5.4	2,066	25,344
N50-3	5.4	1,638	23,188
N50-4	5.2	2,484	21,984
N50-5	5.0	1,831	25,005
Average	5.1	1,899	23,881
C.V., %	7.3	20.8	5.7

confidence (α=0.05) it can be concluded that the standard and re-designed mixture designs have different |E*| values; the |E*| values are higher for the standard mixture.

Table 4.25 shows the FN results for the re-designed and standard mixtures. The data indicate the standard mixture has a higher flow number and less cumulative strain at FN than the re-designed mixture. This suggests the standard mixture may have better rutting performance than the re-designed mixture and is again opposite the FN results obtained during the mixture design phase of the field trial.

To predict behavior after eight years of in-service life, the flow number testing was completed using specimens that had been first been used in the dynamic modulus tests due to lack of materials. The |E*| data from the two mixtures is summarized in Table 4.26 and plotted in Figures 4.15 and 4.16. While the log-log plot (Figure 4.15) shows the two mixtures have similar modulus curves, the semi-log plot indicates otherwise, with the re-designed mixture (N50) having a lower |E*| value over the frequency range.

A standard t-Test was used to determine if the apparent difference in the mean |E*| values of the two mixtures were statistically significant. The Bonferroni method results are shown in Table 4.27. The analysis indicates that with 95 percent confidence (α=0.05) it can be concluded that there is no statistical difference between the mean |E*| values of the standard and re-designed mixtures.

Table 4.28 shows the FN results for the re-designed and standard field mixtures. The data indicates both designs have almost the same flow number and cumulative strain at FN. This suggests both designs may have similar rutting performance in the field. The Bonferroni method results are shown in Table 4.29. The analysis indicates that with 95 percent confidence ($\alpha=0.05$) it can be concluded that there are no significant difference between the mean flow number and mean strain at flow number for the standard and re-designed mixtures.

4.5.1.3.2 Beam Fatigue Test. Samples of both the standard and re-designed mixtures were tested according to AASHTO T 321 (2014). For each mixture (standard and re-designed) two sets of beams (three beams per set) were prepared. One set per mixture was tested in the post-construction condition; the second set was tested after oven conditioning, to approximate the mixtures' fatigue properties after eight years of in-service life. Table 4.30 shows the air void contents of the beam specimens.

The average target air void contents for the beam specimens were 7 percent and 5 percent for the standard and re-designed mixtures respectively. As indicated in the table, these targets were not met, but due to lack of material it was not possible to make additional beam specimens. Also as shown in the table, the beam specimens were placed into groups such that the average air voids of the two groups (for each mixture) were approximately equal. All the testing was completed at 20°C. Tables 4.31 and 4.32 show the initial stiffness and number of cycles to failure results.

During testing of N100 post-construction Specimen 2, the temperature chamber failed so the temperature was not held constant. The result was an abnormal decline in beam flexural stiffness. Additionally, data from one of the standard design (Specimen 3), eight-year specimen does not follow the expected trend of dissipated energy, which has to decrease with load cycle increment. The reason for this has not been identified, but the inaccurate results have not been included in the analyses.

The fatigue test results indicate that although the standard mixture has slightly lower initial stiffness, it may endure more cycles to failure when compared to the re-designed mixture, for both the post-construction and after eight years of service conditions. Also, while the increase in initial stiffness after temperature conditioning is to be expected, the increase in fatigue life after temperature conditioning is counter-intuitive. As an asphalt mixture ages, it would be expected to become more brittle, thereby causing the fatigue life of the mixture to decrease, not increase.

The Bonferroni method results are shown in Table 4.33. The analysis indicates that with 95 percent

TABLE 4.26
Aged (8-year) dynamic modulus data.

	N100		N50	
@ 25 Hz	**Avg. |E*|, MPa**	**C.V., %**	**Avg. |E*|, MPa**	**C.V., %**
4°C	21482	8.8	20261	5.8
21°C	14089	9.5	13060	10.0
37°C	8648	9.0	8007	11.5
50°C	5130	19.3	4561	18.9
@ 10 Hz	**N100**		**N50**	
	Avg. |E*|, MPa	**C.V., %**	**Avg. |E*|, MPa**	**C.V., %**
4°C	20258	5.4	19102	6.2
21°C	12802	10.1	11920	10.7
37°C	7723	10.2	7018	13.4
50°C	4253	21.5	3681	25.6

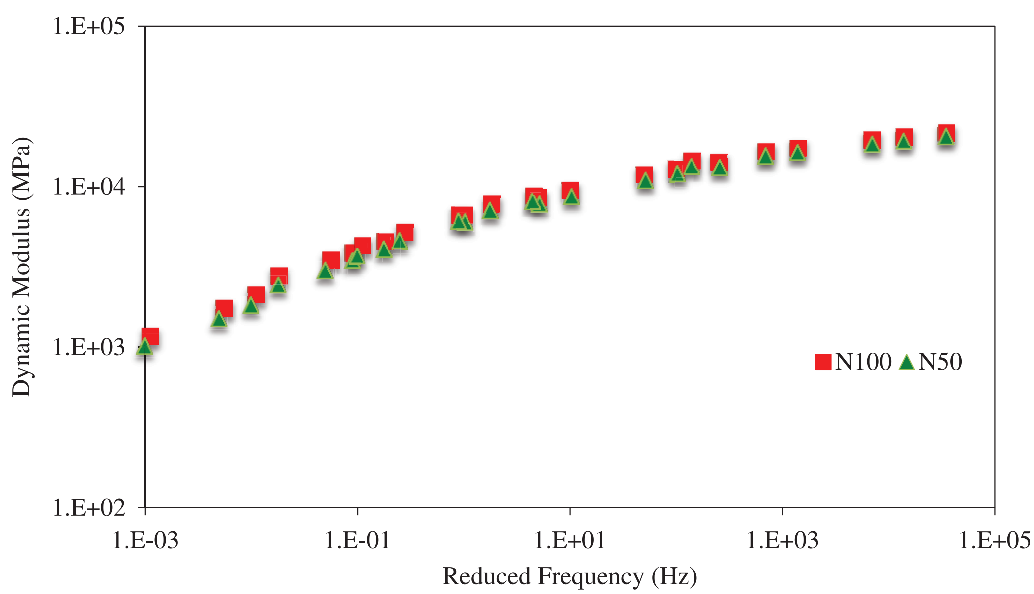

Figure 4.15 Aged (8-year) mixture master curves (log-log).

