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Effect of Compaction Effort on SuperPave Base Course Materials

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Thesis submitted to the College of Engineering and Mineral Resources at
West Virginia University in partial fulfillment of the requirements for the
degree of

Master of Science

In

Civil Engineering

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ABSTRACT

Effect of Compaction Effort on SuperPave Base Course Materials

Cornelius Adamah

SuperPave mixes have performed well with in West Virginia, especially with respects to permanent deformation. Recent studies raise concerns with premature fatigue cracking and difficulty in compaction during construction. Two base course mixes were tested by lowering compaction effort from 100 gyrations to 80 gyrations for the 19mm mix and to 65 gyrations for the 37.5 mm mix and evaluating rutting potential with Asphalt Pavement Analyzer and Indirect Tension test. Only gyration level and binder percent were changed. Lower compaction effort resulted in an increased binder content. The 19mm mixes showed an increased rutting potential. However, investigation of the data demonstrated position of the sample in the APA machine confounded the results of the test. No statistically significant rutting potential was found with the lowered compaction effort or increased binder content for the 37.5 mm mix.

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CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Hot mix asphalt (HMA) is the most widely used pavement surface in the United States. Approximately 96% of the 2.3 million miles of roadways in the United States have HMA surfaces (Roberts, et al, 1996). HMA consists of coarse and fine aggregates, asphalt, air, and occasionally some additives to improve the engineering properties. The proportions of these materials are determined by a mix design process. The SuperPave mix design method, developed during the Strategic Highway Research Program, has been adopted by all state highway agencies in the U.S (Kuennen, 2003).

The design binder content of SuperPave mixes is determined based on the volumetric properties of samples prepared at different asphalt contents. The SuperPave Gyratory Compactor (SGC) is used to prepare the HMA samples. This compactor was selected because it can produce specimens with densities similar to field-compacted HMA (Cominsky, 1994). The density achieved for a given combination of aggregates and asphalt is a function of the compaction effort applied by the SGC. Compaction effort is a composite term that embraces the vertical force, the tilt angle, and the number of gyrations applied to the sample. While conceptually it would be possible to alter the compaction effort by altering any of these three parameters, the standard practice is to keep the vertical force and tilt angle constant and alter the number of gyrations to achieve a desired compaction effort.

The compaction effort used initially for the SuperPave method was established through evaluation of mixes, which preformed well (Roberts et al, 2002). Initially, 28 levels of compaction effort were specified to accommodate seven traffic levels and four climatic conditions (Cominsky, 1994). However, since the inception of the implementation phase of SuperPave, there have been multiple research projects at both the national and state level that have examined the compaction effort, or number of gyrations, used for determining the design binder content (Brown and Buchanan, 1999). Most recently, the National Center for Asphalt Technology (NCAT) has completed the National Highway Research Program (NCHRP) Project 9-9(1), *SuperPave Mix Design: Verifying Gyration Levels in the N_{design} Table*; it concluded that the number of gyrations

used for determining the design binder content should be reduced compared to the current compaction requirements specified in AASHTO Test Method R35.

1.2 PROBLEM STATEMENT

The West Virginia Division of Highways (WVDOH) has been constructing pavements with HMAC designed with the SuperPave method of pavement design for several years. In general, these pavements have performed very well and none have been observed to display problems with rutting (Zaniewski and Patino, 2005). The WVDOH commonly adopts nationally developed methods without modification. If the recommendations of NCHRP Project 9-9(1) are adopted at the national level, then the WVDOH will need to make a decision to either continue with the existing SuperPave compaction effort, which has worked well, or adapt the revised compaction effort. Since the revised compaction effort should reduce in mixes with a higher asphalt content, for a given blend of aggregates, mixes designed using the revised compaction effort should improve the performance of the mixes with respect to fatigue and thermal cracking, but this may come at the expense of increased rutting potential.

1.3 OBJECTIVE

The objective of this research was to determine if mixes designed using the compaction effort recommended by NCHRP 9-9(1) (Prowell and Brown, 2007) have adequate performance with respect to rutting susceptibility as evaluated with the Asphalt Pavement Analyzer, APA, and Indirect Tension test(IDT).

1.4 SCOPE AND LIMITATIONS

The WVDOH predominantly uses four SuperPave mix types; 9.5 and 12.5 mm mixes for surface or wearing courses, and 19 and 37.5 mm mixes for base courses. The characteristics of the wearing courses were evaluated previously (Hornbeck, 2008). Thus, this study evaluated the effects of lowering compaction effort has on 19 and 37.5 mm mixes.

Mix design data were obtained from a contractor for both 19 mm and 37.5 mm mixes. Samples of the aggregate and binder used in these mixes were obtained by the contractor. The percent binder of these mixes was selected using the current

requirements for compaction effort. The compaction effort from the recommendations of Prowell and Brown (2007) were used to determine a new optimum binder content. Samples of both the original and redesigned mix were then tested using the Asphalt Pavement Analyzer and Indirect Tensile Test. No attempt was made to optimize the blend of the aggregates for the new compaction effort. This task was left to another research effort.

1.5 ORGANIZATION OF THESIS

This thesis contains five chapters. The first chapter is the introduction to the research. Chapter 2 is the literature review with changes in the N_{design} table, state research on compaction effort, and a summary of the NCHRP Report #573. Chapter 3 contains the research methodology used for this research. Chapter 4 contains the analysis and results. Conclusions and recommendations are presented in Chapter 5. Laboratory data shown in Appendix A and Appendix B represents the statistical analysis.

CHAPTER 2: LITERATURE REVIEW

2.1: INTRODUCTION

Since the 1940s and 1950s, the Hveem and Marshall Mix design methods have been used for road construction. For many years, these mix design methods performed well but with increased traffic volumes, tire pressure, and heavier loads, it was determined in the early 1980s that an improved method of mix design was needed (Roberts et al, 2002). A method was needed that could be used to rationally design mixtures for various traffic volumes, axle loads and the environment. From October 1987 through March 1993, the Strategic Highway Research Program (SHRP) conducted a federally funded, \$50 million research effort to help develop new ways to specify, design, and test asphalt materials. The research was to help provide a system of checks and balances to ensure that Hot Mix Asphalt (HMA) is durable and rut resistant. It was believed that this research would provide tests and models that would result in better asphalt pavement performance.

SHRP primarily focused on binder specifications, aggregate property specifications, design gradation ranges, a laboratory compaction procedure, specifications for volumetric properties, and an evaluation of moisture sensitivity (Roberts, et al, 2002). Due to the relatively condensed time period available for the development of SuperPave, it was anticipated that ongoing effort would be needed to refine the SuperPave method. One of SHRP's key study areas, and the subject of this research, was the compaction effort used for preparing samples for the volumetric analysis. With all other factors being equal, the design binder content determined in mix design method is a function of this compaction effort.

2.2. SUPERPAVE MIX DESIGN METHOD

2.2.1 Development of SuperPave

Figure 2.1 is an overview of the SuperPave mix design method. Once suitable binder and aggregates are selected, a volumetric procedure is used to evaluate the suitability of a design aggregate structure (DAS). The design binder content (DBC), is

Figure 2.1 Overview of SuperPave mix Design Method

then determined for the selected DAS, again using volumetric analysis. Finally the moisture sensitivity is evaluated for a mix prepared using the DAS and the DBC.

The keys to selection of both the DAS and the DBC are volumetric properties of the mix. Volumetric properties are a function of the specific gravity of the binder, the aggregate blend, and the maximum theoretical specific gravity and bulk specific gravity of the mix. The bulk specific gravity of the mix is directly a function of the energy used to compact the mix.

2.2.2 SuperPave Compaction Effort

Due to the importance of the compaction procedure on the properties of SuperPave mixes a great deal effort was placed on the selection of the compaction equipment and the operational parameters used to control the compaction effort. The SHRP researchers concluded that samples prepared using a gyratory shear method produced samples with suitable characteristics and good repeatability. Machines designed specifically to meet the compaction requirements specified by the SHRP researchers became known as SuperPave gyratory compactors (SGC).

The principles of the SGC are shown in Figure 2.2. There are three parameters which control the compaction effort placed on a sample; the vertical force or pressure, the tilt angle, and the number of revolutions applied to the sample. The SHRP researchers recommended fixing the vertical pressure at 600 kPa and the tilt angle to 1.25° measured external to the sample mold (Brown et al, 2001). The tilt angle was subsequently refined to 1.16° measured internally (Prowell et al, 2003). The easiest parameter to vary in order to change the compaction effort is the number of gyrations or revolutions applied to the sample.

One of the specifications required for a SGC is the ability of the machine to measure the height of the sample with each gyration. The SHRP researchers identified that the height information could be used to indicate the compatibility characteristics of a mix that could be related to the characteristics of the mix in the field. They identified three instances with respect to field characteristics related to the compaction characteristics of a mix.

Figure 2.2 SuperPave Gyratory Compactor

- 1) During construction, a mix needs a minimum amount of internal resistance to allow the mix to densify without being displaced by the rollers.
- 2) At the end of construction, or shortly thereafter, the mix should have an air void content similar to the air void content used in the mix design process.
- 3) After a “long” time period the mix should retain a minimum air void content to reduce the likelihood of rutting and bleeding problems.

The SHRP researchers coined the terminology N_{initial} , N_{design} , and N_{maximum} to coincide with the three field characteristics respectively.

Initially, mixes were always compacted using N_{maximum} gyrations and volumetric properties were estimated for N_{initial} and N_{design} . However, this process lacked precision

for determining the volumetric properties at N_{design} . Since the volumetric properties are critical at N_{design} , the compaction process was altered to use N_{design} or the compaction effort for preparing the samples used for determining DAS and DBC.

Once the methodology and concept for the compaction process were established, it was necessary to associate a specific compaction effort with different design environments. Conceptually, the design of rut resistant mixes should consider both the anticipated traffic levels on the pavement and the expected maximum pavement temperature. In general, mixes with low asphalt contents are more rut resistant than mixes with higher asphalt contents. With respect to mix design, this concept translates into using higher compaction efforts for mixes subjected to high traffic volumes and elevated temperatures. Using this logic, the SHRP researchers defined compaction levels for four temperature ranges and seven traffic classifications as shown in Table 2.1.

Table 2.1 Compaction levels for four temperature and seven traffic classifications

Design ESALs (millions)	AVERAGE DESIGN HIGH AIR TEMPERATURE											
	< 39 °C			39 - 40 °C			41 - 42 °C			43 - 44 °C		
	Ni	Nd	Nm	Ni	Nd	Nm	Ni	Nd	Nm	Ni	Nd	Nm
< 0.3	7	68	104	7	74	114	7	78	121	7	82	127
< 1	7	76	117	7	83	129	7	88	138	8	93	146
< 3	7	86	134	8	95	150	8	100	158	8	105	167
< 10	8	96	152	8	106	169	8	113	181	9	119	192
< 30	8	109	174	9	121	195	9	128	208	9	135	220
< 100	9	126	204	9	139	228	9	146	240	10	153	253
< 100	9	143	233	10	158	262	10	165	275	10	172	288

2.2.3 Current SuperPave Compaction Effort

Experience demonstrated the number of combinations of traffic and temperature were excessive and the compaction effort was excessive. Following the SHRP research program an Expert Task Group (ETG), was established to evaluate the results and recommendation of the research. The SuperPave ETG identified several assumptions and limitations of the SHRP research on compaction during mix design (ETG 1995 meeting as documented by Brown and Mallick, 1998). The NCHRP awarded Project 9-9 to the National Center for Asphalt Technology, NCAT, to evaluate the compaction effort used

for mix design. As the result of Project 9-9, the AASHTO SuperPave mix design procedure was revised to use the compaction levels shown in Table 2.2.

