An Analysis of Asphalt Mix Design and Performance Properties by Using a Gyratory Compactor

Submitted to the Tennessee Department of Transportation

Research Development and Technology Program

Baoshan Huang, Ph.D., P.E.

Pawel Andrzej Polaczyk, Ph.D. Candidate

Wei Hu, Ph.D.

Department of Civil and Environmental Engineering

The University of Tennessee, Knoxville

February 2020

Technical Report Documentation Page

1. Report No. RES2016-02	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Asphalt Mixture Design and Performance Properties by Using a Gyratory Compactor		5. Report Date 6. Performing Organization Code
7. Author(s) Baoshan Huang, Pawel Andrzej Polaczyk, Wei Hu		8. Performing Organization Report No.
9. Performing Organization Name and Address The University of Tennessee 325 John D. Tickle Building Knoxville, TN 37996		10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Federal Highway Administration Central Federal Lands Highway Division 12300 W. Dakota Avenue, Suite 210 Lakewood, CO 80228		13. Type of Report and Period Covered 14. Sponsoring Agency Code HFTS-16.4

15. Supplementary Notes

COTR: First & Last name, FHWA CFLHD; Advisory Panel Members: List First & last names. This project was funded under the FHWA Federal Lands Highway Technology Deployment Initiatives and Partnership Program (TDIPP) or the Coordinated Technology Implementation Program (CTIP).

16. Abstract

TDOT is currently using the Marshall mix design method for designing asphalt mixtures. However, the Marshall method does not allow for designing larger stone mixes and it hinders the technical information exchange and communication among DOTs. To compare the Marshall and Superpave mix design methods and recommend an improved mix design method for TDOT, a wide range of TDOT typical asphalt mixtures were collected, designed and evaluated in this study.

The plant mixtures resulted in a high variation of the results of the equivalent N_{design}. The range of equivalent N_{design} for mixtures compacted in 150-mm mold was from 38 to 77 gyrations and from 32 to 86 gyrations for mixture compacted in 100-mm mold. For BM-2 mixtures the range of N_{design} was from 39 to 75 gyrations for 150-mm mold, and from 42 to 73 gyrations for 100-mm mold. Therefore, it was necessary to repeat the process using laboratory mixtures.

For laboratory mixtures, the N_{design} was defined based on the nine well-performing mixtures provided by TDOT. The results of back calculations showed that for BM2 mixes the range of equivalent N_{design} was from 71 to 75 gyrations (average 73 gyrations), while for D mixtures the range was from 64 to 72 gyrations (average 68 gyrations) by using 150-mm mold. Additionally, the concept of the locking point was investigated and a new method of determining the impact locking point was developed. This study will help TDOT keep the positive aspects of both mix design methods and take advantage of technical advancement, while a statewide campaign is necessary for the implementation of new mix design.

17. Key Words Superpave, Marshall, locking point, accelerometer	18. Distribution Statement No restriction. This document is available to the public from the sponsoring agency at the website http://www.cflhd.gov.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	xxx	

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

DISCLAIMER

This research was funded through the State Research and Planning (SPR) Program by the Tennessee Department of Transportation and the Federal Highway Administration under RES #: 2016-02, Research Project Title: Asphalt Mixture Design and Performance Properties by Using a Gyratory Compactor.

This document is disseminated under the sponsorship of the Tennessee Department of Transportation and the United States Department of Transportation in the interest of information exchange. The State of Tennessee and the United States Government assume no liability of its contents or use thereof.

The contents of this report reflect the views of the author(s) who are solely responsible for the facts and accuracy of the material presented. The contents do not necessarily reflect the official views of the Tennessee Department of Transportation or the United States Department of Transportation.

Acknowledgements

We would like to begin by thanking the Tennessee Department of Transportation (TDOT) for funding this research project. We have continued to collaborate closely with regional engineers and local technicians at the TDOT Materials and Test Division and local asphalt plants. They have provided valuable support towards the fulfillment of the research objectives. Without their support, it would be impossible for us to finish this research project. We would also like to thank the administrative staff from the TDOT research office who have worked very closely with our research team and kept the whole project on the proposed schedule.