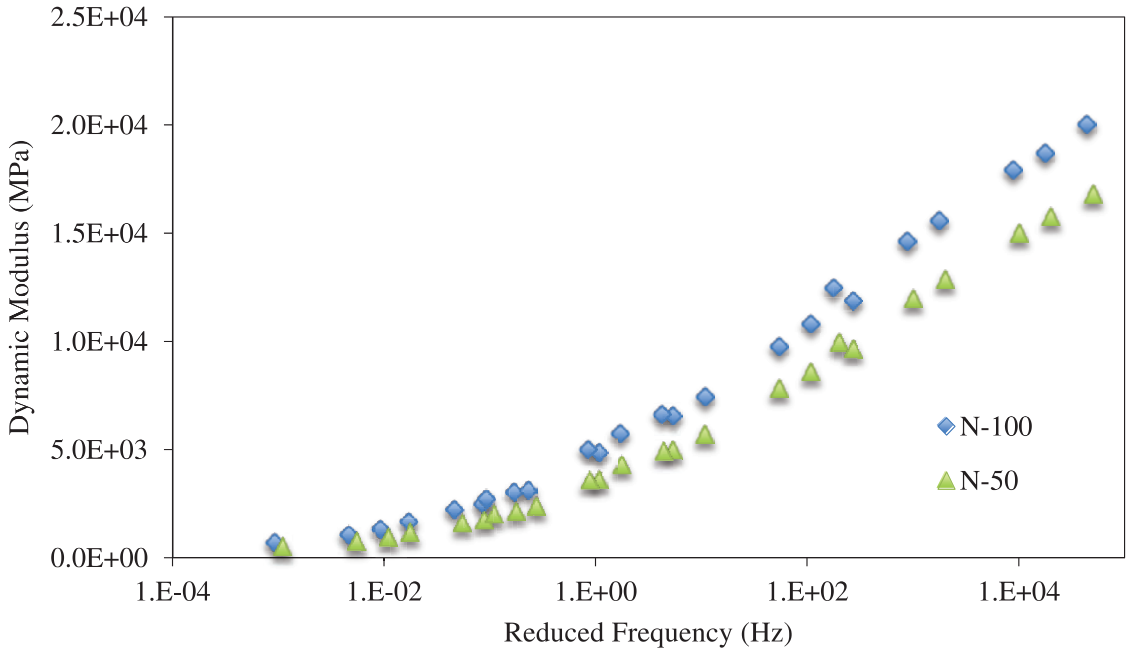

Figure 4.16 Aged (8-year) mixture master curves (semi-log).

TABLE 4.27
Aged (8-year) mixture Bonferroni groupings.

Test Frequency	25 Hz		10 Hz	
Test Temperature	p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
4°C	0.33	(N100=N50)	0.24	(N100=N50)
21°C	0.35	(N100=N50)	0.41	(N100=N50)
37°C	0.38	(N100=N50)	0.34	(N100=N50)
50°C	0.45	(N100=N50)	0.46	(N100=N50)

TABLE 4.28
Aged (8-year) flow number data.

	N100					
	Sample 1	Sample 2	Sample 3	Sample 4	Avg.	SD
Air voids, %	8.1	7.8	6.0	–[1]	7.3	1.1
Flow Number (FN)	8976	9380	8194	–[1]	8850	603
Strain @ FN, $\mu\varepsilon$	12,066	9,751	15,966	–[1]	12594	3141
	N50					
	Sample 1	Sample 2	Sample 3	Sample 4	Avg.	SD
Air voids, %	6.3	6.9	6.0	5.2	6.1	0.7
Flow Number (FN)	8759	7345	10000	10000	9026	1264
Strain @ FN, $\mu\varepsilon$	13,477	15,878	9,952	17,058	14091	3136

[1]No specimens were available for testing.

confidence (α=0.05) it can be concluded that there is no significant difference between the initial stiffness and cycles to failure for the standard and re-designed mixtures, which might be a result of high standard deviation within the replicates. Overall, the beam fatigue results of the experiment are questionable, at best.

4.5.1.3.3 Asphalt Binder Grading. The asphalt binders in the mixture samples at the two aged conditions (post-construction and eight-year in-service life) were recovered and tested to determine the binder grades according to AASHTO M320 (2010). A PG 70-22 was the original binder used for both designs. Due to lack of materials, it was necessary to recover the binder from the beam fatigue specimens, after the fatigue testing had been completed. In order to do so without further aging the binder, once the beams had been fatigue tested they were heated only enough to allow them to be broken apart into loose mixture. Binder recovery was then completed according to AASHTO T 319 (2015). Following recovery the performance grade of the binders was determined according to AASHTO T 315 (2011) and AASHTO T 313 (2008). The continuous performance grades of the recovered binders are shown in Table 4.34.

Asphalt binder aging, through the oxidation process, stiffens binder thereby potentially increasing both the high and low temperatures at which an asphalt binder will meet the applicable specifications. As the data in Table 4.31 show, the high temperature grade for the post-construction binder was three standard grades higher than the original PG 70 grade for the re-designed mixture and over four standard grades higher than the original binder grade (using extrapolated 6-degree increment). There was no low temperature grade change for the post-construction re-designed mixture and only a slight temperature grade change for the post-construction original mixture.

The results for the specimens conditioned in the laboratory to mimic eight years of aging had more aging than did the post-construction mixtures. The high temperature grade increased nearly five standard grades beyond the original PG 70 for the re-designed mixture and nearly six standard grades for the original mixture. The low temperature grade for the post-construction re-designed mixture was four degrees warmer, while the low temperature grade for the original mixture was six degrees warmer.

The trend in the binder changes are as expected (i.e., aging stiffens the binder), although in some cases the results show more aging than might be expected. This could possibly be due to the addition of shingles in the mixtures. However, the results do seem to indicate the re-designed mixtures tend to age less, when compared to the standard mixture. Therefore, the re-designed mixture may offer better durability than the original mixtures.

TABLE 4.29
Aged (8-year) flow number Bonferroni groupings.

Flow Number (FN)		Strain @ FN, με	
p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
0.84	(N100=N50)	0.56	(N100=N50)

TABLE 4.30
Beam specimen air void contents.

Specimen	Age, years	Air Voids, %
N100[1]-2	0[3]	11.2
N100-4	0	7.9
N100-5	0	9.4
Average		9.5
Standard Deviation		1.65
N100-1	8	9.8
N100-3	8	10.6
N100-6	8	8.4
Average		9.6
Standard Deviation		1.11
Specimen	**Age, years**	**Air Voids, %**
N50[2]-1	0	4.6
N50-2	0	4.4
N50-5	0	8.7
Average		5.9
Standard Deviation		2.43
N50-3	8	4.0
N50-4	8	6.8
N50-6	8	8.0
Average		6.3
Standard Deviation		2.05

[1]N100 denotes the standard mixtures
[2]N50 denotes the re-designed mixtures
[3]Denotes the post-construction condition

TABLE 4.31
Beam fatigue, initial stiffness.

Mixture	Age, years	Initial Stiffness (MPa)				
		Specimen 1	Specimen 2	Specimen 3	Average	SD
N100[1]	0[3]	6266	–[4]	6878	6572	433
N100	8	7056	7965	–[5]	7511	643
N50[2]	0	8415	8569	6927	7970	907
N50	8	10186	7476	6446	8036	1932

[1]N100 denotes the standard mixtures
[2]N50 denotes the re-designed mixtures
[3]Denotes the post-construction condition
[4]Temperature was not constant during testing
[5]Did not follow the expected trend of dissipated energy versus load cycles

TABLE 4.32
Beam fatigue, number of cycles to failure.