Table 2.2 Consolidated SGC Compaction Table with Four Levels of Traffic

Design ESALs (millions)	Compaction Parameter		
	N _{initial}	N _{design}	N _{maximum}
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 30	8	100	160
≥ 30	9	125	205

The requirements in Table 2.2 are considerably lower than the recommendations in Table 2.1. The maximum N_{design} in Table 2.1 is 172 gyrations whereas in Table 2.2 the maximum is 125 gyrations. The West Virginia DOH requires the compaction effort defined in Table 2.2, with the following note:

***Note:** For design traffic levels of ≥ 3 million ESALs, specifications or contract documents may require the Job Mix Formula (JMF) gyratory compaction criteria for mixtures located ≥ 4 inches (100 mm) below the pavement surface be lowered by one ESAL level. Also, if a different binder grade is required in mixtures located ≥ 4 inches (100 mm) below the pavement surface, this information will be provided in the specifications or in the contract documents. If less than 25% of the mixture layer is within 4 inches (100 mm) of the surface, the layer shall be considered to be below 4 inches (100 mm) for design purposes.*

2.2.4 AASHTO SuperPave Requirements

The SuperPave volumetric mix design procedure (AASHTO MP2 and M323) was developed from the SHRP. When the method was first developed by the SHRP researchers, it was known as AASHTO PP28, based on the intention of having three levels of design. Level one design consisted of materials selection and volumetric proportioning and was designated for low-traffic roads having less than 1 million ESALs. Levels two and three were designed for increasingly heavier traffic with level three pavements being defined by SHRP as those experiencing over 10 million ESALs. Level two pavements were designed for loads between 1 to 10 million ESALs and level three was for roads with more than 10 million ESALs. The concept of design levels was never

implemented. All SuperPave mixes are designed using the volumetric requirements originally developed for level one (Roads & Bridges, 1996).

The detailed Specification for SuperPave Volumetric Design can be found in AASTO M323-04 and the description of the mix design procedure can be found in AASTO R35-04 (AASHTO, 2004(a)). AASHTO SuperPave mix design must meet all the following requirements:

- a) The asphalt mixture must have a target air voids at 4% when compacted to N_{design} gyrations.
- b) The Voids in Mineral Aggregate (VMA) of the compacted mixture at N_{design} gyrations must meet the minimum VMA requirements shown in Table 2.3.
- c) Voids Filled with Asphalt (VFA) of the compacted mixture at N_{design} gyrations must fall within the range shown in Table 2.4.
- d) Ratio of the weight of the mineral filler to the weight of the effective binder, dust-to-binder ratio, must be in the range of 0.6 and 1.2. Agencies can elect to use a dust to binder ratio of 0.8 to 1.6 for coarse graded mixes.
- e) The $\%G_{\text{mm}}$ of the asphalt mixture compacted to N_{initial} must not exceed the limits as shown in Table 2.3.
- f) The $\%G_{\text{mm}}$ of the mixture compacted to N_{maximum} must not exceed 98%.

The asphalt binder should be a PG grade meeting the requirements of AASHTO MP1a-04.

Table 2.3 SuperPave Criteria on %G_{mm}, VMA, VFA and Dust-to-Binder Ratio

Design Traffic (million ESALs)	VFA (%)	Required %Gmm			Required minimum VMA (%)					Dust-to-Binder Ratio
		N _{initial}	N _{design}	N _{max}	Nominal Max. Agg. Size, mm					
					37.5	25.0	19.0	12.5	9.5	
< 0.3	70 - 80	≤ 91.5	96.0	≤ 98.0	11.0	12.0	13.0	14.0	15.0	0.6 - 1.2
0.3 to < 3	65 - 78	≤ 90.5								
3 to < 10	65 - 75	≤ 89.0								
10 to < 30										
> 30										

The binder grade selected for a project is determined based on the expected pavement temperature range at the project location. The binder upper temperature grade is increased one level for either heavy truck traffic volume or for slow moving traffic. If both heavy truck traffic and slow speeds are expected the binder grade is increased by two levels for the upper temperature. Source properties such as L.A abrasion, soundness, and deleterious materials requirements must be met according to local highway agency specifications. The consensus aggregate properties are shown in Table 2.4 and the gradation classifications are shown in Table 2.5.

Table 2.4 SuperPave Consensus Aggregate Properties

Design Traffic (million ESALs)	Coarse Aggregate Angularity, Minimum (% with one fractured face/% with two fractured faces)		Uncompacted Void Content of Fine Aggregate, Minimum (%)		Sand Equivalent, Minimum (%)	Flat and Elongated, Maximum (%)
	Thickness ≤ 100 mm	Thickness > 100 mm	Thickness ≤ 100 mm	Thickness > 100 mm		
< 0.3	55/-	—	—	—	40	—
0.3 to < 3	75/-	50/-	40	40	40	10
3 to < 10	85/80	60/-	45	40	45	
10 to < 30	95/90	80/75	45	40	45	
≥ 30	100/100	100/100	45	45	50	

Table 2.5 SuperPave Gradation Classifications

SuperPave Gradation Requirements						
Standard Sieve (mm)	Percent Passing Criteria (Control Points)					
	Nominal Maximum Sieve Size					
	4.75	9.5 mm	12.5 mm	19 mm	25 mm	37.5 mm
50						100
37.5					100	90-100
25				100	90-100	90.0 max.
19			100	90-100	90.0 max.	
12.5	100	100	90-100	90.0 max.		
9.5	95-100	90-100	90.0 max.			47
4.75	90-100	90.0 max.		47	40	
2.36		32-67	28-58	23-49	19-45	15-41
		47	39			
1.18	30-60					
0.075	6-12	2.0-10.0	2.0-10.0	2.0-8.0	1.0-7.0	0.0-6.0
Gradations which fall below the primary control sieve are classified as coarse gradations						

2.3 STATE RESEARCH ON COMPACTION EFFORT

The Colorado Department of Transportation (CDOT) conducted a research to collect and validate data regarding the number of N_{design} gyrations used to select the optimum asphalt and recommend a new set of values for the number of gyrations. Research objectives were focused on the determination of laboratory compactive effort to ensure that the proper binder content was being used in mix designs (Harmelink and Aschenbrener, 2002).

The research examined the densification of 25 evaluation selections selected from 22 projects selected to represent the range of environmental conditions and traffic levels in the state. Mixes were designed with the Texas gyratory compactor, but it was verified that the mixes met the SuperPave mix criteria. The researcher found that, in general, pavement density increased for three years following construction then the pavements did not densify any further. The air voids three years after construction were higher than derived leading the researchers to conclude that the mixes were too stiff. The researchers conclude that the problem was associated with the asphalt content being too low and recommended either lowering the number of gyrations used for mix design or adjusting

the target air voids during mix design. The researchers did not provide specific values for the changes in their parameters.

Brown (2005) summarized the experience of several states with respect to the asphalt content versus compaction level of SuperPave mixes. Based on the work of Harmelink and Aschenbrener (2002), Colorado mixes are designed at four percent air voids, but the state may allow the production target air void to be three to allow for a higher asphalt content to three percent to increase binder content. The Maryland Asphalt Association encourages local agencies to use N_{design} of 50 gyrations as a means to have higher binder contents. Virginia DOT uses a single compaction effort of 65 gyrations for all traffic levels but increases the upper temperature binder grade to provide mix stiffness.

The most common approach to increasing asphalt content is to reduce the compaction effort Brown (2005) quotes Huber, associate director of research at Heritage Research Group.. Huber asserts this is flawed logic and the control limits for the voids in the mineral aggregates should be analyzed. If VMA is not changed, the aggregates can be altered such that the lower compactive effort does not necessarily increase the binder content used in mixes.

2.4. NCHRP PROJECT 9-9(1)

As documented by Brown and Mallick (1998), the data set used to develop Table 2.2 was very limited.

As documented by Brown (2005) states were taking the initiative to alter the AASHTO compaction specifications. The NCHRP funded Project 9-9(1) to verify the compaction effort on a nationwide basis (Prowell and Brown, 2007). Projects with a performance history of four years were evaluated across 16 states. More than 4,000 SGC compacted samples and over 5,600 cores were examined during the study.

Figure 2.3 is a cumulative distribution curve for the projects (Prowell and Brown, 2007). It was found that 55 percent of the projects had an initial density following construction of less than 92 percent of G_{mm} . This low level of density, which corresponds to 8 percent air voids, indicates the pavements are permeable (Roberts, et al,

Figure 2.3 Cumulative frequency distribution of as-constructed, in-place density

1996) and also air void contents in excess of 8 percent have interconnected passages which permit rapid water damage and oxidation of the asphalt.

While there are several factors which can contribute to low field densities, a low mix design asphalt content would certainly contribute to the problem.

The densification of the asphalt following construction was examined producing the cumulative distribution curve shown in Figure 2.4 (Prowell and Brown, 2007). The majority of the asphalt concrete densification occurred during the first three months following construction. The rate of densification decreases with time and after two years little additional densification occurs.

As would be expected on a project of this magnitude, many factors affecting compaction were evaluated including region of country, binder grade, construction season, etc. Although the researchers encountered a great deal of variability in the samples, they did recommend altering the SuperPave gyrations requirements to the values in Table 2.6.

Figure 2.4 Cumulative frequency plot for in-place density by sampling period

The researchers recommend elimination of the N_{initial} and N_{maximum} requirements as these were not reliable indicators of field performance. The researchers included compaction requirements for 2-year design traffic, but retained the 20 year design traffic levels from the current SuperPave mix design method. The recommended gyrations for SuperPave mixes in the top 100 mm, constructed with PG 76 – xx binders, are lower than the current requirements. For designs with a “stiff” binder, $\geq$ PG 76 – xx, or for mixes more than 100 mm from the surface, the researchers recommended a further reduction in the compaction effort.

Table 2.6 Recommended N_{design} Levels from NCHRP Project 9-9(1)

20-Year Design Traffic, ESALs	2-Year Design Traffic, ESALs	N_{design} for binders < PG 76- XX	N_{design} for binders $\geq$ PG 76 - XX or mixes placed > 100 mm from surface
< 300,000	< 30,000	50	NA
300,000 to 3,000,000	30,000 to 230, 000	65	50
3,000,000 to 10,000,000	230,000 to 925,000	80	65
10,000,000 to 30,000,000	925,000 to 2,500,000	80	65
> 30,000,000	> 2,500,000	100	80

2.5: METHOS FOR EVALUATING RUTTING PERFORMANCE

The current SuperPave mix design criteria partially address rutting performance of HMA mixes through gradations control points and consensus aggregate specifications. With aggregate being the main load carrying component of the mix, the aggregate properties are highly critical in providing the resistance to permanent deformation at higher temperatures. There are currently no mechanical testing procedures that pertain to rutting. Performance related tests are used to develop different models which are also used to predict the rutting potential. The performance test includes diametrical tests, uniaxial tests, triaxial tests, shear tests, empirical tests, and simulative tests. Simulative tests are conducted by using a wheel tracking device that measures quality or performance by applying repeated loads across the surface of the specimen. These wheel tracking devices can also be used to predict fatigue and moisture susceptibility which can be correlated to actual pavement conditions.