Executive Summary

Over the past two decades, Superpave mix design has been further refined and most state Departments of Transportation (DOT) have fully or partially adopted it. TDOT is currently still using the Marshall mix design method for designing asphalt mixtures. However, the Marshall method does not allow for designing larger stone mixes and it hinders the technical information exchange and communication among DOTs. The goal of this research project is to compare the Marshall and Superpave mix design methods and recommend an improved mix design method for TDOT. Therefore, a wide range of TDOT typical asphalt mixtures were collected, designed and evaluated in this study in order to combine the benefits of both methods, which included the usage of the 150-mm mold to design large stone mixtures and the 100-mm mold to design surface mixtures.

Based on the investigation, all the Tennessee neighbor states utilize the modified version of Superpave based on their experience and best practice with Marshall method. Most neighbors use a modified gradation that is a combination of Superpave control points and Marshall gradations. Most of the states use modified consensus aggregate properties based on Marshall specifications, and only Arkansas uses the original American Association of State Highway and Transportation Officials (AASHTO) R35 criteria. Also, most of the states made modifications to the compaction effort (N_{design}) included in AASHTO R35. Currently, only thirteen states use AASHTO R35 compaction effort and only two use values recommended by NCHRP 573.

The plant mixtures resulted in a high variation of the results of the equivalent N_{design}. The range of equivalent N_{design} for Dmixtures compacted in 150-mm mold was from 38 to 77 gyrations and from 32 to 86 gyrations for mixture compacted in 100-mm mold. For BM-2 mixtures the range of N_{design} was from 39 to 75 gyrations for 150-mm mold, and from 42 to 73 gyrations for 100-mm mold. Therefore, it was necessary to repeat the process using laboratory mixtures.

The gradation of TDOT's D and BM2 mixes are close to Superpave 12.5 mm and 25 mm. Based on the implementations of the Superpave mix design in other states, it can be suggested that TDOT can choose any of the three options for aggregate gradation: 1) keeping current grading table, 2) implementing Superpave control points, or 3) making small modification to TDOT grading tables.

For laboratory mixtures, the N_{design} was defined based on the nine well-performing mixtures provided by TDOT. The results of back calculations showed that for BM2 mixes the range of equivalent N_{design} was from 71 to 75 gyrations (average 73 gyrations), while for D mixtures the range was from 64 to 72 gyrations (average 68 gyrations) by using 150-mm mold.

For all the mixtures included in this study, a higher N_{design} yielded lower asphalt binder content and slightly lower Tensile Strength Ratio (TSR). D-mixes have significantly higher moisture resistance (average 91%) than BM-2 mixes (average 81%). The resilient modulus increases when increasing N_{design} for all the mixtures included in this study, which could be attributed to the higher asphalt content for mixes with lower N_{design}. Similarly, all the mixtures showed higher failure stress with higher N_{design}. The samples prepared with lower N_{design} represented a larger strain when failed, indicating that higher asphalt content increased the ductility of asphalt mixtures. Samples using lower N_{design} generally had higher dissipated creep strain energy to fracture ($DCSE_f$), which indicates that lower N_{design} provided better resistance to failure as it required more energy to fracture asphalt mixture. All except one mixture presented higher rutting depth with lower N_{design}.

The accelerometer can determine different stages in the impact compaction process and to obtain the impact locking point. The impact locking point can be determined as the point where the acceleration and the response duration become stable. The gyratory locking points for the 150-mm specimens were significantly higher than those for 100-mm specimens, but the ranking for different mixtures kept unchanged regardless of the mold size. This finding allows the comparison of the locking points for different mixtures if the same size of mold is utilized.

Superpave mix design is generally utilizing the 150-mm mold for mix design purpose, even that the 100-mm mold can accommodate the aggregate smaller than 1". In this study, following the Superpave specification, limited research was performed on utilizing the 100-mm mold for D mixes design, which resulted in the equivalent N_{design} of 49 gyrations. It is lower than the equivalent N_{design} of 68 gyrations by using the 150-mm mold. This study will help TDOT keep the positive aspects of both mix design methods and take advantage of technical advancement. A statewide campaign is recommended for the implementation of new mix design.