Mixture	Age, years	Cycles to failure (multiples by 1000 sec)				
		Specimen 1	Specimen 2	Specimen 3	Average	SD
N100[1]	0[3]	177	–[4]	1030	604	603
N100	8	1050	507	–[5]	779	384
N50[2]	0	354	599	200	384	201
N50	8	755	318	249	441	274

[1]N100 denotes the standard mixtures
[2]N50 denotes the re-designed mixtures
[3]Denotes the post-construction condition
[4]Temperature was not constant during testing
[5]Did not follow the expected trend of dissipated energy versus load cycles

TABLE 4.33
Beam fatigue Bonferroni groupings.

Age, years	Initial Stiffness (MPa)		Cycles to Failure (multiples by 1000 sec)	
	p-value ($\alpha = 0.05$)	Grouping	p-value ($\alpha = 0.05$)	Grouping
0[3]	0.14	(N100[1]=N50[2])	0.76	(N100=N50)
8	0.75	(N100=N50)	0.88	(N100=N50)

[1]N100 denotes the standard mixtures
[2]N50 denotes the re-designed mixtures
[3]Denotes the post-construction condition

TABLE 4.34
Recovered asphalt binder continuous grades.

Mixture	Age, years	Original Binder Grade	Recovered Continuous Grade
Original	0	PG 70-22	PG 97-20
Original	8		PG 104-16
Re-designed	0		PG 88-22
Re-designed	8		PG 99-18

4.5.2 Georgetown Road

The city of Indianapolis, in cooperation with the INDOT and the asphalt paving contractor, agreed to a second trial project on Georgetown Road. As part of a widening and re-paving project, the city agreed to place an experimental section of intermediate asphalt mixture that was designed using the modified mixture design method. The standard and trial mixtures were placed as a 3-inch thick intermediate layer. The standard mixture was designed as a Category 3, 19.0-mm mixture using 100 gyrations of the SGC and choosing the optimum binder content at 4 percent air voids. Both mixtures made use of limestone coarse aggregate, dolomite sand, RAP, RAS, and a PG 64-22 binder.

In order to again test the hypothesis that re-designing mixtures to be more compactable will lead to more durable mixtures without making them more susceptible to permanent deformation, the testing plan shown in Table 4.35 was proposed and carried out for the Georgetown Road project.

4.5.2.1 Plant Mixed-Laboratory Compacted Testing. Material for the laboratory testing, approximately 650 lbs. each for both the original and re-designed mixtures, was collected from trucks before they left the hot mix plant. The sampled mixture was placed into boxes and returned to the laboratory for testing. As in the first trial project, test specimens for the original mixture design were laboratory compacted to produce 7 percent air voids according to current test method standards; the re-designed mixture specimens were produced at 5 percent air voids. In order to produce the test specimens, the field-sampled mixture had to be re-heated, split into appropriate sample sizes, and the specimens prepared. All re-heating was performed carefully and consistently at the lowest possible temperature.

4.5.2.1.1 Dynamic Modulus and Flow Number Testing. Dynamic modulus and FN were determined according to AASHTO TP 79 for both the original and re-designed mixtures in both post-construction (no oven conditioning) and long-term aged (AASHTO R30, 2002) conditions. In this case, separate sets of specimens were produced and tested to determine |E*| and FN. Figure 4.17 shows the |E*| master curve data for the two mixtures and two aging conditions. Figure 4.18 shows the same data in semi-log form, allowing a better view of the high frequency data.

TABLE 4.35
Georgetown Road testing plan.

Conditioning	Test	Plant Mixed-Laboratory Compacted[1]		Plant Mixed-Field Compacted (Cores)[1]	
		Control Mixture	Test Mixture	Control Mixture	Test Mixture
No Temperature Conditioning	Dynamic Modulus	4	4		
	Flow Number	4	4		
	Fatigue Properties	4	4		
Long-term Oven Conditioning (AASHTO R30)	Dynamic Modulus	4	4		
	Flow Number	4	4		
	Fatigue Properties	4	4		
No Temperature Conditioning	Pavement Density			20	20
	Dynamic Modulus			4	4
	Fatigue Properties			4	4
	Hamburg Testing			2	2
	Binder Recovery Grading			3	3
Long-term Oven Conditioning (AASHTO R30)	Dynamic Modulus			4	4
	Fatigue Properties			4	4
	Binder Recovery & Grading			3	3

[1]The number indicates the number of replicates for the test. The shaded portion indicates the corresponding testing will not be completed.

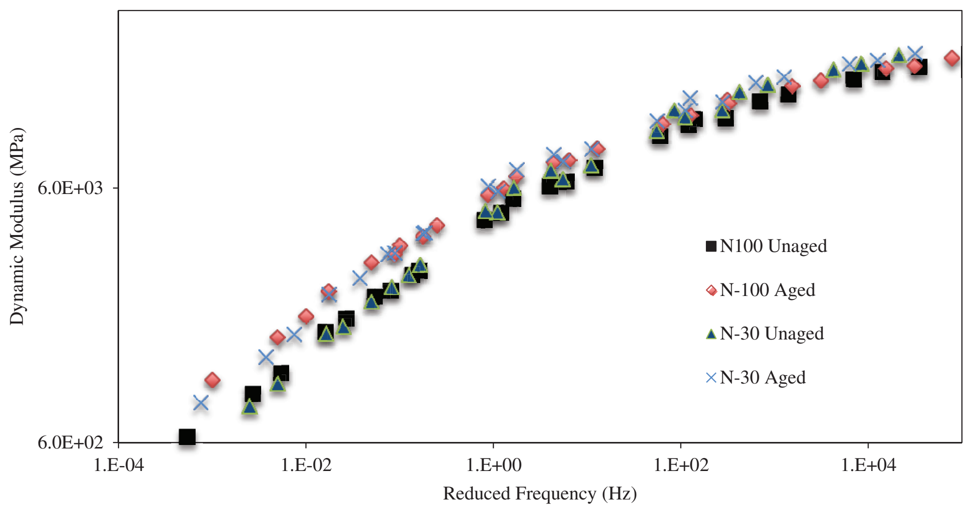

Figure 4.17 Laboratory compacted specimen master curves (log-log plot).

The results of both plots clearly show the re-designed mixture (N30) has slightly higher |E*| values over the frequency range than does the standard (N100) mixture for both aging conditions. This result is consistent with the findings from the laboratory portion of this research study, which indicated the re-designed mixtures have |E*| values at least as high as, and often higher, than the standard design.

Table 4.36 is a summary of the average |E*| data for the mixtures. To determine if significant differences exist between the |E*| values of the two mixtures a t-Test was used to compare the means. The Bonferroni method results are shown in Table 4.37. In this case the |E*| values are slightly higher for the re-designed mixture however, the statistical analysis indicates that with 95 percent confidence (α=0.05), it can be concluded that the standard and re-designed mixture designs' |E*| values are not significantly different.