The most common form of the wheel device used for testing rutting mixes in the laboratory has been the Georgia loaded wheel tester (GLWT) (Kandhal and Cooley, 2002). The Georgia loaded wheel tester was developed through a research study between the Georgia Department of Transportation (DOT) and the Georgia Institute of Technology (Lai, 1986). This device can be used to test either cylindrical or beam

asphalt concrete specimens. Beam dimensions are generally 125 mm wide, 300 mm long, and 75 mm high (5 in x 12 in x 3 in). Cylindrical specimens are prepared to 150 mm in diameter and 75 mm tall. Both specimens are compacted to either 4 or 7 percent air voids but some success has been accomplished by using a targeted air void as low as 2 percent (Collins, et al, 1996). The test consists of applying a 445-N (100-lb) load onto a pneumatic linear hose pressurized to 690 kPa (100 psi). The load is applied through an aluminum wheel onto the linear hose, which resides on the sample. Test specimens are tracked back and forth under the applied stationary loading for a total of 8,000 loading cycles (one cycle is defined as the backward and forward movement over samples by the wheel device).

Figure 2.5 Georgia Loaded Wheel Tester

Test temperatures are within the typical ranges from 35°C to 60°C (95°F to 140°F). The original work performed by Lai was tested at 35°C (95°F) (Lai, 1986). The rut depth is obtained at the conclusion of the 8,000 cycle and it's calculated by determining the difference in the measurement of the surface prior to testing and after testing. This is achieved by using some type of micrometer (Kandhal and Cooley, 2002).

The GLWT has been used extensively by many researchers and state agencies to evaluate permanent deformation in HMA mixes. The Florida DOT used the GLWT to

rank the rut susceptibility of three different kinds of mixes which resulted in a good relationship between the rankings and the field rut depths (Mohammad, 2006). The demand for research in permanent deformation is very high because it is the earliest premature failure (Brown, et al, 2001). Determining the rutting susceptibility of a mix by analyzing the compaction curves of the SuperPave Gyratory Compactor as stated in previous studies is not a feasible approach due to the different high temperatures and binder grades (Coree and VanDerHorst, 1999).

2.5.1: Asphalt Pavement Analyzer (APA)

The Asphalt Pavement Analyzer is a multifunctional loaded wheel tester used for evaluating permanent deformation. It is a modified version of the Georgia loaded wheel tester. A major advantage or improvement the APA has over the Georgia loaded wheel tester is that samples can be submerged and tested in water. Samples can be either beam specimens or cylindrical specimens. The beam specimens can be either fabricated in the laboratory or sawed from an asphalt concrete core obtained from the field. Laboratory beam specimens are compacted by the rolling compaction method, or the vibratory compaction method. Beam specimens are 125 ± 0.5 mm in width, $300 \text{ mm} \pm 0.5$ mm in length and $75 \text{ mm} \pm 0.5$ mm in height. Cylindrical specimens are $150 \text{ mm} \pm 0.5$ mm in diameter and $75 \text{ mm} \pm 0.5$ mm in height. They can be either cored from an asphalt pavement in the field or compacted in the laboratory by the gyration compaction method or the vibratory compaction method. The testing temperature ranges from 40.6°C to 64°C (105°F to 147°F) and wheel force and hose pressure still remains at 100 lbs and 100 psi respectively. Other studies have also been conducted at slightly above the average seven day pavement temperature (Kandhal and Mallick, 1999) which has been successful by using a wheel load of 533 N (120 lbs) and hose pressure of 830 kPa, (120) psi (Williams and Prowell, 1999). The study composed of a 19 mm SuperPave mix, two aggregate types (sandstone and limestone), and four different asphalt binder types. Kandhal and Cooley (2002) studied several mixes of known rutting potential to better correlate rut susceptibility to mix types. A regression analysis was used conclude that both the cylindrical and beam specimens are acceptable means for determining the rut susceptibility. Moreover, evaluation of a standard rut depth was approached to establish a parameter whether a mix has a rutting potential. Shear and creep tests were performed

to evaluate lab and field rutting to set a representative formula to calculate a standard rut depth. A rut depth of 8.0 mm was determined and was used to evaluate the rutting potential of mixes at higher performance grade temperature (Kandhal and Cooley, 2002).

Figure 2.6 Asphalt Pavement Analyzer

In 2002, Choubane, et al, used APA test results in conjunction with field measurements to establish values within the ranges of 7 mm to 9 mm to suggest the limiting criteria for the 8,000 repetitive load cycle for both beam and cylinder specimens. The test data was only based on three mixes and 3 specimens per mix, therefore, adaptation of the results for performance testing were not necessary since different kinds of mix designs are used at different locations. Choubane, et al, (2002) also evaluated procedures for pass and fail which basically pertains to the comparison of rutting mixes with respect to which ruts more.

The rut depth measured with the APA cannot be directly correlated with field measured rut depths due to the non-mechanistic based measurement. Stress and strain conditions demonstrated by the APA cannot be defined mechanistically but can be correlated to field rut depths to predict rutting as studied by the Minnesota DOT (Skok, et al, 2002). The mission of the study was to establish a critical in-service rut depth

criterion for field measurements in Minnesota by identifying projects throughout the state and correlating laboratory averages and variations in rut depths measured with the APA. A criteria value of 7.0 mm (0.28 in) was established and recommended for 8,000 pass cycles. Each wheel pass was equivalent to 130 ESAL's which represented 1,040,000 ESAL's for the 8,000 cycles. Testing parameters for the APA recommended by the Minnesota Department of Transportation can be found in Table 2.7.

The West Virginia University Asphalt Technology Laboratory Center has been using the APA for several years and has achieved satisfactory results. SuperPave mixes within the state of West Virginia has been evaluated with the APA, and a steady and reliable result has been determined with mixes that rutted in the lab with mixes that rutted in the field. No relationship was established between the APA rutting potential and the actual field performance (Zaniewski and Patino, 2005).

Table 2.7 APA Testing Parameters

Parameter	Specification
Test Temperature	Upper Performance Grade of HMA Mixture being Tested
Environmental Condition	Dry
Specimen Size, mm (in.)	150 (6) diameter, 75 (3) height
Load, N (lb)	445 (100)
Hose Pressure, kPa (psi)	689 (100)
Wheel Speed, m/sec	0.61
Air Voids, %	7 ± 0.5
Number of Test Wheel Load Cycles	8000
Laboratory Compaction Device	SuperPave Gyratory Compactor
Pretest Specimen Conditioning	4 hours @ test Temperature
Number of Seating Cycles	50

2.5.2: Temperature Effect Model

Temperature has an important effect in the rutting depth of APA samples. The 60°F testing temperature was used to remain consistent with previous tests conducted at West Virginia University Asphalt Technology Lab Center. The temperature effects

model was developed by Shami, et al (1997) to compare the difference between samples with different testing temperatures (Zaniewski and Patino, 2005). If any two of the three variables are known, the other variable can be determined.

$$\frac{R}{R_0} = \left(\frac{T}{T_0} \right)^{2.625} \left(\frac{R}{R_0} \right)^{0.276}$$

Where,

R = Predicted rut depth.

R₀ = Reference rut depth obtained from the APA test at the reference conditions T₀, N₀.

T, N = Temperature and load cycles the rut depth is sought.

T₀, N₀ = Reference temperature and load cycles for R₀.

Test results concluded that this model could be used to test samples at different temperatures and load cycles. This technique can be used to compare specimens tested at different test temperatures and parameters. Lesser load cycles can be used to expedite testing procedures (Zaniewski and Patino, 2005).

2.5.3: Indirect Tensile Test

The indirect tensile test (IDT) is a diametrical test use to evaluate the rutting potential of a mix (Roberts, et al, 2002). The IDT in most cases is also used to evaluate thermal cracking susceptibility and creep compliances of asphalt mixtures at low and immediate pavement temperatures. For low temperatures where thermal stresses are more significant, estimates from stiffness that are reasonable can be obtained from the indirect tensile test. Asphalt stiffness increases as temperature is reduced, which results in the asphalt concrete behaving as a linear elastic material. In 2002, The National Cooperative Highway Research Program (NCHRP) initiated numerous projects to evaluate a simple performance test for permanent deformation of asphalt mixtures. NCHRP Project 9-16 validated that the SGC compaction slope indicates a change in aggregate characteristics which was not a representative of stiffness and binder volume. Anderson, et al, (2003) studied this experiment with the use of IDT, volumetric

properties, and SuperPave Gyratory Compactor properties to evaluate the rutting potential of a mix. The findings of the study concluded that the use of compaction slope in conjunction with the IDT strength can be used to provide an indicator of a rutting resistance of an asphalt mix and an increase in IDT strength could potentially lead to decreased rutting depth (Anderson, et al, 2003). However, the testing equipments used by Anderson to perform the experiment was expensive and not available for most the highway agencies, so it was not adaptable.

Christensen and Bonaquist (2004) suggest that the indirect tensile strength test performed at high temperatures is potentially a simple and accurate test for evaluating the rut resistance of asphalt concrete mixtures. The IDT strength test performed by Christensen and Bonaquist (2004) had a loading rate of 3.75 mm/min using a standard Lottman breaking head of 150 mm diameter specimens with a test temperature of 33 °C. Results from the study indicated a significant relationship between cohesion of mix and rutting. A current procedure uses the modified Lottman procedure (AASHTO T283) to evaluate the moisture susceptibility. This procedure is recommended because the testing equipment (Marshall Stabilometer) is equipped with a modified loading head to measure the strength of IDT specimens and readily available to state agencies. Other parameters influenced in testing hot mix asphalt are the test temperature and the load rate. The Marshall Stabilometer uses a load rate of 50 mm/min, where Anderson used a load rate of 3.75 mm/min. Moreover, further testing also indicated that using a load rate of 50 mm/min at a test temperature of 10° C below the 7-day average high pavement temperature provided similar results as tests conducted with a load rate 3.75 mm/min at a test temperature of 33 °C. Failure is more rapid under these conditions due to the larger loading rate and higher temperature and also the test can be completed in seconds, which makes it more efficient (Christensen and Bonaquist, 2004). The test is performed by loading a cylindrical specimen with a single or repeated compressive load which acts parallel to and along the vertical diametric plane (Figure 2.7a). The Loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametric plane, which ultimately causes the specimen to fail by splitting along the vertical diameter (Figure 2.7b).

Figure 2.7 IDT Loading and failure mode

In 2004, Zaniewski and Srinivasan conducted a study with the IDT test at West Virginia University Asphalt Technology Laboratory and noticed a correlation between the IDT strength and the rut depth of samples tested in the APA. From the study, the IDT test supported and provided information on the cohesion of the mix with respect to stiffness of the asphalt binder, but does not evaluate other parameters such as the angle of internal friction of the aggregates (Zaniewski and Srinivasan, 2004). The Marshall Stabilometer was used at a loading rate of 50 mm/min with a modified loading head and a test temperature of 60° C. Nominal Maximum Aggregate Size (NMAS) was not a main effect of the tensile strength and that binder grade was the most important factor by using the analysis of variance (ANOVA) table (Zaniewski and Srinivasan, 2004).