Table of Contents

List of Tables

List of Figures

CHAPTER 1 INTRODUCTION

1.1 Problem Statement

The Superpave mix design was developed in the early 1990s during the Strategic Highway Research Program (SHRP) (1987-1992) to replace the Hveem and Marshall mix design methods. It is one of the primary outcomes from the SHRP study. Superpave is a comprehensive asphalt mix design and analysis system, including a Performance Grade (PG) asphalt binder specification, a series of aggregate tests and specifications, a hot mix asphalt (HMA) design and analysis system, and a computer software to integrate the system components. The Superpave mix design procedure involves careful material selection and volumetric proportioning as a first approach in producing a mix that will perform successfully. The four basic steps of Superpave asphalt mix design are materials selection, selection of the design aggregate structure, selection of the design asphalt binder content, and evaluation of the mixture for moisture sensitivity (Figure 1-1) (Cominsky et al. 1994).

One of the unique features of Superpave mix design is a new laboratory compactor for asphalt mixture called Superpave Gyratory Compactor (SGC). SGC was developed to improve laboratory simulation of the field compaction (Cominsky et al. 1994), which has the following advantages: (1) the compaction curve of samples can reflect compaction energy and indicate the compaction level needed for field compaction; (2) its 6” diameter mold is adequate for compacting asphalt mixtures with aggregate large than 1” sieve, such as TDOT’s A and A-S mixes. Figure 1-2 shows two types of SGC commonly used in the United States.

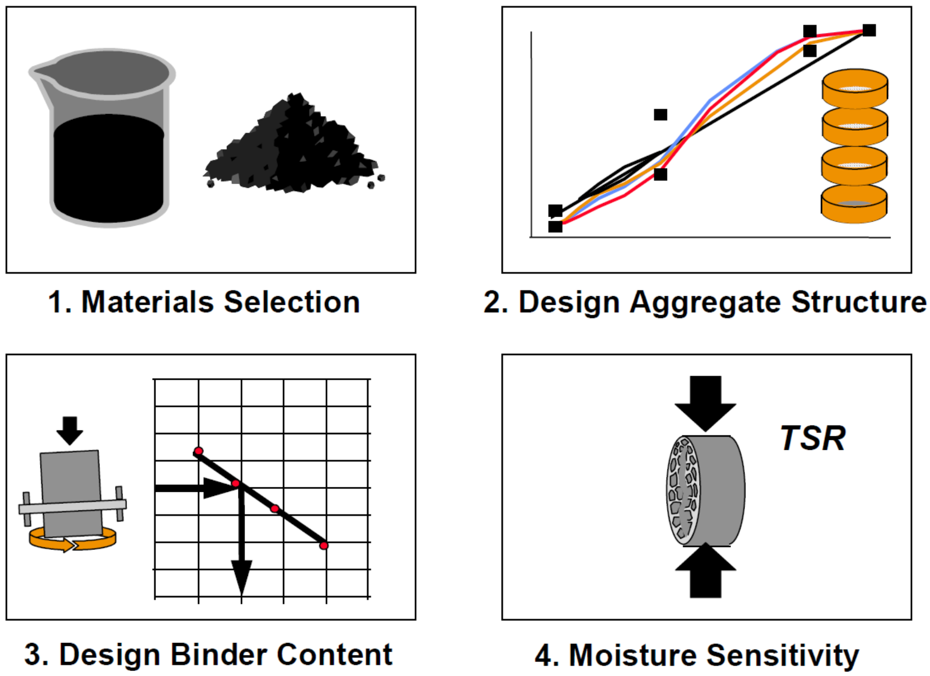


Figure 1-1. Four Steps of Superpave Mix Design

Figure 1-2. Two Types of Superpave Gyratory Compactor

The Marshall mix design was originally developed by Bruce Marshall, a Mississippi State Highway Department Engineer, and later refined in the 1940s by the Corps of Engineers for designing asphalt mixtures for airfield pavements. The primary features of Marshall mix design are a density/void analysis and the stability/flow test. Prior to Superpave, Marshall mix design was widely used in the United States and is by far the most commonly used mix design procedure worldwide (Herman et al. 2002).

Over the past two decades, Superpave mix design has been refined and most state DOTs have fully or partially adopted it. Unlike other state DOTs, TDOT is currently still using the Marshall mix design method for designing asphalt mixtures. TDOT has also refined this method over the years by specifying certain criteria to be met. This method has served TDOT well in the past and continues to be effective. However, the Marshall method currently does not allow for design of larger stone mixes due to the size limitations in the design method. It also hinders the technical information exchange and communication of TDOT engineers, staff, and administrators with their counterparts from other state DOTs that have adopted Superpave mix design (Herman et al. 2002).