Table 4.38 shows the FN results for the re-designed and standard mixtures for two aging conditions. The data indicate the standard mixture has a higher FN and less cumulative strain at FN than does the re-designed mixture for both aging conditions. This suggests the standard mixture may have better rutting performance than the re-designed mixture, opposite of the FN results obtained during the mixture design phase of the field trial. Statistical analyses indicate there are significant differences between the standard and re-designed mixture flow numbers for both aging conditions. Only the aged original mixture has a strain at flow number that is statistically different from the other mixtures and aging conditions (see Table 4.39).

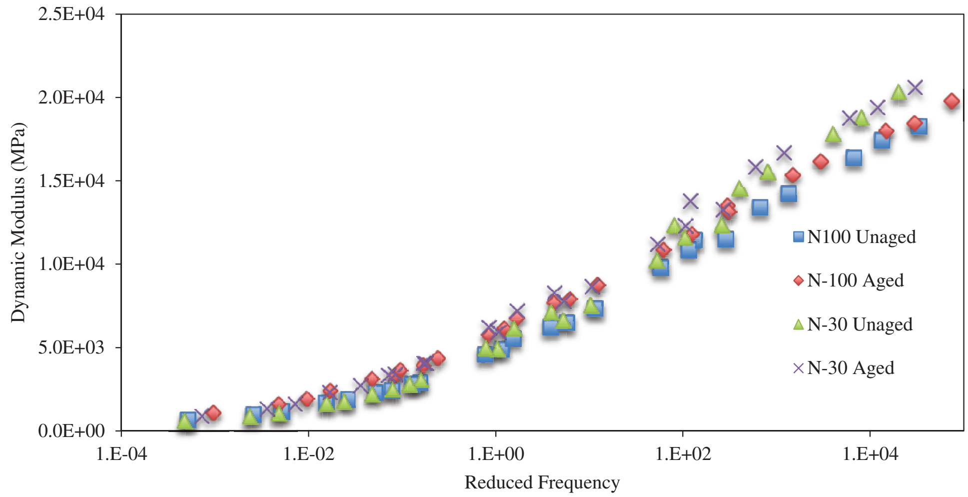

Figure 4.18 Laboratory compacted specimen master curves (semi-log plot).

TABLE 4.36
Laboratory specimen dynamic modulus statistical summary.

25 Hz	N100 Unaged		N100 Aged		N30 Unaged		N30 Aged	
	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %	Average |E*|, MPa	C.V., %
4°C	18,236	9.1	19779	7.7	20326	4.3	20603	2.4
21°C	11,488	4.7	13129	6.4	12333	4.7	13280	5.7
37°C	6,205	3.2	7674	10.3	7130	11.3	8276	6.1
50°C	2,770	8.9	4355	5.2	2778	5.3	4071	8.9
10 Hz	**N100 Unaged**		**N100 Aged**		**N30 Unaged**		**N30 Aged**	
	Average |E*|, MPa	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
4°C	17421	9.3	18422	6.4	18789	5.4	19396	3.6
21°C	10834	3.5	11774	5.1	11594	6.0	12297	3.9
37°C	5533	8.6	6778	9.7	6134	8.2	7209	6.1
50°C	2285	7.7	3630	2.8	2185	6.3	3355	9.4

TABLE 4.37
Laboratory specimen dynamic modulus Bonferroni groupings.

Test Frequency	25 Hz		10 Hz	
Test Temperature	p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
4°C	0.07713	(N30A=N30U=N100A=N100U)	0.16789	(N30A=N30U=N100A=N100U)
21°C	0.01158	(N30A=N100A=N30U); (N100U)	0.01995	(N100A=N30U=N30A); (N100U)
37°C	0.0591	(N30A=N100A=N30U); (N100U)	0.00778	(N100A=N30U=N30A); (N100U)
50°C	0.00001	(N100A= N30A); (N100U=N30U)	0.00001	(N100A= N30A); (N100U=N30U)

4.5.2.1.2 Semi-Circular Bend Testing. Due to the poor fatigue results obtained from the beam fatigue testing during the SR-13 first field trial, the research team chose to use the semi-circular bend (SCB) test to determine the fracture resistance of the asphalt mixtures for the Georgetown Road trial. This test is based on the elastic-plastic mechanism concept, which determines the critical strain energy release rate, or what is called the critical value of the J-integral (J_c). J_c is a function of the rate of strain energy change per notch depth as shown in Equation 4.1 and represents the strain energy consumed to produce a unit area of fractured surface (crack) in a mixture. Higher J_c values indicate a material is more resistant to cracking and to crack propagation.

TABLE 4.38
Laboratory compacted FN summary (51°C).

	N100-Unaged					
	Sample 1	Sample 2	Sample 3	Sample 4	Average	C.V. %
Air voids, %	6.7	6.7	7.0	7.4	7.0	4.8
Flow Number (FN)	5629	4179	4315	3218	4335	22.9
Strain @ FN, με	21,777	18,601	18,883	24,361	20906	13.0
	N30-Unaged					
	Sample 1	Sample 2	Sample 3	Sample 4	Average	C.V. %
Air voids, %	5.0	5.1	4.9	5.4	5.1	4.2
Flow number (FN)	2397	2608	3100	2293	2600	13.8
Strain @ FN, με	24,419	22,170	20,421	24,294	22826	8.4
	N100-Aged					
	Sample 1	Sample 2	Sample 3	Sample 4	Average	C.V. %
Air voids, %	6.5	7.2	6.9	7.1	6.9	4.5
Flow Number (FN)	8124	8249	8507	10000	8720	10.0
Strain @ FN, με	13,140	15,555	14,628	14,800	14531	7.0
	N30-Aged					
	Sample 1	Sample 2	Sample 3	Sample 4	Average	C.V. %
Air voids, %	5.0	5.3	5.1	5.1	5.1	2.5
Flow Number (FN)	6402	6180	6009	5412	6001	7.1
Strain @ FN, με	20,278	19,078	18,475	17,514	18836	6.1

TABLE 4.39
Laboratory compacted FN Bonferroni groupings.

Flow Number (FN)		Strain @ FN, με	
p-value (α = 0.05)	Grouping	p-value (α = 0.05)	Grouping
0.00001	(N100U); (N100A); (N30U); (N30A)	0.00896	(N100U= N30U= N30A); (N100A)

$$J_c = -\left(\frac{1}{b}\right)\frac{dU}{da} \tag{4.1}$$

where:
J_c = critical strain energy release rate (kJ/m^2)
b = specimen thickness (m)
a = notch depth (m)
U = strain energy to failure (kN-m or kJ)

The SCB testing requires that each combination of mixture and aging be tested with three different notch depths. Since four replicates of each combination were tested, 12 specimens were needed. These were produced by compacting 150-mm diameter, 57-mm tall specimens in the SGC to meet the expected air void in the field (7% air void for standard and 5% air void for re-designed mixtures). The SGC specimens were sawn in half to produce semi-circular specimens and notches were cut perpendicular to the diameter at 0.025, 0.030 and 0.035 m in depth.