The R squared value for IDT strength versus APA rut depth, and the standard error showed a significant correlation between rut depth and tensile strength in a way that as rut depth increased, IDT strength decreased (Zaniewski and Srinivasan, 2004).

2.6 SUMMARY OF LITERATURE REVIEW

The WVDOH has been constructing HMA pavements with the SuperPave mix design method for several years and these mixes have performed well with regards to rutting (Zaniewski and Patino, 2005). Two base course mixes are currently used by the WVDOH, 19 mm and 37.5 mm NMAAS designed with a 100 gyrations.

The original SuperPave N_{design} table was based on limited data. The consolidation of the N_{design} table was generated based on a laboratory study designed to evaluate the sensitivity of volumetric properties. A study conducted by Prowell and Brown, NCHRP Report 9-9(1) recommended lowering the compaction effort from 100 to 80 gyrations for 19 mm mixes and from 100 gyrations to 65 gyrations for 37.5 mm mixes. Results from the study showed that the compaction effort in the SuperPave N_{design} table were excessive and could be lowered up to 25 gyrations.

Lowering compactive effort should improve the quality and performance of a pavement with respect to cracking but this could lead to permanent deformation in the wheel path due to increased binder content. The literature indicates that there is a need to verify rutting potential with the recommended changes proposed by Prowell and Brown.

CHAPTER3: RESEARCH METHODOLOGY

3.1 INTRODUCTION

This research evaluated what effect the NCHRP 9-9(1) (Prowell and Brown, 2007) would have on the rutting performance of typical 19 mm and 37.5 mm SuperPave mixes used by the WVDOH. Mix design and materials were donated by West Virginia Paving. The mix designs were verified at the West Virginia University Asphalt Technology Laboratory. Using the original aggregate blends, and the compaction efforts recommended by the NCHRP study, a new percent binder was established. Samples of the original and modified mixes were then prepared for testing with the Asphalt Pavement Analyzer and Indirect Tension Test. Finally, an Analysis of Variance, ANOVA, was performed to compare the factors in the experiment.

3.2 MIX DESIGNS

The mixes used for this research were 19 mm and 37.5 mm base mixes. The binder used for all tests was a PG 70 – 22 supplied by Shelly Company, Kanauga, Ohio. The types of aggregates and proportions are shown in Table 3.1. Gradations and specific gravity data are presented in appendix A.

Table 3.1 Blend Properties for 19 mm and 37.5 mm Mixes

19 mm Mix		37.5 mm Mix	
Aggregate Type	Percent of Total Aggregate	Aggregate Type	Percent of Total Aggregate
LS # 67	30%	LS #467	50%
LS #8	23%	LS #67	11%
LS SAND	36%	SLAG SAND	24%
RAP	11%	RAP	15%
FILLER	0.0%	FILLER	0.0%
BLEND	100%	BLEND	100.0%

3.3 AGGREGATE PREPARATION

Material from each stockpile was sieved and the aggregates retained on each sieve were washed to remove fines. The sieve sizes used were 50 mm, 37.5 mm, 25 mm, 19 mm, 12.5mm, 9.5mm, 4.75mm, 2.36 mm, 1.18 mm, 600 μm , 300 μm , and 75 μm . The aggregates were then dried and stored in bins. The fines washed off were discarded and bag house fines were used for the mineral filler. The aggregate from each stockpile and sieve were combined to produce the aggregate blend specified in the job mix formula.

The specific gravities of the aggregates were the specific gravities that were measured by the contractor. The calculations for the combined specific gravities of the blends can be seen in appendix A for the mixes.

3.3.1: AGGREGATE BLENDS

The mix design sheets used for this experiment were provided by West Virginia Paving and can be seen in Appendix A for both 19 mm mix and the 37.5 mm mix. A weigh-out table was generated for each mix design, which shows the mass of each aggregate size that should be used from each stockpile based on the gradations from the contractor. Weight-out tables are in appendix B.

The use of Reclaimed Asphalt Pavement (RAP) and slag made it difficult to produce the weigh-out tables. The RAP in the mix provided a portion of the binder used in the total mix, which had to be taken into consideration. The RAP binder was subtracted from the amount of virgin binder to be added tot the mix. In order for the proper amount of binder percentage to be obtained, the binder from the RAP had to be calculated. In the mix design analysis, 15 percent of RAP means that the RAP accounts for 15 percent of the stone in the mix.

The determination of the amount of binder in the RAP was achieved by calculating the amount of RAP stone to be used and finding the amount of binder the RAP contained. Based on ignitions tests performed in the lab, the percent of binder in the RAP was 5.9 percent. This was done by the contractor and verified in the WVU Asphalt Technology Lab (AASHTO T 308-99.) The NCAT Asphalt Content Tester (Model

F85930, Series 945) was used for this test. The following equations were used to calculate the amount of binder in the RAP.

$$P_s = 100 - P_b \quad (3.1)$$

$$M_s = M_m * P_s \quad (3.2)$$

$$M_{sR} = M_s * P_R \quad (3.3)$$

$$M_R = M_{sR} / (1 - P_{bR}) \quad (3.4)$$

$$M_{bR} = M_R * P_{bR} \quad (3.5)$$

Where,

P_s = Percent stone in mix

P_b = Percent binder in mix

M_s = Mass stone in the mix

M_m = Mass of mix

M_{sR} = Mass of stone from RAP

P_R = Percent RAP in the mix

M_R = Mass of RAP in the mix

P_{bR} = Percent binder in the RAP

M_{bR} = Mass binder from RAP

The total amount of RAP that was weight out for each sample was T_R , which included binder and stone. The quartering method was used to select the RAP material used in each blend.

The amount of virgin binder to add to the mix was calculated using the following equations:

$$M_b = M_m * P_b \quad (3.6)$$

$$M_{bV} = M_b - M_{bR} \quad (3.7)$$

Where,

M_b = Mass of total binder

M_{bv} = Mass of virgin binder

Another difficulty was the use of slag aggregates in the 37.5 mm mix. The absorption of slag is high, due to this property; the dry-back test method was used for the maximum theoretical specific gravity test (AASHTO T 209-99). The dry back procedure requires removing the sample from the water bath after taking the submerged weight and drying the sample on a No. 200 sieve to obtain the dry weight. This dry weight was used in the denominator of the theoretical maximum specific gravity equation.

3.4: TESTING MIX DESIGNS

The weighed-out aggregates, binder, mixing bucket and tools were placed and heated in an oven at the mixing temperature of 165° C. When the materials were at the mixing temperature, the aggregates were poured in the mixing bucket and a desired amount of the heated binder at the same temperature was added. The aggregates and binder were mixed quickly and thoroughly in the bucket until uniform distribution of binder or complete coating of aggregates. The mix was then poured on a table and quartered to select the material for each sample. Two samples were weighed in a pan to create bulk specific gravity pills, G_{mb} , and one maximum theoretical specific gravity sample, G_{mm} . Sample weights obtained for bulk specific gravity pills (G_{mb}) were between 4500 – 4700 grams in accordance to AASHTO T 312-03 and maximum theoretical specific gravity (G_{mm}) was weighed in accordance with AASHTO T 209-99. The samples were cured in the oven at the compaction temperature of 154° C for 2 hrs $\pm$ 5 mins, stirring after one hour. The samples were transferred into molds and compacted in the SuperPave Gyrator Compactor to the desired N_{design} number of gyrations. The SGC used in the lab was an AFGC125X model manufactured by Pine Instrument Company, Grove City, PA. The bulk specific gravity of the pills was determined in accordance with AASHTO T 166 and the theoretical maximum specific gravity was determined in

accordance with AASHTO T 209. From these values the void content and other volumetric properties of the mixes could be calculated.

To verify that the mix design could be produced in the lab to attain the same parameters as the contractor, the mix was produced according to the mix design provided by West Virginia Paving. After the mix design was verified, the new design binder content was found for the lower gyration level. There were no attempts to alternate the blend of aggregates for the new design, only the percent of the virgin binder was optimized.

Four binder contents were selected to find the design binder content to use with the new mix design of 80 gyrations. Two bulk specific gravity pills and one maximum specific gravity sample was made for each binder content. The design binder content was the binder content that resulted in four percent air voids. The design binder content for the 19 mm mix with 80 gyrations was 5.9 percent. The design binder content for the 37.5 mm mix with 65 gyrations was 4.1 percent. The design binder content for the 19 mm mix increased by 0.5 percent and the 37.5 mm mix increased by 0.4 percent.

3.5: PREPARING APA SAMPLES

Asphalt Pavement Analyzer (APA) samples were constructed in the SuperPave Gyrotory Compactor (SGC) to a specified height of 75 mm and air voids between the ranges of 7.0 ± 0.5 . The mass of material per APA sample was based on the maximum specific gravity, G_{mm} , of the sample with considerations with the internal and external voids. The SGC was programmed to compact the sample to the specified height of 75 mm rather than compact to a certain number of gyrations. The mass of material needed to prepare the APA samples were calculated using the following equations (Zaniewski and Patino, 2005):

$$V_{mm} \approx 0.93 * V_c \quad (3.8)$$

$$V_c = \pi * R^2 * H \quad (3.9)$$

$$M_m = V_{mm}/G_{mm} \quad (3.10)$$

Where,

V_{mm} = Volume of the mix

V_c = Gross volume of the sample

R = Radius of the mold

H = Height of the sample

M_m = Mass of the mix

G_{mm} = Theoretical maximum specific gravity of the mix

Three samples were constructed using the original design binder content and six samples were constructed for the new design binder content. Samples were weighed out according to the mix design for each sample so each mix would make three APA samples. Samples were weighed out in groups of three, so one mix would produce three APA samples from the same mix design with the same binder content. The samples were prepared in a randomized fashion and were cooled for one week before they were tested in the APA machine.

3.6: TESTING APA SAMPLES

APA samples were allowed to cool and tested one week after they were compacted in the SuperPave Gyratory Compactor. The APA sample layout is shown in Figure 3.1. Six samples were randomly placed and tested in the mold positions for each run. Each run contained the two types of mixes (19mm and 37.5mm) with three specimens per mix. The distribution of the mixes and their specimens in each run can be seen in Table 3.3. The specimens were conditioned for four hours at a temperature of 60° C. They were all tested at a temperature of 60° C, a hose pressure of 100 ± 5 psi, a wheel load pressure of 100 ± 5 psi, and a cycle count of 8000. The APA machine used in the lab was the Asphalt Pavement Analyzer II manufactured by Pavement Technology, Inc., serial number 97-005.

Rut depths measurements were taken with a digimatic depth gauge. Two measurements were taken, one at the front of the specimen and one at the back of the specimen before each run and after each run. Measurements were to the nearest 0.1 mm.