The research project aims to compare Marshall and Superpave mix design methods using typical TDOT asphalt mixtures and recommend an improved mix design method for TDOT. This proposed method should be able to keep the positive aspects of both mix design methods, while enabling TDOT to take advantage of technical advancement and making technical communication and information exchange easier.

1.2 Objectives

The objectives of the proposed research were:

- To compare different asphalt mixture design methods using TDOT typical asphalt mixtures, including TDOT refined Marshall method, currently specified Superpave System (AASHTO M323/R35/T312), the modified version of the Superpave system (NCHRP 573);
- To recommend a modified mix design method combining the benefits of both Marshall and Superpave methods;
- To recommend a mix design method for larger stone mixtures that TDOT does not currently have.

1.3 Scope of Study

The scope of the research work included:

- To complete a synthesis of literature review on different asphalt mixture design methods in the US;
- To compare different asphalt mixture design methods using typical TDOT well-performing mixtures as well as good mixtures for the neighboring states.
- To conduct a statistical analysis on critical mix design parameters (such as volumetrics, compaction efforts, aggregate gradation) to determine the design requirements or criteria;

- To conduct a series of laboratory tests on typical well-performing TDOT mixtures.

1.4 Overview of the Final Report

The whole report was organized as follows: Chapter 1 gives a brief background of the project. Chapter 2 performs a comprehensive literature review on Superpave mix design. Chapter 3 evaluates the equivalent N_{design} of nine well-performing plant mixtures. Chapter 4 evaluates the equivalent N_{design} of nine well-performing laboratory mixtures. Chapter 5 summarizes the laboratory performance test results. Chapter 6 analyzes the effects of pre-treatment on the performance evolution of OGFC. Chapters 7 and 8 explore the concept of the locking point for gyratory compaction and develop the method to determine the locking point with impact compaction. The report concludes by summarizing the findings from the laboratory studies and performance evaluations.

CHAPTER 2 LITERATURE REVIEW

2.1 Background and Development of Asphalt Mix Design

The history of asphalt mix design could be dated to the beginning of the twentieth century when the paving industry realized the importance of correct mix design. Asphalt mix design is the process of determining the optimum proportions of asphalt cement, coarse aggregates, and fine aggregates for creating well-performing and long-lasting pavements. The first method to determine optimum binder content in the asphalt mix was the pat test, which was highly imprecise as it was based on visual appraisement. However, for asphalt mix designing pioneers, it still served as a useful tool for the improvement of asphalt quality and performance. Based on that, the bitulithic pavement (see Figure 2-1) was developed and patented by Federick Warren. This mix incorporated large stones up to three inches, allowing lower asphalt cement consumption and a lower price (Brown et al. 2009).

Figure 2-1. Bitulithic pavement placement process. Tampa, Florida (1920s.)

Another method, the Hubbard-Field test named after the inventors and members of the Asphalt Institute, was used to establish the optimum components especially for fine-graded mixes. This method was popular in designing asphalt mixes in many states until it was replaced by the Marshall method (Asphalt Institute 2007).

The 1930s was characterized as an important decade for asphalt mix design development. During this period, the Hveem and Marshall mix design methods were developed and dominated the paving industry in the United States and the rest of the world for over fifty years. Invented by Francis Hveem, the Hveem design method was based on three principles: the asphalt mix needs sufficient asphalt cement to cover each aggregate particle with optimum thickness; the asphalt mix needs enough stability to support traffic load; and finally as the thickness of the asphalt cement film increases, its durability also increases. Hveem’s method was used by 25% of States in the United States, especially on the West Coast until 1990. The Marshall method was developed by Bruce Marshall around 1939 and improved by the Corps of Engineers of the United States Army. This method of design uses values of flow, stability, and density to define optimum asphalt cement content. The Marshall mix design method was the most widely used method to design asphalt mix in the United States (75% of all States utilized this method) until 1990 as it started to be gradually replaced with the Superpave method (Asphalt Institute 2007; Asphalt Institute 2014; Brown et al. 2009; Pavement Interactive).