Figure 4.19 shows how fracture energy changes with notch depth for the combinations of mixture and aging condition. The J_c values for these combinations are given in Table 4.40. As previously stated, the higher the J_c value, the more resistant a mixture is to cracking and to crack propagation. In the post-construction condition the re-designed mixture appears to have better cracking performance than does the standard mixture. However, after oven conditioning to simulate eight years of in-service life, the two mixtures have almost the same cracking performance. While the former result seems plausible, the latter result is counter-intuitive, if indeed the increased pavement density can improve mixture durability.

4.5.2.2 Plant Mixed-Field Compacted Testing. For both the standard and re-designed mixtures, 20 cores each were taken from the pavement immediately after construction. These 150-mm diameter cores were brought to the laboratory where the underlying layers were removed and the bulk specific gravity (G_{mb}) of each core determined. Un-compacted mixture samples taken during mixture production were used to establish the maximum theoretical specific gravity (G_{mm}) for each mixture, allowing the in-place air voids to be determined. The results are shown in Table 4.41.

4.5.2.2.1 Hamburg Testing. Four pavement cores (two cores per run) each from the control section and

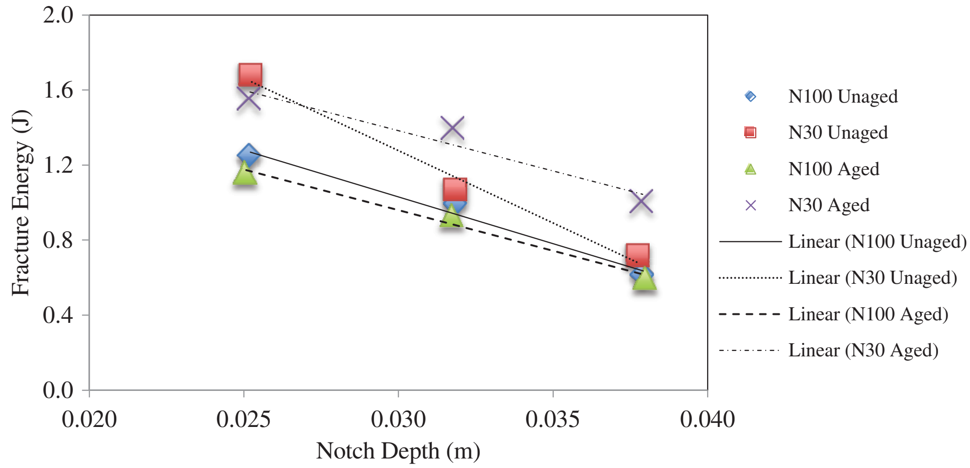

Figure 4.19 Laboratory specimen SCB test results.

TABLE 4.40
Laboratory specimen SCB critical strain energy release rate.

Mixture	Age, years	J_c (J/m^2)
N100	0	0.860
N30		1.402
N100	8	0.776
N30		0.747

the experimental field section were tested in the Hamburg wheel-tracking machine to determine mixture susceptibility to permanent deformation (rutting). This testing was completed without performing oven conditioning on the cores since such conditioning would only serve to increase a mixture's rutting resistance due to the stiffening effect. The test was completed according to AASHTO T 324-14 (2014) at a test temperature of 50°C; the Illinois Department of Transportation (IDOT) maximum threshold of 12.5 mm rut depth at 20,000 passes was applied to determine if the mixtures might have rutting problems in the field. Figures 4.20 and 4.21 show the Hamburg results for standard and re-designed mixtures. As the results indicate, the standard mixture (N100) had slightly lower rut depth than the re-designed mixture, although both mixtures have low rut depths and are well below the maximum allowable 12.5 mm of rutting.

4.5.2.2.2 Dynamic Modulus and Flow Number Testing. The pavement cores were not thick enough to use in dynamic modulus testing. Thus, the AMPT small specimen protocol was used instead to obtain the |E*| and FN results. Due to a limited number of cores, it was necessary to perform FN testing on specimens after the |E*| testing was complete. The air voids of the specimens were determined after |E*| testing and prior to FN testing, for informational purposes.

Specimens for small-scale |E*| and FN are testing are 38 mm in diameter and 110 mm tall and are cored horizontally from the road cores as shown in Figure 4.22. This allows two small specimens to be taken from each road core as shown in Figure 4.23. The testing protocol for small-scale samples is according to AASHTO TP 79, but only three test temperatures were used (4, 25, and 37C). At 50°C the small specimens are unable to survive the testing.

As with the conventional |E*| test, a master curve can be developed using the data from the small-scale test results (Figure 4.24). Figure 4.25 shows the data in semi-log form. The plots indicate the re-designed mixture (N30) has slightly higher |E*| values over the frequency range than the standard (N100) mixture design for both aging conditions. This result is consistent with the laboratory study results and those of the first field trial on SR-13.

TABLE 4.41
Plant mixed-field compacted air voids.

N100				N30			
Cores No.	G_{mb}	G_{mm}	AV%	Cores No.	G_{mb}	G_{mm}	AV%
N100-1	2.378	2.509	5.2	N30-41	2.393	2.502	4.4
N100-2	2.352		6.3	N30-42	2.401		4.0
N100-3	2.347		6.5	N30-43	2.401		4.0
N100-4	2.363		5.8	N30-44	2.396		4.2
N100-5	2.369		5.6	N30-45	2.371		5.2
N100-6	2.359		6.0	N30-46	2.382		4.8
N100-7	2.352		6.3	N30-47	2.368		5.4
N100-8	2.360		5.9	N30-48	2.365		5.5
N100-9	2.362		5.8	N30-49	2.380		4.9
N100-10	2.317		7.7	N30-50	2.369		5.3
N100-11	2.316		7.7	N30-51	2.369		5.3
N100-12	2.343		6.6	N30-52	2.378		4.9
N100-13	2.368		5.6	N30-53	2.388		4.5
N100-14	2.408		4.0	N30-54	2.382		4.8
N100-15	2.379		5.2	N30-55	2.376		5.0
N100-16	2.417		3.7	N30-56	2.382		4.8
N100-17	2.345		6.5	N30-57	2.383		4.8
N100-18	2.329		7.2	N30-58	2.377		5.0
N100-19	2.325		7.3	N30-59	2.368		5.4
N100-20	2.358		6.0	N30-60	2.387		4.6
Average AV%		6.0		Average AV%		4.8	
SD AV%		1.05		SD AV%		0.44	