Figure 3.1 APA Specimen Layouts

Table 3.2 Specimen Order and Location in APA

Run Num.	Specimen Type	Specimen Label	APA Position
1	19 mm 100 Gyration	10	RB
		11	MF
		12	LB
	37.5 mm 80 Gyration	40	LF
		41	MB
		42	RF
2	19 mm 80 Gyration	20	LF
		21	LB
		22	MB
	37.5 mm 80 Gyration	43	RB
		44	MF
		45	RF
3	19 mm 80 Gyration	23	MB
		24	RF
		25	RB
	37.5 mm 100 Gyration	30	LF
		31	LB
		32	MF

The rut depth was calculated by subtracting the initial measurement from the ending measurement. The rut depths from the front and back of the specimen were then averaged together.

3.7 INDIRECT TENSILE STRENGTH TEST

The IDT strength test was performed on the G_{mb} samples originally compacted from each mix design to determine the air voids. The G_{mb} samples were submerged in a water bath with a temperature of 140° F for an hour and 15 minutes and were tested in the Marshall Stabilometer with a modified loading rate of 50 mm/min. The load was applied parallel and along the vertical diametrical plane. The Marshall Stabilometer with a failure plane on a specimen is shown in Figure 3.2 (Zaniewski and Srinivasan, 2004). The force required for the samples to experience failure is plotted on a graph which is shown in Figure B.11 of Appendix B. From the graph, the ultimate force can be obtained which is used to determine the indirect tensile stress at failure by using the equation

$$\sigma_x = \frac{2P}{\pi dt} \quad (3.11)$$

σ_x = Horizontal tensile stress at the center of the specimen, (psi)

P = Maximum load at failure, (lbs)

d = Diameter of sample, (inches)

t = Thickness of sample, (inches)

Figure 3.2 IDT Strength Test Setup with Marshall Stabilometer

The strength of the mix was determined by averaging the indirect tensile stress for two samples designed with single binder content.

3.8 PARAFFIN COATED SAMPLES

The paraffin coated samples were prepared to determine the bulk specific gravity (G_{mb}) of 37.5 mm mixes. The proper measurement of G_{mb} is very critical when using SuPerpave volumetrics. G_{mb} establishes the basis for volumetric calculations used during the HMA mix design. Improper calculations and measurement of G_{mb} could result in an air void target error, voids in the mineral aggregate error, etc. (Hall, et al, 2001).

Test was performed on the 37.5 mm bulk gravity specimens due to the aggregate structure and the orientation of the aggregates. With coarse-graded mixtures, the internal air voids can become interconnected which allows water to infiltrate into the sample rapidly during the saturation procedure. Figure 3.3 illustrates internal air voids of a coarse-graded mix compared with a fine-graded mix.

Figure 3.3 Interconnected air voids of Coarse and Fine Graded mixes

When using the AASHTO T-166 method to determine the G_{mb} , water tends to seep out quickly and this could result in false measurements. If the absorption of water by the specimen exceeds 2%, paraffin wax should be used in accordance with AASHTO T 275-91 (2000). The G_{mb} obtained from AASHTO T-16 and AASHTO T 275-91 methods are presented in Table 3.4.

Specimens to be tested were dried to constant mass and the weight was obtained. The specimens were then coated on all surfaces with melted paraffin sufficiently thick to seal all voids and the weight was recorded. After retaining the coated weight, the specimen was immersed in a water bath at $77 \pm 2^\circ\text{F}$ and the submerged weight was determined. The bulk gravity (G_{mb}) of the samples was calculated using the following equation:

Table 3.3 AASHTO T-166 and AASHTO T 275-91 G_{mb} for 37.5 mm mix

Binder %	Sample Number	G_{mb}	
		AASHTO T-166	AASHTO T 275 - 91
3.4	1	2.3893	2.3426
	2	2.3791	2.3673
3.9	1	2.3924	2.3900
	2	2.3622	2.3550
4.4	1	2.4271	2.4285
	2	2.4222	2.4156
4.9	1	2.4419	2.4208
	2	2.4349	2.4234

$$G_{mb} = \frac{A}{D-E-\left[\frac{D-A}{F}\right]} \quad 3.12$$

Where

A = Mass of the dry specimen in air

D = Mass of the dry specimen plus paraffin coating in air

E = Mass of the dry specimen plus paraffin in water

F = Specific gravity of the paraffin at $77 \pm 2^\circ\text{F}$

CHAPTER 4: ANALYSIS AND RESULTS

The results and analysis of the research are discussed and presented according to volumetric properties, APA testing results, and the results of the IDT strength test.

4.1 VOLUMETRIC PROPERTIES

Volumetric properties and criteria for West Virginia SuperPave mix design for 19 mm and 37.5 mm mixes can be seen in Table 4.1. The mixes were designed with the same aggregate blend gradations. The design binder content increased for the lower gyration mixes as expected. By lowering the compactive effort while maintaining the VTM at 4.0 resulted in an increased binder content of 0.5 percent for 19 mm mixes. Lowering the gyrations for 37.5 mm mixes resulted in an increased binder content of 0.4 percent. The VMA increased which met the criteria for minimum requirements and the VFA was in the acceptable range.

Table 4.1 Volumetric Properties and West Virginia Criteria

NMAS	19 mm			37.5 mm		
	Criteria	100 Gyr.	80 Gyr.	Criteria	80 Gyr.	65 Gyr.
Pb, %	-	5.4	5.9	-	3.7	4.1
VTM, %	4.0	4.0	4.0	4.0	4.0	4.0
VMA, %	13.0	13.9	15.8	11.0	12.4	13.3
VFA, %	65-75	71.0	72.0	65-75	67	70.0

4.2 APA RESULTS

The rut depth values for the Asphalt Pavement Analyzer test are shown in Table 4.2. This shows the three values compacted at the original mix design and six values for mixes compacted with the compaction levels recommended by Prowell and Brown (2007). For the 19 mm mix, the average rut depth with 100 gyrations was 5.7 mm and

with 80 gyrations was 9.3 mm. For the 37.5 mm mix, the average rut depth with 100 gyrations was 5.3 mm and with 65 gyrations was 7.1 mm. For both mix designs, the mixes with higher asphalt content showed increased rutting potential.

Table 4.2 Rut depths (mm) for each specimen

Replication	19 mm 100 Gyrations (Original)	19 mm 80 Gyrations (Revised)	37.5 mm 80 Gyrations (Original)	37.5 mm 65 Gyrations (Revised)
1	6.1	6.0	5.3	8.2
2	4.3	8.0	6.5	5.4
3	6.8	10.0	4.0	7.5
4		13.4		5.0
5		12.6		7.8
6		6.9		8.8
Average	5.7	9.5	5.3	7.1

Research conducted by the Georgia Department of Transportation (GDOT) established a rut depth criterion of 5 mm to be good mixes (Collins, et al, 1995). Since the test was conducted at a temperature of 50 °C, the temperature effect model developed by Shami, et al (1997) was used to adjust the temperature to 60 °C to better correlate rut depth measured in the APA. This increased the limit from 5 mm to 8 mm. The GDOT criteria had an important effect on WVDOH since the lab data indicated the SuperPave mixes constructed in the state meet the criteria and have good field performance (Zaniewski and Patino, 2005).

4.2.1 ANOVA for Position in APA

Prior to investigating if the mix type or compaction effort factors were insignificant a check was performed. Previous work with the APA produced mixed results with respect to the issue of the effect of sample position in the APA. Patino (2005) found no effect; Hornbeck (2008) found samples in the right front and back, and the left front position had different means than the other positions. Patino's data set was much larger than Hornbeck's.

The effect of position on rut depth was evaluated with an Analysis of Variance. The p value from the ANOVA indicates the probability of getting a mean difference between the groups as high. Low p values indicate a low probability that a difference between groups occurred by chance. A p value of 0.05 was used in this research to identify when to reject the null hypothesis of equal means. All statistical analysis was performed using the Statistical Analysis Software (SAS).

Position was used as the factor with six levels. The three samples tested in each of the positions were treated as replicate samples, producing the data set shown in Table 4.3. The results of the ANOVA for sample position are shown in Table 4.4.

The ANOVA output is in Appendix B. The null hypothesis of equal means, at $\alpha = 0.05$, is rejected for the contrasts between the left and right channels, the middle and right channels, and for the combined left and middle versus the right channel. There is insufficient evidence to reject the null hypothesis for the contrast of front to back positions.

The rejection of the null hypothesis for several positions was investigated with the Duncan multiple range test. The result is shown in Figure 4.1. The positions are defined in Table 4.3. Figure 4.1 identifies that all positions other than position 3, middle front, "share a common mean." Given the limited size of the data set and the variability of the APA test the fact that not all positions shared a common mean can be accepted as an artifact of the experiment. However, in this case the position effect needs to be addressed to proceed with the investigation with the primary experimental factors: mix type and compaction effort.

Table 4.3 Sample Position in APA

Factor level	Replication		1	2	3	Average
1	Mix type		37.5	19	37.5	7.17
	Compaction		Revised	Revised	Original	
	Position	LF	8.2	8.0	5.3	
2	Mix type		19	19	37.5	6.20
	Compaction		Original	Revised	Original	
	Position	LB	6.1	6.0	6.5	
3	Mix type		19	37.5	37.5	4.43
	Compaction		Original	Revised	Original	
	Position	MF	4.3	5.0	4.0	
4	Mix type		37.5	19	19	7.43
	Compaction		Revised	Revised	Revised	
	Position	MB	5.4	10.0	6.9	
5	Mix type		37.5	37.5	19	9.63
	Compaction		Revised	Revised	Revised	
	Position	RF	7.5	8.8	12.6	
6	Mix type		19	37.5	19	9.33
	Compaction		Original	Revised	Revised	
	Position	RB	6.8	7.8	13.4	

Table 4.4 ANOVA Results for Sample Position in APA

Source of Variance	Sum of Squares	Degrees of Freedom	Mean Square	P-value
Treatments	57.0	5	11.4	0.0961
F = B	1.5	1	1.5	0.5818
LF = LB	1.4	1	1.4	0.5946
L = M	1.7	1	1.7	0.5597
L = R	23.5	1	23.5	0.0448
M = R	37.8	1	37.8	0.0149
L + M = R	40.3	1	40.3	0.0125
Error	56.3	12	5.0	0.1502
Total	113.0	17		

Figure 4.1 Duncan Multiple Range Test for APA Positions

RUT DEPTH VERSUS LOCATION IN APA					15:05 Friday, November
13, 2009					
The GLM Procedure					
Duncan's Multiple Range Test for DEPTH					
NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.					
Alpha					0.05
Error Degrees of Freedom					12
Error Mean Square					4.689444
Number of Means	2	3	4	5	6
Critical Range	3.852	4.032	4.141	4.214	4.264
Means with the same letter are not significantly different.					
Duncan Grouping	Mean	N	POSITION		
A	9.633	3	5		
A					
A	9.333	3	6		
A					
B A	7.433	3	4		
B A					
B A	7.167	3	1		
B A					
B A	6.200	3	2		
B A					
B	4.433	3	3		

The data were reorganized into the format shown in Table 4.2.3. The samples which were tested in position 3 were:

19 mm, original

37.5, original

37.5, revised

There was no assignable cause identified to support elimination of these data, so subsequent analyses were performed using both the original data set and a modified data set. The modification to the data set was to replace the observations from the middle front position with the average from the replicates prepared with the same mix type and compaction effort. The modified data set is shown in Table 4.5.