The Superpave design method was developed by the Strategic Highway Research Program with a budget of 150 million dollars from 1987 to 1992. Since all existing mix design methods were empirical methods based on observations and experiences, this new method was expected to design asphalt mixes with more predictable performance. The main research topics addressed by this research program included traffic volume and speed, long-term and short-term aging, climatic effects including a wide range of temperatures during production, compaction, and service. The Superpave introduced two important innovations in the equipment: the gyratory compactor which simulates the compacting process more efficiently than the Marshall hammer; and a new mold with a diameter of 6” compared with the Marshall method’s 4” diameter mold (see Figure 2-2). The Superpave design method has been gaining territories in the United States Since 1990, and currently most of the States have switched to this design method (Asphalt Institute 2007; Brown et al. 2009; Fishbaugh 2016).

Figure 2-2. Superpave specimen on the left and Marshall specimen on the right.

2.2 Current Practices of Asphalt Mix Design in U.S.

2.2.1 Florida Department of Transportation (FDOT) Experiences

The FDOT initialized the implementation of the Superpave mix design in 1996 to replace the Marshall method previously utilized. For more than ten years before the introduction of the Superpave, this state struggled with rutting failure problems. It was concluded that 50 Marshall hammer blows were inadequate for fine-graded asphalt mixes because of continually incrementing loading conditions (Fishbaugh 2016; Musselman et al. 1997).

During the initial stage of Superpave implementation, eight projects with a total of 325,000 tons of the asphalt mixes were chosen to be switched from Marshall to Superpave. The new mix design method seemed to offer potential improvement to rutting problems, but it lacked enough records of successful uses at national level. FDOT speeded up its implementation process by making supplemental agreements with the contractor of the ongoing projects, which gave chance to industry to learn about the new technology and shared potential risks of the speedup implementation. An investigation indicated that the first serious challenge was water since the coarse-graded Superpave mix was much more penetrable than traditional fine-graded Marshall mix. Due to that FDOT initially developed temporary specifications based on the existing standard specifications, the Superpave mixes were designed by contractors and verified by FDOT and had to fulfill the SHRP specifications A-379 and A-407, which addressed volumetric properties, aggregate consensus properties and moisture susceptibility. The fact that the areas with the highest traffic level had to be coarse graded means a gradation below the restricted zone (this zone was eliminated later from the Superpave specifications). Before the contractor was allowed to begin normal production, a trial production of 100 tons was evaluated based on the capacity of production, paving and compacting. During normal production, the contractor was required to strictly control the volumetric properties of the mix, and to stop operations if air voids were outside specifications during any laboratory test. Independently from the contractors, FDOT ran laboratory tests on the produced mix. The first Superpave project was designed with standard specifications with one exception related to binder selection as it was AC-30 that met specifications for PG 67-22. No significant problems were encountered during the production and placement process. However, problems arose during compaction. During the first Superpave project, FDOT found that in-situ air voids ranged from 10 to 13 percent compared to typical values ranged from 7 to 9 percent in Florida. To address the concerns of stripping failure, FDOT measured the permeability of extracted road cores, and the results showed that Marshall mixes were impermeable even with air voids above 9%, whereas 75% of Superpave mixes had permeability issues. It was determined that 6-7% of air voids are needed to decrease the permeability of coarse-graded Superpave mixes and match the permeability level of Marshall mixes, since the permeability depends not on the amount of air voids but the quantity of interconnections between them. The conclusions after the first project of Superpave included: 1) it is notably harder to compact coarse-graded Superpave mixes and they need higher density to decrease permeability than Marshall mixes; 2) the nuclear density measurement method in backscatter mode is less precise than in Marshall mixes; and 3) the Superpave has a different relation between layer thickness and compatibility from the Marshall mixes. To decrease permeability, FDOT adjusted specifications in relation to minimum percentage of Gmm as 94% and increased the minimum TSR to 85% (Fishbaugh 2016; Musselman et al. 1997). FHWA also recommended a minimum layer thickness of at least 4 times nominal maximum aggregate size of the mix to solve density and permeability problems (Fishbaugh 2016; Musselman et al. 1997).