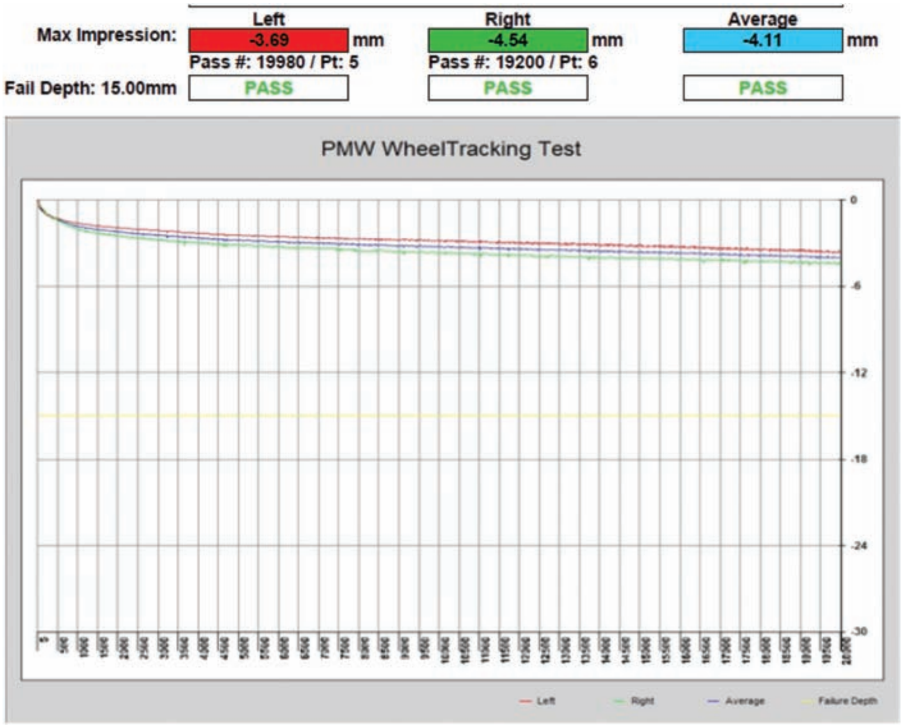

Figure 4.20 Hamburg test results, standard mixture cores.

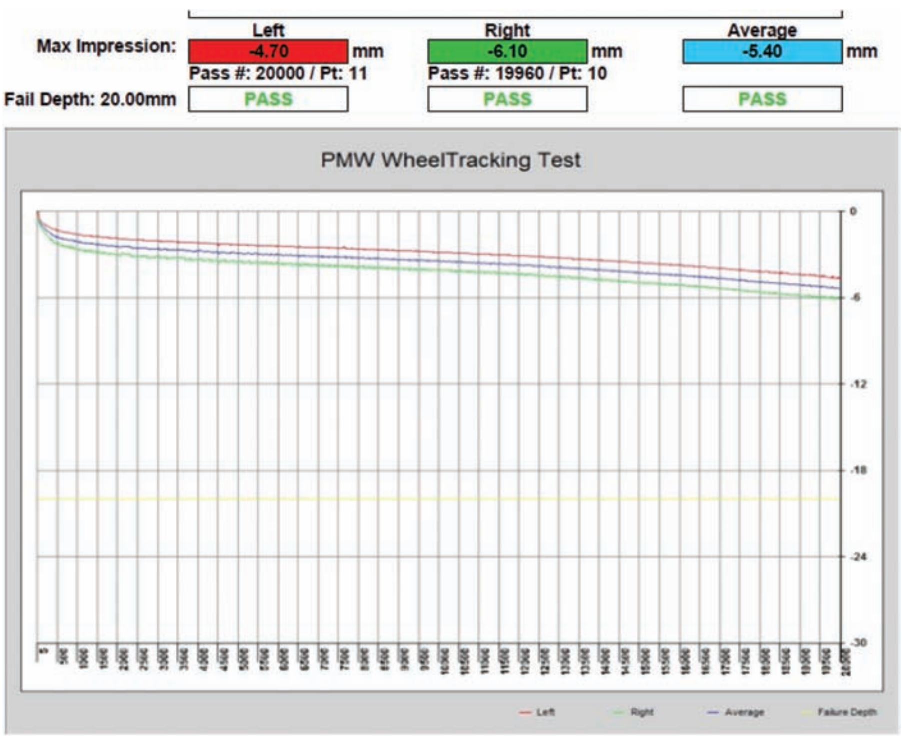

Figure 4.21 Hamburg test results, re-designed mixture cores.

To decide if any statistically significant differences exist between the |E*| values of the two mixtures, a t-Test was used to compare the means. Table 4.42 is a summary of the |E*| data. In this case the |E*| values are higher for the re-designed mixture, however the statistical analysis indicates that at a 95 percent confidence level (α=0.05), the |E*| values of the standard and re-designed mixtures are not significantly different as shown in Table 4.43.

4.5.2.2.3 Semi-Circular Bend Testing. The nine specimens were taken from field cores by cutting the cores in half. Before testing the air void content was determined for each specimen as shown in Table 4.44. Figure 4.26 shows the change in fracture energy with notch depth for the mixtures and aging conditions. Table 4.45 contains the calculated J_c for all mixtures. Again, the higher the J_c value, the more resistant mixtures are to cracking and to crack propagation. The results indicate that in the unaged condition (post-construction) the re-designed mixture has a slightly lower J_c value than the standard mixture; after oven conditioning to simulate eight years of in-service life the

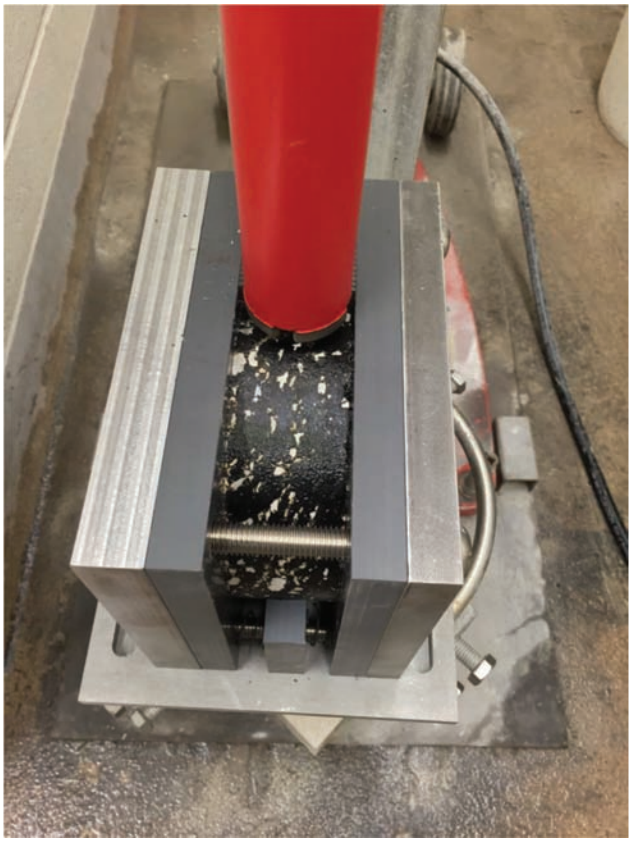

Figure 4.22 Coring horizontally from road cores.

Figure 4.23 Two small specimens (38-mm x 110-mm) taken from road cores.