Table 4.5 Modified Data Set for Position in APA

Factor level	Replication		1	2	3	Average
1	Mix type		37.5	19	37.5	7.17
	Compaction		Revised	Revised	Original	
	Position	LF	8.2	8.0	5.3	
2	Mix type		19	19	37.5	6.20
	Compaction		Original	Revised	Original	
	Position	LB	6.1	6.0	6.5	
3	Mix type		19	37.5	37.5	4.43 6.86
	Compaction		Original	Revised	Original	
	Position	MF	4.3 6.4	5.0 5.9	4.0 8.3	
4	Mix type		37.5	19	19	7.43
	Compaction		Revised	Revised	Revised	
	Position	MB	5.4	10.0	6.9	
5	Mix type		37.5	37.5	19	9.63
	Compaction		Revised	Revised	Revised	
	Position	RF	7.5	8.8	12.6	
6	Mix type		19	37.5	19	9.33
	Compaction		Original	Revised	Revised	
	Position	RB	6.8	7.8	13.4	

The ANOVA to test for position was run with the modified data set with the results shown in Table 4.6. The null hypothesis of equal means was rejected for the contrast of the left versus right channels. However, the Duncan multiple range test, Figure 4.2, suggest common means for all positions.

Table 4.6 ANOVA for test Position with modified data set

Source of Variance	Sum of Squares	Degrees of Freedom	Mean Square	P-value
Treatments	29.0	5	5.8	0.3737
F = B	0.2	1	0.2	0.8270
LF = LB	1.4	1	1.4	0.6030
L = M	0.6	1	0.6	0.7217
L = R	23.5	1	23.5	0.0492
M = R	16.3	1	16.3	0.0932
L + M = R	26.3	1	26.3	0.0391
Error	58.9	12	4.9	
Total	17	87.9		

Figure 4.2 Modified Duncan Multiple Range Test for APA Positions

RUT DEPTH VERSUS LOCATION IN APA MODIFIED MF

15:41 Friday

November 13, 2009

The GLM Procedure

Duncan's Multiple Range Test for DEPTH

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	12
Error Mean Square	4.912778

Number of Means	2	3	4	5	6
Critical Range	3.943	4.127	4.239	4.313	4.364

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	POSITION
A	9.633	3	5
A			
A	9.333	3	6
A			
A	7.433	3	4
A			
A	7.167	3	1
A			
A	6.867	3	3
A			
A	6.200	3	2

4.2.2 ANOVA for Experimental Factors

The laboratory test on rutting was evaluated with the analysis of variance (ANOVA) method using the General Linear Model (GLM) with two factors being the mix design type (19 mm and 37.5 mm) and the compaction effort (100 and 80 gyrations for 19 mm mix and 100 and 65 gyrations for 37.5 mm mix). The factor levels are presented in Table 4.7. The ANOVA table for the original data set is shown in Table 4.8. The null hypothesis was that the original and redesigned mixes have the equal mean rutting potential.

Table 4.7 Factor Levels for Rut Depth Analysis

Mix Type	Gyrations	
19 mm	100 = M1	80 = M2
37.5 mm	100 = M3	65 = M4

The p value in the analysis for the means between M1 and M2 was 0.0273 which is less than 0.05, meaning the null hypothesis can be rejected for the 19 mm mixes, with 100 and 80 gyrations. The p value for the means for 37.5 mm mixes, M3 = M4 was 0.2442. Since this value is larger than 0.05, it indicated that there is insufficient evidence to reject the null hypothesis. The equal means for all factor levels was 0.1937 meaning there is no difference between means for the factors.

Table 4.8 ANOVA for Experimental Factors, original data set

Source of Variance	Sum of Squares	Degrees of Freedom	Mean Square	P-value
Treatments	48.4	3	16.1	0.0445
M1 = M2	28.1	1	28.1	0.0273
M3 = M4	6.8	1	6.8	0.2442
M1+M2 = M3 +M4	8.0	1	8.0	0.2091
Error	64.8	14	64.8	
Total	113.3	17		

The contrast between the mixes produced a p value of 0.2091 with the conclusion that there is insufficient information to reject the null hypothesis of equal means. The failure to reject the null hypothesis for the mix type comparison was unexpected as the conventional wisdom is increasing aggregate size should reduce rutting potential. Examining the means in Table 4.2 shows the mean rutting potential for the 19 mm

revised mix is much larger than the other means. None of the samples for the 19 mm revised mix, were tested in the middle front position.

The ANOVA was rerun with the modified data set producing Table 4.9. With the modified data from Table 4.5, the contrast for the mix type produced a p value of 0.2532 resulting in a conclusion that there is insufficient information to reject the null hypothesis. This indicates that even with the modified data set, there are no significant differences between the middle front position and the other positions.

Table 4.9 ANOVA for Experimental Factors with modified data set

Source of Variance	Sum of Squares	Degrees of Freedom	Mean Square	P-value
Treatments	33.5	3	11.1	0.0738
M1 = M2	18.6	1	18.6	0.0462
M3 = M4	6.2	1	6.2	0.2259
M1+M2 = M3 +M4	5.2	1	5.2	0.2532
Error	54.4	14	3.8	
Total	87.9	17		

4.3 INDIRECT TENSILE TEST (IDT) RESULTS

The IDT test was performed on the bulk specific gravity mix (G_{mb}) specimens that were used to determine the air voids of the mixes. The IDT test results are shown in Table 4.10. Strength versus binder results were graphed and a polynomial regression model was fit to determine IDT versus binder content for mixes designed using the lowered compaction effort recommended by NCHRP 9-9(1) (Prowell and Brown, 2007). Figure 4.3 and 4.4 shows the graph of IDT strength versus binder percent for 19 mm and 37.5 mm mixes.

Table 4.10 IDT Strength Test

Percent Binder	Mix Type	Label	Gyrations	Strength (lbs)	Average	Thickness	Tensile stress (σ_x) psi
5.9	19 mm	B1	80	500	560	4.45	12.12
	19 mm	B2	80	620		4.39	15.24
6.4	19 mm	B3	80	670	679	4.33	16.70
	19 mm	B4	80	688		4.40	16.87
5.4	19 mm	B5	80	618	616.5	4.45	14.98
	19 mm	B6	80	615		4.49	14.78
6.9	19 mm	B7	80	650	677.5	4.29	16.35
	19 mm	B8	80	705		4.28	17.77
5.4	19 mm	A5*	100	550	550	4.41	13.46
	19 mm	A6*	100	550		4.43	13.40
4.4	37.5 mm	A1	65	780	827.5	4.34	19.39
	37.5 mm	A2	65	875		4.30	21.96
3.9	37.5 mm	A3	65	695	545.5	4.46	16.81
	37.5 mm	A4	65	396		4.58	9.33
3.4	37.5 mm	A5	65	495	610	4.46	11.98
	37.5 mm	A6	65	725		4.45	17.58
4.9	37.5 mm	A7	65	790	727.5	4.34	19.64
	37.5 mm	A8	65	665		4.29	16.73
3.7	37.5 mm	A3*	80	810	977.5	4.30	20.33
	37.5 mm	A4*	80	1145		4.28	28.87

The IDT strength for the 19 mm mixes can be determined directly from Table 4.10 or Figure 4.3. The two specimens generated from the original 19mm mix design with 100 gyrations were averaged to represent the IDT strength, thus resulting in strength of 13.4 psi. The IDT strength for 19 mm mixes with 80 gyrations was 13.7 psi. This value was used for IDT strength due to the fact that the 19 mm mixes constructed with 80 gyrations resulted in a design binder content of 5.9 percent at 4 percent air voids.

The IDT strength for 37.5 mm mixes constructed with 100 gyrations could also be determined directly from Table 4.10 or Figure 4.4. The two specimens generated from the

mix design were averaged to represent the IDT strength of 24.6 psi. For 37.5 mixes with a compactive effort of 80 gyrations, the strength can be determined by using the design binder content (4.1 percent) from Figure 4.4.

Figure 4.3 IDT Strength vs Binder Percent for 19 mm Mixes

Figure 4.4 IDT Strength vs Binder Percent for 37.5 mm Mixes

IDT strengths for 19 mm and 37.5 mm NMAS and the corresponding binder content is presented in Table 4.11.

Table 4.11 IDT Strength for Design Binder Contents

NMAS	Gyrations	Binder Content (%)	IDT Strength (psi)
19 mm	100	5.4	13.4
	80	5.9	13.7
37.5 mm	80	3.7	24.6
	65	4.1	16.8

For the 19 mm mixes, the indirect tensile strength increased as the compaction level decreased with respect to an increase in binder content. This could be a result of cohesion and adhesion of the aggregates due to an increase in binder content with respect to stiffness. IDT strength for 37.5 mm mixes decreased significantly when the binder content was increased and the compaction effort decreased. This is a direct result of the larger aggregates being directly on top of each other during compaction in the SGC.

4.3.1 Comparing IDT with APA

The IDT test results and the APA rut depths were graphed to determine if the strength of the mixes increased or decreased with respect to rutting potential. Figure 4.5 and Figure 4.6 shows the IDT strength and APA rut depth for 19 mm mix and 37.5 mm mixes, respectively. For 19 mm mixes, IDT strength increases as rutting potential increases and for the 37.5 mixes, the IDT strength decreases as the rutting potential increases.

Figure 4.5 IDT Strength vs APA Rut Depth for 19 mm Mix

Figure 4.6 IDT Strength vs APA Rut Depth for 37.5 mm Mix

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

This research was performed to evaluate the affect lowering compaction effort, as recommended by the NCHRP 9-9(1) (Prowell and Brown, 2007), would have on SuperPave base course mixes used by the West Virginia Department of Highways (WVDOH). The two base course mixes currently used by the WVDOH are 19 mm NMAS with a design binder content of 5.4 percent and 37.5 mm NMAS with a design binder content of 3.7 percent. These mixes are designed with a PG 70-22 binder for a traffic load of 3 to 30 million ESALs. Both mixes were originally designed using a compaction effort of 100 gyrations under the current WVDOH practices.

The compaction effort was decreased from a 100 to 80 gyrations for the 19 mm mix and the 37.5 mm mix, was decreased from 100 to 65 gyrations as recommended by Prowell and Brown, (2007) and new design binder content was determined. The new design binder content increased for the lower gyration mixes as expected. The 19 mm mix resulted in an increased binder content of 0.5 percent and the 37.5 mm mix increased in a binder content of 0.4 percent. The original and new design binder contents were used to produce APA samples to evaluate the influence of increasing binder content on rutting potential. The Statistical Analysis Software was used along with a 95 percent confidence level to test the null hypothesis of equal means across the mix types and binder contents. The ANOVA test indicates that the means for the high and low gyrations for the 19 mm mix resulted in a p-value of 0.0273 which concludes that null hypothesis can be rejected for the 19 mm mixes. The means for the high and low gyrations for the 37.5 mm mixes resulted in a p-value of 0.2442 which concludes that there was insufficient evidence to reject the null hypothesis. Based on the findings from the ANOVA table, 19 mm mixes constructed with a lower compaction effort are not rut susceptible and 37.5 mixes constructed with a lower compaction effort had insufficient evidence to reject that it is rut susceptible.