2.2.2 Washington State Department of Transportation (WSDOT) Experiences

The WSDOT has been evaluating components of Superpave since 1993 and initiated the implementation in 1996 to replace the Hveem method. Unlike Florida, in Washington State, the process of new method implementation was divided in stages: binder specification validation, gyratory compactor, Superpave Shear Tester, and finally the performance of mixes. WSDOT was especially interested in low temperature cracking and conducted a forensic study in 28 old projects constructed between 1973 and 1983 in which the PG requirements were determined using two methods: the original SHRP and the LTPBIND software. The results showed very high agreement between field behavior of the mixes and PG requirements. When the original SHRP method was used it predicted performance correctly in 22 out of 28 projects, and LTPBIND predicted 26 out of 28 projects correctly. This data illustrated excellent efficiency of Superpave binder specifications (Willoughby et al. 2004; Leahy and Briggs 2001).

WSDOT expected that new gyratory compactor to be more suitable for field quality control purpose due to the size and weight of Hveem compactor. In addition, the Hveem mix design does not account for traffic loads expected in the new pavement, whereas Superpave considers these loads by adjusting compactive effort based on number of gyrations. Various mix designs were conducted with a goal to limit the number of compaction levels that was originally set to 28 by SHRP (combination of 7 traffic levels and 4 temperature levels). The results of initial research confirmed that it was possible to limit the number of compaction levels (Willoughby et al. 2004; Leahy and Briggs 2001).

In 2002, WSDOT reviewed the Superpave mixes performance placed in the last 6 years. The result showed that the Superpave mixes performed at the same level or better than the Hveem mixes for these indexes: Pavement Structural Condition (PSC), IRI, and rutting. The PSC comparison indicated that 60% of Superpave sections had less patching, cracking and rutting than traditional mixes. The comparison of low-temperature cracking failures showed that Hveem mixes had two to three times more cracks than Superpave mixes. The initial price of Superpave was considerably more expensive but the total cost for the service life was at the same level as Hveem mixes. Since Superpave was a national goal and it was performing at similar or better level as conventional mixes, WSDOT continued the implementation of the Superpave mix design method (Willoughby et al. 2004; Leahy and Briggs 2001).

2.2.3 Other States Department of Transportation Experiences

In 1999, Iowa Department of Transportation started to develop Superpave implementation plan. As the main point in the plan, the mixture analysis included evaluating gyratory properties of existing Marshall mixes. Most of the existing Marshall mixes met the gradation points requirement in Superpave. Some mixes only required minor changes in gradation. In December 2000, the implementation team reviewed the validation data collected and concluded that 80% of Marshall Mixes were fulfilled with Superpave mix design criteria, and the rest 20% of the mixes could meet the criteria with minor adjustments to job mix formula (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005).

The Louisiana Department of Transportation and Development started Superpave implementation in 1997, and the performance of 21 Superpave projects was evaluated after ten years of service. Based on the performance data, it was concluded that most of the projects are in good conditions with good rut resistance and roughness except for one project. The failure of the exceptional project was due to a failure of soil-cement base after visual inspection. In general, the degree of deterioration was lower than that from the performance model predicted (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005).

The North East Asphalt User/Producer Group (NEAUPG) Annual Meeting in 2003 concluded that Superpave Mixes were susceptible to material alterations caused by the Asphalt Plant (breakdown, change in texture) and were more affected by variations in gradation. The "best practices" of asphalt plant and paving for Marshall and Hveem Mixes also applied to Superpave and should be used to avoid segregation of the mix in the plant, truck, and paver. Coarse-graded Superpave mixes were generally harder to compact than Marshall mixes, and some coarse-graded Superpave mixes showed a Tender Zone and it was important to build a test strip (Brown et al. 1999; Choubane et al. 1998; Federal

Highway Administration #131053; Transportation Research Board 2005). In 2005, the TRB Superpave Committee published the final report including benefit indicators from different agencies of the United States and Canada (**Table 2-1**). Most of the agencies included in the report indicated a lower or the same cost of asphalt mixes after Superpave implementation and only Washington State reported 3% cost increases. All agencies reported some types of performance improvement, especially in reducing the rutting. The lifespan increases of asphalt mixtures ranged from one to three years (Transportation Research Board 2005).