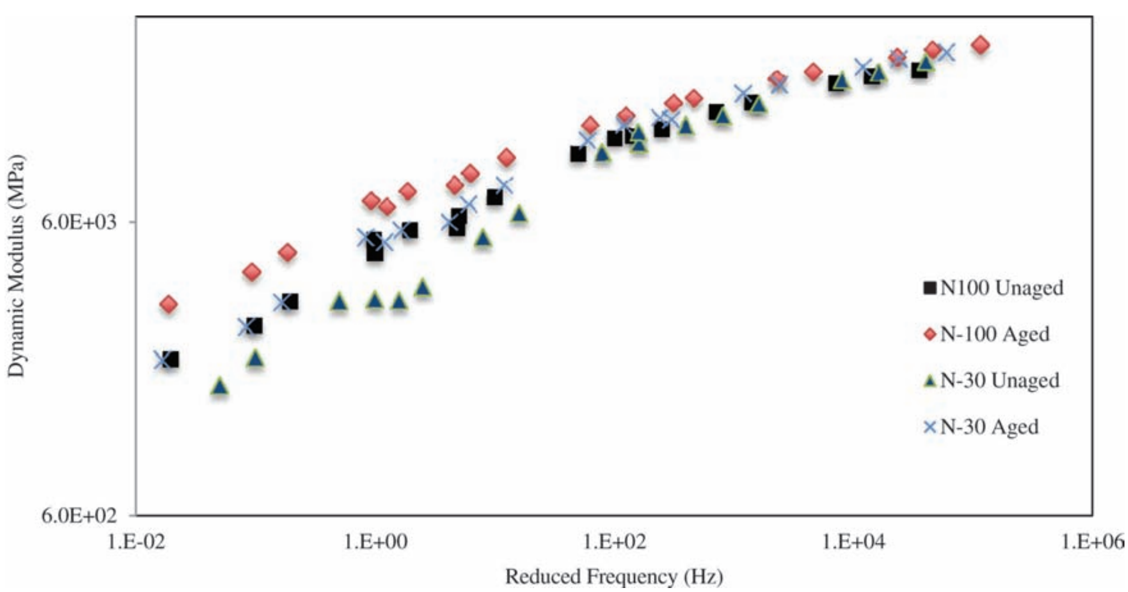

Figure 4.24 Master curves, field cores (log-log plot).

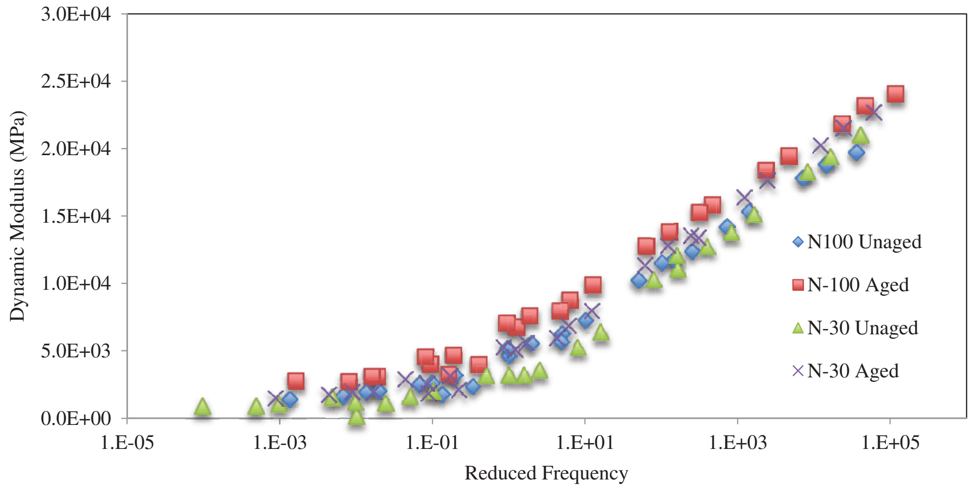

Figure 4.25 Master curves, field cores (semi-log plot).

TABLE 4.42
Dynamic modulus summary, field cores.

	N100 Unaged		N100 Aged		N30 Unaged		N30 Aged	
25 Hz	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
4°C	18236	9.1	19779	7.7	20326	4.3	20603	2.4
21°C	11488	4.7	13129	6.4	12333	4.7	13280	5.7
37°C	6205	3.2	7674	10.3	7130	11.3	8276	6.1
10 Hz	**N30**		**N50**		**N100**			
	Average |E*|, MPa	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**	**Average |E*|, MPa**	**C.V., %**
4°C	17421	9.3	18422	6.4	18789	5.4	19396	3.6
21°C	10834	3.5	11774	5.1	11594	6.0	12297	3.9
37°C	5533	8.6	6778	9.7	6134	8.2	7209	6.1

TABLE 4.43
Dynamic modulus, field cores Bonferroni groupings.

Test Frequency	**25 Hz**		**10 Hz**	
Test Temperature	**p-value (α = 0.05)**	**Grouping**	**p-value (α = 0.05)**	**Grouping**
4°C	0.00097	(N30A=N30U=N100A); (N100U);	0.00577	(N30U=N100U); (N30A=N100A)
21°C	0.00113	(N30A=N100A=N30U); (N100U)	0.0237	(N30A=N100A=N30U); (N100U)
37°C	0.000001	(N30A); (N100A=N30U); (N100U)	0.000003	(N30A=N100A=N30U); (N100U)

re-designed mixture shows a significantly higher J_c value than the standard mixture.

4.5.2.2.4 Binder Recovery and Grading. The asphalt binder from field cores was recovered and graded in both the unaged (post-construction) and long-term aged (eight-year in-service life) conditions. The binders were recovered using AASHTO T 319 and tested to determine its performance grading according to AASHTO M320. The continuous high and low grades for the recovered binders are shown in Table 4.46.

The data in Table 4.46 indicate the original binder high temperature performance grades increased from 58 to 87 and 80 for the post-construction standard and re-designed mixtures, respectively. The low temperature performance grades increased from -28 to -26 for the post-construction standard mixture, while there was no

TABLE 4.44
Georgetown Road field semi-circular AV% data.

N100				N30			
Cores No.	G_{mb}	G_{mm}	AV%	Cores No.	G_{mb}	G_{mm}	AV%
N100-11	2.391	2.509	4.7	N30-411	2.408	2.502	3.8
N100-12	2.383		5.0	N30-412	2.401		4.0
N100-41	2.379		5.2	N30-421	2.410		3.7
N100-42	2.370		5.5	N30-422	2.406		3.8
N100-51	2.378		5.2	N30-431	2.407		3.8
N100-52	2.372		5.4	N30-432	2.411		3.6
N100-61	2.363		5.8	N30-441	2.407		3.8
N100-62	2.367		5.6	N30-442	2.402		4.0
N100-71	2.362		5.9	N30-461	2.393		4.3
N100-72	2.362		5.9	N30-462	2.386		4.6
N100-81	2.363		5.8	N30-471	2.374		5.1
N100-82	2.369		5.6	N30-472	2.376		5.1
N100-91	2.352		6.2	N30-481	2.375		5.1
N100-92	2.372		5.5	N30-482	2.367		5.4
N100-181	2.334		7.0	N30-521	2.389		4.5
N100-182	2.338		6.8	N30-522	2.386		4.6
N100-191	2.325		7.3	N30-531	2.393		4.4
N100-192	2.331		7.1	N30-532	2.397		4.2
N100-201	2.360		5.9	N30-541	2.387		4.6
N100-202	2.366		5.7	N30-542	2.391		4.4
Average AV%		5.9		Average AV%		4.3	
SD AV%		0.71		SD AV%		0.53	