The Indirect Tensile strength was also evaluated using the G_{mb} samples. With increased binder content, the 19 mm mixes demonstrated that, IDT strength increases as

rutting potential increases and for the 37.5 mixes, the IDT strength decreases as the rutting potential increases.

5.2 RECOMMENDATIONS

Determining the appropriate design binder content for asphalt concrete is crucial to long term pavement performance. The history of SuperPave mixes in WV has demonstrated the mixes are rut resistant. There has not been a long term evaluation of SuperPave performance. Given the generally good performance of SuperPave mixes, any change to the mix design methodology must be made with care.

The evaluation of the two mix types used in this study did demonstrate that selecting the design binder content from samples prepared with the same aggregate blend is increased when the compaction effort is reduced. This is in accordance with the recommendations of Prowell and Brown, (2007).

However, the data set used in this research was limited to allow interpolation of the results to the general population of asphalt concrete mixes used in the state. The mixes evaluated contained both slag and recycled asphalt pavement. Mix designs with these materials may or may not be representative of mixes prepared with 100 percent crushed limestone.

Only three samples per APA and two samples per IDT observation were used in this research. In light of the inherent variability of mix preparation and testing with these devices, research should be performed to evaluate the required complete size needed to produce more robust results.

The rutting potential of the mixes was evaluated. The statistical analysis indicate the 19 mm base with higher binder content has more rutting potential than the samples prepared using the current methodology. The null hypothesis of equal means was not rejected for the 37.5 mixes. However, since there are base mixes rutting potential may not be a critical issue. Modern pavement design methods, such as the Mechanistic Empirical Pavement Design System, demonstrate rutting is primarily a concern with SuperPave mixes. Base mixes are designed for fatigue resistance and the increasing asphalt content and film thickness should be beneficial to fatigue performance. However,

evaluation of fatigue behavior is outside the current capabilities of the West Virginia University Asphalt Technology Laboratory.

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APPENDIX A

Table A.1 Gradation and G_{sb} for 19 mm Mix

Asphalt Specific Gravity

G_b 1.036

							Design Aggregate Structure 1						
Aggregate Gradations							Percent of Aggregates						
	Size	L/S #6	L/S #8	L/S Sand	RAP	Spec	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	Combined
25	25	100	100	100	100		30	23	36	11.0	0	0	100
19	19	95	100	100	98.8		28.5	23	36	10.87	0	0	98
12.5	12.5	41	100	100	90	100	12.3	23	36	9.9	0	0	81
3/8	9.5	29	76	100	80	90-100	8.7	17.5	36	8.8	0	0	71
4	4.75	9	22	92	64	90 max	2.7	5.06	33.1	7.04	0	0	48
8	2.36	3.8	5	61	44.2	32-67	1.14	1.15	22	4.862	0	0	29
16	1.18	2.9	2.8	28	25		0.87	0.64	10.1	2.75	0	0	14
30	600	2.6	2.1	18	22		0.78	0.48	6.48	2.42	0	0	10
50	300	2.4	1.9	14	14.8		0.72	0.44	5.04	1.628	0	0	8
200	75	2.2	1.7	8.8	9.5	2-10	0.66	0.39	3.17	1.045	0	0	5.3
	G _{sb}	2.613	2.647	2.580	2.723							G _{sb}	2.623
	G _{sa}	2.713	2.729	2.735	2.723							G _{sa}	2.727

Table A.2 Gradation and G_{sb} for 37.5 mm Mix

Asphalt Specific Gravity

G_b 1.036

							Design Aggregate Structure 1						
Aggregate Gradations							Percent of Aggregates						
	Size	L/S #46	L/S #67	Slag Sand	RAP	Spec	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	Combined
50	50	100	100	100	100		50	11	24.0	15.0	0	0	100
37.5	37.5	96.1	100	100	100		48.05	11	24	15	0	0	98
25	25	55.5	100	100	100		27.75	11	24	15	0	0	78
19	19	34.7	97.1	100	98.8		17.35	10.7	24	14.82	0	0	67
12.5	12.5	9.8	70.6	100	97.6	100	4.9	7.77	24	14.64	0	0	51
3/8	9.5	3.1	51.5	100	93.8	90-100	1.55	5.67	24	14.07	0	0	45
4	4.75	2.2	16.8	94.5	66.1	90 max	1.1	1.85	22.68	9.915	0	0	36
8	2.36	2.2	6	70.5	44.2	32-67	1.1	0.66	16.92	6.63	0	0	25
16	1.18	2.1	4.5	49.2	33.2		1.05	0.5	11.81	4.98	0	0	18
30	600	1.8	4.1	32.2	26		0.9	0.45	7.728	3.9	0	0	13
50	300	1.7	4	19.4	14.8		0.85	0.44	4.656	2.22	0	0	8
200	75	1.4	3.6	6.1	6.9	2-10	0.7	0.4	1.464	1.035	0	0	3.6
	G _{sb}	2.639	2.632	2.779	2.723							G _{sb}	2.683
	G _{sa}	2.725	2.729	2.888	2.723							G _{sa}	2.763

Table A.3 Specific Gravity for 19 mm and 37.5 mm Blends

19 mm Mix			37.5 mm Mix		
Aggregate Type	Bulk Specific Gravity	Apparent Specific Gravity	Aggregate Type	Bulk Specific Gravity	Apparent Specific Gravity
LS #67	2.613	2.713	LS #467	2.639	2.725
LS #8	2.647	2.729	LS #67	2.632	2.729
LS SAND	2.58	2.735	SLAG SAND	2.779	2.888
RAP	2.723	2.723	RAP	2.723	2.723
Filler	2.628	2.628	Filler	2.628	2.628
Blend	2.620	2.726	Blend	2.683	2.783

Table A.4 Volumetric Parameters for 19 mm and 37.5 mm Verification Mix

Parameter	19 mm Mix	37.5 mm Mix
VTM	5.3	3.9
VMA	14.8	11.8
VFA	64.0	67.1
%G _{mm,Nini}	84.2	87.4
G _{se}	2.663	2.708
P _{ba}	0.6	0.4
P _{be}	4.834	3.358
D/B	1.3	1.1

Table A.5 Optimum Binder Content for 19 mm 80 Gyrations Mix

Parameter	Binder Percent (%)			
	5.4	5.9	6.4	6.9
VTM	5.9	4.0	3.1	0.1
VMA	16.2	15.8	15.7	15.1
VFA	63.4	75.0	80.4	99.4
%G _{mm,Nini}	84.250	85.964	86.847	89.245
G _{se}	2.7	2.7	2.7	2.7
P _{ba}	0.85	0.71	0.93	0.42
P _{be}	4.596	5.232	5.530	6.509
D/B	1.1	1	1	0.8

Table A.6 Optimum Binder Content for 37.5 mm 65 Gyrations Mix

Parameter	Binder Percent (%)			
	3.4	3.9	4.4	4.9
VTM	6.2	5.1	3.0	1.5
VMA	14.2	13.1	13.6	13.6
VFA	56.3	60.5	78.2	88.9
%G _{mm,Nini}	86.9	87.6	89.3	90.7
G _{se}	2.678	2.658	2.672	2.666
P _{ba}	-0.1	-0.4	-0.2	-0.2
P _{be}	3.468	4.256	4.553	5.128
D/B	1.0	0.8	0.8	0.7

Figure A.1 VTM vs Binder Percent for 19 mm 80 Gyrations Mix

Figure A.2 VMA vs Percent Binder for 19 mm 80 Gyrations Mix

Figure A.3 VFA vs Percent Binder for 19 mm 80 Gyrations Mix

Figure A.4 D/B vs Percent Binder for 19 mm 80 Gyrations Mix

Figure A.5 %G_{mm} vs Percent Binder for 19 mm 80 Gyrations Mix

Figure A.6 VTM vs Binder Percent for 37.5 mm 65 Gyrations Mix

Figure A.7 VMA vs Percent Binder for 37.5 mm 65 Gyrations Mix

Figure A.8 VFA vs Percent Binder for 37.5 mm 65 Gyrations Mix

Figure A.9 D/B vs Percent Binder for 37.5 mm 65 Gyrations Mix

Figure A.10 %G_{mm} vs Percent Binder for 37.5 mm 65 Gyrations Mix

APPENDIX B

Figure B.1 Data for SAS Program evaluating Rutting Potential

APA BASE COARSE RUT DEPTHS

22:14 Sunday, November 15, 2009

OBS SAMPLE FACTOR DEPTH

1	1	1	6.1
2	2	1	4.3
3	3	1	6.8
4	4	2	6.0
5	5	2	8.0
6	6	2	10.0
7	7	2	13.4
8	8	2	12.6
9	9	2	6.9
10	10	3	5.3
11	11	3	6.5
12	12	3	4.0
13	13	4	8.2
14	14	4	5.4
15	15	4	7.5
16	16	4	5.0
17	17	4	7.8
18	18	4	8.8

Figure B.2 Factor Levels for SAS Program Evaluating Rutting Potential

APA BASE COARSE RUT DEPTHS

22:14 Sunday, November 15, 2009

The GLM Procedure

Class Level Information

Class	Levels	Values	
SIZE	4	1 2 3 4	
Number of Observations Read			18
Number of Observations Used			18

Figure B.3 ANOVA Results for Base Course Rutting Potential

APA BASE COARSE RUT DEPTHS					
22:14Sunday,November 15, 2009					
The GLM Procedure					
Dependent Variable: DEPTH					
Source	DF	Squares	Sum of Mean Square	F Value	Pr > F
Model	3	48.4900000	16.1633333	3.49	0.0445
Error	14	64.8300000	4.6307143		
Corrected Total	17	113.3200000			
	R-Square	Coeff Var	Root MSE	DEPTH Mean	
	0.427903	29.21144	2.151909	7.366667	
Source	DF	Type I SS	Mean Square	F Value	Pr > F
FACTOR	3	48.4900000	16.1633333	3.49	0.0445
Source	DF	Type III SS	Mean Square	F Value	Pr > F
FACTOR	3	48.4900000	16.1633333	3.49	0.0445
Contrast	DF	Contrast SS	Mean Square	F Value	Pr > F
M1=M2	1	28.1250000	28.1250000	6.07	0.0273
M3=M4	1	6.8450000	6.8450000	1.48	0.2442
M1+M2=M3+M4	1	8.0277778	8.0277778	1.73	0.2091
EQUAL MEANS	2	17.1300000	8.5650000	1.85	0.1937
Parameter	Estimate	Standard Error	t Value	Pr > t	
H0: M1=M2	-3.7500000	1.5216297	-2.46	0.0273	
H0: M3=M4	-1.8500000	1.5216297	-1.22	0.2442	
H0: M1+M2=M3+M4	2.8333333	2.1519094	1.32	0.2091	

Figure B.4 Duncan Multiple Range Test for Rutting Potential

APA BASE COARSE RUT DEPTHS

22:14 Sunday, November 15, 2009

The GLM Procedure

Duncan's Multiple Range Test for DEPTH

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05		
Error Degrees of Freedom	14		
Error Mean Square	4.630714		
Harmonic Mean of Cell Sizes	4		

NOTE: Cell sizes are not equal.