Table 2-1. Superpave benefit indicators

Arkansas	Problems that were common with Marshall mixes occurred considerably less often.
Connecticut	Noticed reduced rutting on pavement segments prone to rutting
Louisiana	Less rutting observed
Minnesota	Better ride & pavement sufficiency, slightly lower cost
New York City	No cost increase, 1-3 years in extra performance
Ontario	2% lower in cost, 1-2 years increased performance
Pennsylvania	Seems to have resolved the rutting problem
Test Section SPS-9	Reflective cracking retarded 3-4 years
Utah	3-years life increase. 10% LCC savings and crack sealing cost down 70%, patching cost down 20%
Washington State	3% higher in cost, 12-20% longer performance
City of Calgary	Better performance at the same cost
City of Ottawa	Marked reduction in cracking

New Hampshire Department of Transportation initiated first two Superpave projects in 1997 and fully implemented the new mix design method in 2003. Prior to the implementation of Superpave, the mix designed by the Marshall method suffered from environmental cracking issues. In the early stage of Suerpave implementation the new mixes were difficult to compact with high in-place voids, low asphalt content, and an elevated permeability. To improve durability of the mixes, changes including a lower number of gyration (originally 125 and 100 gyrations), eliminating restricted zone, establishing minimum asphalt content, using finer gradation, and using joint adhesive eliminated those initial problems of Superpave mixes (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005).

Nevada Department of Transportation built test track paved with Superpave and Hveem mixes to evaluate and compare the performance. Laboratory evaluations were conducted, and the performance of the mixes was monitored. After five years of service, it was found that the mix designed with the same material could have similar or very different optimum asphalt content. The fine-graded mixes were less sensitive to variation in asphalt content and fines content. Mixes with gradation curve passing the restricted zone had better performance than coarse-graded mixtures. The Hveem mixtures showed less durability, cracking and fatigue resistances when exposed to low traffic and less rutting resistance when exposed in high traffic (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005).

In 2014, Texas Department of Transportation indicated that the advantages of new mix design method included the coarser surface for increased safety during raining, better performance on high or medium traffic roads, and the asphalt content modified by adjusting N_{desing} level. The disadvantages list included challenges of compacting, intermediate temperature tender zone, and infiltration of water (Lee and Hoelscher 2014).

2.2.4 Superpave Construction Issues

By 2000, the leading state DOTs including Wisconsin, Maryland, Arizona, Virginia, Colorado, and New York started to survey the Superpave performance and construction experiences with the new method of asphalt mix design (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005). For asphalt cements, most agencies modified PG specifications to be more restricted during implementation process. The majority of asphalt suppliers did not have a problem in providing PG graded asphalt. As for aggregates, suppliers were able to produce satisfied materials required by Superpave, even though the new aggregate specifications required a washing process. Also, the number of different sizes of aggregate was increased to follow Superpave specifications. During the mix design process, the main problems in satisfying volumetric properties included low voids and the need for more crushed sand and more cubical aggregates. Also, at the initial stage mix designers usually needed more than ten trial blends to define the optimum one, but with gaining experience the number of trial mixes was significantly reduced and for inexperienced designers Superpave Centers offered help and advice (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005). The increased number of different aggregates required more feed bins in the asphalt plants ranged from 6 to 8. The production of Superpave in the asphalt plant generally had similar production process and rate to the Hveem/Marshall. However, most agencies and contractors reported that the new mixes required more control in the gradation and baghouse operation. Paving operations did not require different approaches from Hveem/Marshall mixes, and the velocity of work was the same as the conventional mixes. Due to the coarse nature of Superpave mixes it was noticed that the cooling process was faster than that of Hveem/Marshall mixes (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005). As a result, the most critical part of the new mix was the compaction process. Most agencies and contractors reported various problems in compacting, and more compaction effort was required to compact Superpave than conventional mixes. The tenderness zone was another problem which is defined as the specific range of temperatures hard to achieve an effective compaction. It was reported that for the Superpave mixes the range of tender zone is around 275 ºF to 200 ºF (Brown et al. 1999; Choubane et al. 1998; Federal Highway Administration #131053; Transportation Research Board 2005).