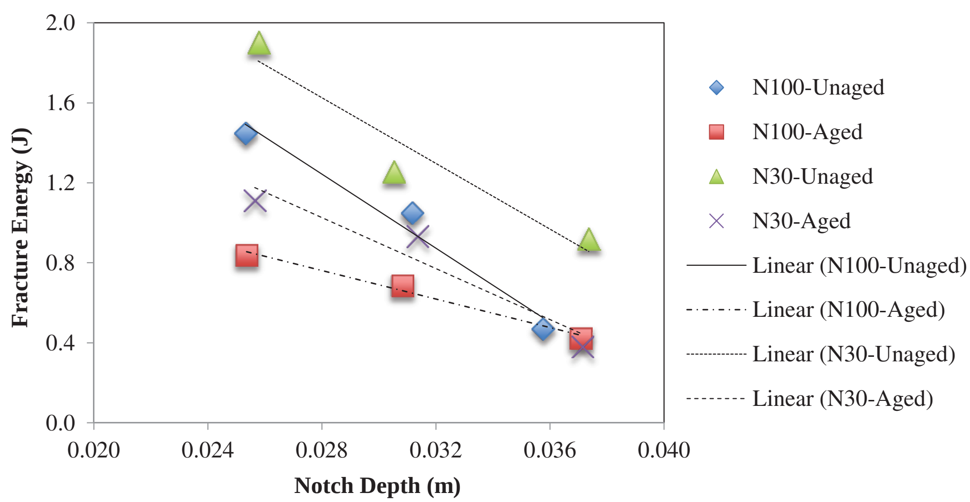

Figure 4.26 Georgetown Road field cores, SCB test results.

TABLE 4.45
Georgetown Road field cores, critical strain energy release rate.

Mixture	Aging Condition, years	J_c (J/m^2)
N100	0	1.627
N30		1.424
N100	8	0.624
N30		1.103

TABLE 4.46
Georgetown Road field cores, recovered asphalt binder grades.

Mixture	Aging Condition, years	Original PG Grade	Recovered PG Grade
Standard	0	PG 58-28	PG 87-26
	8		PG 93-22
Re-designed	0	PG 58-28	PG 80-28
	8		PG 85-24

change in the low temperature performance grade for the post-construction re-designed mixture.

After simulating eight years of in-service life with oven conditioning, the high temperature performance grade for the standard mixture increased from 58 to 93, while the re-designed mixture increased to only 85. For the low temperature performance grade, the standard mixture increased from -28 to -22 and the re-designed mixture increased to -24. The results seem to indicate the standard mixture did in fact "age" more (binder became stiffer) when compared to the re-designed mixture, for both aging conditions. The re-designed mixture may therefore age less in the field, resulting in better mixture and pavement durability.

5. SUMMARY AND CONCLUSIONS

The objective of the research was to optimize asphalt mixture laboratory design compaction as it relates to field compaction in order to increase asphalt pavement durability without sacrificing the permanent deformation characteristics of the mixtures. INDOT's current asphalt mixture design method specifies a design air voids content of 4 percent. Asphalt mixtures thus designed are typically placed with 7 percent air voids or higher. This can result in lower than desired asphalt pavement service lives due to durability loss as the asphalt prematurely ages.

Compacting asphalt pavements to 5 percent in-place air voids (95 percent G_{mm} density), without the possibility of further densification from traffic would make them more durable thus extending asphalt pavement life. However, producing asphalt mixture laboratory designs by choosing optimum asphalt binder content at 4 percent and then attempting to compact such mixtures to 5 percent air voids in the field makes no more sense than the current method of designing at 4 percent air voids and compacting to 7 percent air voids in the field. Instead, the hypothesis of this research based on the precedent provided by the LCPC mix design approach, was that asphalt mixtures should be designed in the laboratory at 5 percent air voids and then compacted in the field to the same air voids.

Given this hypothesis, the research investigated the possibility of re-designing standard asphalt mixtures by varying the aggregate gradation, holding the effective binder content constant, and compacting mixture design test specimens to 5 percent air voids. Keeping the effective binder content constant for each mixture type ensures that mixture durability will not be compromised by removing asphalt binder from the mixtures. In doing so the major concern becomes possible changes to the permanent deformation characteristics of the mixtures. For this reason, the |E*| and FN values for the mixtures were examined to see if the re-designed mixtures could be expected to have |E*| and FN values as high as or higher than the standard mixtures. Results indicated that overall the re-designed mixtures did have |E*| and FN values as high as or higher than the standard mixtures.

Finally, two field trials were placed to compare re-designed mixtures to conventionally designed mixtures. The re-designed mixtures were placed in the field at approximately 5 percent air voids without the necessity of changing roller types or patterns. No extra effort was required to achieve the target field density (air voids). Post-construction testing results for the standard and re-designed mixtures from the two field projects were varied, but overall it is expected that the re-designed mixtures should perform as well as, if not better than, the standard mixtures.

5.1 Conclusions

The findings from this study lead to the following conclusions:

- It is possible to design asphalt mixtures with 5 percent air voids without lowering the effective binder content; this can be accomplished by varying the aggregate gradation of the mixture.
- Asphalt mixtures designed using lower compaction levels than currently specified by the Superpave method can be designed to have |E*| and FN values as high as, or higher than, conventionally designed mixtures, without lowering the effective binder content.
- On average, the mixture data appeared to indicate that 53, 52, and 42 were the optimum numbers of gyrations for the three re-designed mixtures developed in the laboratory.
- Asphalt mixtures designed in the laboratory at 5 percent air voids can be compacted to 5 percent air voids in the field. The field test indicates this can be done without additional compaction effort.
- Results of testing re-heated, plant-produced mixtures are somewhat variable, as indicated in the post-trial test results. Some test results seem to indicate the standard mixtures may perform better, while other results favor the re-designed mixtures. The performance of the test sections will help confirm if the re-designed mixtures are, or are not better performers than the standard mixtures.

6. RECOMMENDATIONS FOR IMPLEMENTATION

While the findings of this study are encouraging, the amount of testing was limited, making it difficult to recommend the exact manner in which INDOT should proceed. The following recommendations are offered as a starting point:

- For medium to high traffic levels, 50 gyrations appear to be the correct number of gyrations to be used in designing asphalt mixtures with optimum binder content chosen at 5 percent air voids.
- For lower volume roads, 30 gyration mixture designs might be more appropriate. Further laboratory and field testing should be conducted to verify this number.
- Low-temperature and moisture susceptibility testing, should be completed for both standard and re-designed mixtures, to help quantify the enhanced durability of the re-designed mixtures.
- Additional laboratory work should be done so as to include the effects of more traffic levels; mixtures

containing RAP, RAS, or both; and additional binder grades and aggregate types.

- The performance of the trial projects placed as part of this study should be monitored for performance.
- Additional trial projects should be placed and evaluated.

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