Number of Means	2	3	4
Critical Range	3.264	3.420	3.516

Means with the same letter are not significantly different.

Duncan Grouping		Mean	N	FACTOR
A		9.483	6	2
A				
B	A	7.117	6	4
B				
B		5.733	3	1
B				
B		5.267	3	3

Figure B.5 Data for SAS Program evaluating Rutting Potential vs APA Position

RUT DEPTH VERSUS LOCATION IN APA

15:05 Friday, November 13, 2009

OBS SAMPLE POSITION DEPTH

1 1 1 8.2

2 2 1 8.0

3 3 1 5.3

4 4 2 6.1

5 5 2 6.0

6 6 2 6.5

7 7 3 4.3

8 8 3 5.0

9 9 3 4.0

10 10 4 5.4

11 11 4 10.0

12 12 4 6.9

13 13 5 7.5

14 14 5 8.8

15 15 5 12.6

16 16 6 6.8

17 17 6 7.8

18 18 6 13.4

Figure B.6 Factor Levels for SAS Program evaluating Rutting Potential vs APA Position

RUT DEPTH VERSUS LOCATION IN APA

15:05 Friday, November 13, 2009

The GLM Procedure

Class Level Information

Class	Levels	Values
POSITION	6	1 2 3 4 5 6
Number of Observations Read		18
Number of Observations Used		18

Figure B.7 ANOVA Results for Rutting Potential vs APA Position

RUT DEPTH VERSUS LOCATION IN APA					
15:05 Friday, November 13, 2009					
The GLM Procedure					
Dependent Variable: DEPTH					
Source	DF	Squares	Sum of Mean Square	F Value	Pr > F
Model	5	57.0466667	11.4093333	2.43	0.0961
Error	12	56.2733333	4.6894444		
Corrected Total	17	113.3200000			
	R-Square	Coeff Var	Root MSE	DEPTH Mean	
	0.503412	29.39610	2.165513	7.366667	
Source	DF	Type I SS	Mean Square	F Value	Pr > F
POSITION	5	57.0466667	11.4093333	2.43	0.0961
Source	DF	Type III SS	Mean Square	F Value	Pr > F
POSITION	5	57.0466667	11.4093333	2.43	0.0961
Contrast	DF	Contrast SS	Mean Square	F Value	Pr > F
F=B	1	1.5022222	1.5022222	0.32	0.5818
LF=LB	1	1.4016667	1.4016667	0.30	0.5946
L=M	1	1.6875000	1.6875000	0.36	0.5597
L=R	1	23.5200000	23.5200000	5.02	0.0448
M=R	1	37.8075000	37.8075000	8.06	0.0149
L+M=R	1	40.3225000	40.3225000	8.60	0.0125
EQUAL MEANS	4	38.5506667	9.6376667	2.06	0.1502
Parameter	Estimate	Standard Error	t Value	Pr > t	
H0: F=B	-1.7333333	3.06249717	-0.57	0.5818	
H0: LF=LB	0.9666667	1.76813356	0.55	0.5946	
H0: L=M	1.5000000	2.50051846	0.60	0.5597	
H0: L=R	-5.6000000	2.50051846	-2.24	0.0448	
H0: M=R	-7.1000000	2.50051846	-2.84	0.0149	
H0: L+M=R	-12.7000000	4.33102503	-2.93	0.0125	

Figure B.8 ANOVA Results for Base Course Rutting Potential (Modified Data)

APA BASE COARSE RUT DEPTHS					
22:44 Monday, November 16, 2009					
The GLM Procedure					
Dependent Variable: DEPTH					
Source	DF	Squares	Sum of Mean Square	F Value	Pr > F
Model	3	33.52777778	11.17592593	2.87	0.0738
Error	14	54.44833333	3.88916667		
Corrected Total	17	87.97611111			
	R-Square	Coeff Var	Root MSE	DEPTH Mean	
	0.381101	25.37366	1.972097	7.772222	
Source	DF	Type I SS	Mean Square	F Value	Pr > F
FACTOR	3	33.52777778	11.17592593	2.87	0.0738
Source	DF	Type III SS	Mean Square	F Value	Pr > F
FACTOR	3	33.52777778	11.17592593	2.87	0.0738
Contrast	DF	Contrast SS	Mean Square	F Value	Pr > F
M1=M2	1	18.60500000	18.60500000	4.78	0.0462
M3=M4	1	6.24222222	6.24222222	1.61	0.2259
M1+M2=M3+M4	1	5.52250000	5.52250000	1.42	0.2532
EQUAL MEANS	2	10.32750000	5.16375000	1.33	0.2965
Parameter	Estimate		Standard Error	t Value	Pr > t
H0: M1=M2	-3.05000000		1.39448318	-2.19	0.0462
H0: M3=M4	-1.76666667		1.39448318	-1.27	0.2259
H0: M1+M2=M3+M4	2.35000000		1.97209702		

Figure B.9 Duncan Multiple Range Test for Rutting Potential (Modified Data)

APA BASE COARSE RUT DEPTHS

22:44 Monday, November 16, 2009

The GLM Procedure

Duncan's Multiple Range Test for DEPTH

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	14
Error Mean Square	3.889167
Harmonic Mean of Cell Sizes	4

NOTE: Cell sizes are not equal.

Number of Means	2	3	4
Critical Range	2.991	3.134	3.222

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	SIZE
A	9.483	6	2
A			
B A	7.667	6	4
B A			
B A	6.433	3	1
B			
B	5.900	3	3

Figure B.10 ANOVA Test for Rutting Potential vs APA Position (Modified Data)

RUT DEPTH VERSUS LOCATION IN APA MODIFIED MIDDLE FRONT					
15:41 Friday, November 13, 2009					
The GLM Procedure					
Dependent Variable: DEPTH					
Source	DF	Squares	Sum of Mean Square	F Value	Pr > F
Model	5	29.02277778	5.80455556	1.18	0.3737
Error	12	58.95333333	4.91277778		
Corrected Total	17	87.97611111			
	R-Square	Coeff Var	Root MSE	DEPTH Mean	
	0.329894	28.51795	2.216479	7.772222	
Source	DF	Type I SS	Mean Square	F Value	Pr > F
POSITION	5	29.02277778	5.80455556	1.18	0.3737
Source	DF	Type III SS	Mean Square	F Value	Pr > F
POSITION	5	29.02277778	5.80455556	1.18	0.3737
Contrast	DF	Contrast SS	Mean Square	F Value	Pr > F
F=B	1	0.24500000	0.24500000	0.05	0.8270
LF=LB	1	1.40166667	1.40166667	0.29	0.6030
L=M	1	0.65333333	0.65333333	0.13	0.7217
L=R	1	23.52000000	23.52000000	4.79	0.0492
M=R	1	16.33333333	16.33333333	3.32	0.0932
L+M=R	1	26.35111111	26.35111111	5.36	0.0391
EQUAL MEANS	4	16.55333333	4.13833333	0.84	0.5245
Parameter	Estimate	Standard Error	t Value	Pr > t	
H0: F=B	0.7000000	3.13457422	0.22	0.8270	
H0: LF=LB	0.9666667	1.80974727	0.53	0.6030	
H0: L=M	-0.9333333	2.55936914	-0.36	0.7217	
H0: L=R	-5.6000000	2.55936914	-2.19	0.0492	
H0: M=R	-4.6666667	2.55936914	-1.82	0.0932	
H0: L+M=R	-10.2666667	4.43295738	-2.32	0.0391	

Figure B.12 Duncan Multiple Range Test for Rutting Potential (Modified Data)

RUT DEPTH VERSUS LOCATION IN APA MODIFIED MIDDLE FRONT

15:41 Friday, November 13, 2009

The GLM Procedure

Duncan's Multiple Range Test for DEPTH

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	12
Error Mean Square	4.912778

Number of Means	2	3	4	5	6
Critical Range	3.943	4.127	4.239	4.313	4.364

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	POSITION
A	9.633	3	5
A			
A	9.333	3	6
A			
A	7.433	3	4
A			
A	7.167	3	1
A			
A	6.867	3	3
A			
A	6.200	3	2

Table B.1 Weight-out Table for 19 mm Mix with RAP

Mix Weighout Table				19 mm																	
							P_b Total			5.4%		10,406.0			P_s						
2 Bulk				1 Gmm																	
4,500.0 g				2,000.0 g			Total Sample Weight			11,000.0 g		Wgt. Virgin Matr.			9,783.5 g						
Percent Stone P_s				94.6%			% Virgin Binder			4.7%											
Weight of Stone W_s				10,406.0 g			Weight of Binder W_b			522.1 g		% RAP			11.1%		P_b RAP		5.9%		
												Weight RAP			1216.5 g		W_b in RAP		71.77		
																	W_s in RAP		1,144.70		

Table B.2 Weight-out Table for 37.5 mm Mix with RAP

[illegible]

Figure B.11 IDT Test Plot

Figure B.12 IDT Strength vs Percent Binder for 19 mm Mix with Regression

Figure B.13 IDT Strength vs Percent Binder for 37.5 mm Mix with Regression

Figure B.14 APA Data Recording Sheet for Initial Readings

APA Initial Readings										12/1/2008		
			Position									
	Left	Middle	Right		Left	Middle	Right		Left	Middle	Right	
BACK	-0.58	-0.25	0.00	BACK	-0.13	-0.02	0.00	BACK	0.07	0.06	0.00	
FRONT	-0.47	-0.24	-0.02	FRONT	0.30	0.28	0.02	FRONT	0.14	-0.19	0.00	
<div style="display: flex; justify-content: space-around; align-items: center;"> <div style="text-align: center;"> <p>19 mm, 100 Gyr</p> </div> <div style="text-align: center;"> <p>37.5 mm, 65 Gyr</p> </div> <div style="text-align: center;"> <p>19 mm, 100 Gyr</p> </div> </div>												
BACK												
BACK	-0.03	-0.25	0.00	BACK	0.38	0.04	0.00	BACK	0.25	0.36	0.00	
FRONT	0.09	-0.33	0.00	FRONT	0.17	-0.05	0.00	FRONT	-0.08	0.22	0.00	
<div style="display: flex; justify-content: space-around; align-items: center;"> <div style="text-align: center;"> <p>37.5 mm, 65 Gyr</p> </div> <div style="text-align: center;"> <p>19 mm, 100Gyr</p> </div> <div style="text-align: center;"> <p>37.5 mm, 65 Gyr</p> </div> </div>												
FRONT												

Figure B.15 APA Data Recording Sheet for Final Readings

APA Final Readings										12/1/2008	
			Position								
			Left	Middle	Right				Left	Middle	Right
BACK				-6.28		BACK				-6.63	
FRONT				-5.28		FRONT				-7.21	
<p>19 mm, 100 Gyr</p>			<p>37.5 mm, 65 Gyr</p>			<p>19 mm, 100 Gyr</p>					
BACK											
			Position								
			Left	Middle	Right				Left	Middle	Right
BACK				-9.38		BACK				-7.01	
FRONT				-7.56		FRONT				-7.58	
<p>37.5 mm, 65 Gyr</p>			<p>19 mm, 100Gyr</p>			<p>37.5 mm, 65 Gyr</p>					
FRONT											