It was commonsense for many years that with traditional mixes the layer thickness should be 2 to 2.5 times of the maximum aggregate size. However, traditional mixes were designed using the maximum aggregate size when Superpave utilized the nominal maximum size. It is recommended that the thickness of Superpave layers should be 3 to 4 times of the nominal maximum size of aggregates. The segregation of the mix was similar in Superpave as in other mixes, but in Superpave it required more attention to notice its presence due to a coarser structure of the mix.

2.3 Summary of HMA mix design methods in the neighbor states

The survey result about the standard specification (**Table 2-2**) shows that the implementation of Superpave Mix Design was based on the experience with Marshall Method. Most of the States modified the original Superpave method to include their best practice from previous methods. All the neighbor States adopted the original Superpave AASHTO M323/R53/T312 Method. The influence of experience with Marshall method is indicated in types of gradation used by different states as most of them use a modified control point based on gradation in previous methods. Only two States, Kentucky and Arkansas, use original Superpave control points. Seven out of eight States use 4% of air voids as design criteria when Arkansas uses 4.5%. Five States use Superpave Fine and Coarse Aggregate Consensus property and the other three states use modified properties. All surveyed States use the Performance Grading system and one State, Virginia, uses an improved PG Plus classification. The design criteria adopted by different states are significantly different. Only Arkansas uses original AASHTO R35 criteria, and the rest of the States made modifications based on numerous factors like equivalent single axle load (ESAL), mix, and binder types.

Table 2-2. Superpave specifications in the neighbor states

State	Kentucky	Virginia	North Carolina	Georgia	Alabama	Missisipi	Arkansas*	Missouri
Design Method	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312	Superpave AASHTO M323/R35 / T312
Type of Mix	AASHTO M 323	3/8", 1/2", 3/4", 1"	9.5mm, 12.5mm, 19.0mm, 25.0mm	AASHTO M 323	AASHTO M 323	AASHTO M 323	AASHTO M 323	4.75mm, 9.5mm, 12.5mm, 19.0mm, 25.0mm
Gradation	AASHTO M 323	Modified Control points	Modified Control points	Modified Control points	Modified Control points	Modified Control points	AASHTO M 323	Modified Control points
Aggregate	Superpave Fine and Coarse Aggregate Consensus Property	Modified Superpave Fine and Coarse Aggregate Consensus Property	Modified Superpave Fine and Coarse Aggregate Consensus Property	Modified Superpave Fine and Coarse Aggregate Consensus Property	Superpave Fine and Coarse Aggregate Consensus Property	Superpave Fine and Coarse Aggregate Consensus Property	Superpave Fine and Coarse Aggregate Consensus Property	Superpave Fine and Coarse Aggregate Consensus Property
Binder Classification	PG	PG Plus	PG	PG	PG	PG	PG	PG
N_{design}	<3 50 3-30 75 >30 100	65	Vary on ESAL, Mix Type and Binder	65, 50 for 4.75mm mixes	60	50, 65, 85	AASHTO R 35	50, 75, 100(80), 125
Performance tests	TSR	APA	APA	TSR, APA, Flexural Bending, Hamburg Wheel	TSR	TSR	APA	TSR

*Note, *- uses 4.5% as optimum air void content.*

2.4 Superpave compaction effort (N_{design}) used by State Departments of Transportation

The Superpave mix design has been utilized in the United States since 1993. Currently, forty-six states and Washington DC utilize Superpave mix design method, and three states, Alaska, Hawaii, and Tennessee, still use Marshall mix design, and one state, Nevada, uses Hveem mix design. Since the introduction of Superpave, material specifications and test procedure has been continuously refined due to gained field experiences. The initial compaction effort was included in AASHTO R-35. There has been voices from different contractors and agencies that Superpave mix design resulted in mixtures containing low asphalt cement content. After that, the National Cooperative Highway Research Program (NCHRP) evaluated the issue in the NCHRP 573 report and suggested a new, lower gyratory compaction level. This recommendation was based on studies performed on field densification and statistical analyses. **Table 2-3** shows the compaction effort comparison between AASHTO R-35 vs. NCHRP 573.

Table 2-3. Comparison between AASHTO R-35 vs. NCHRP 573 compaction effort

20-Year Design Traffic, ESALs (millions)	AASHTO R 35 N_{design}	NCHRP N_{design} <PG 76-XX	NCHRP N_{design} >PG 76-XX or mixes placed >100 mm from surface
< 0.3	50	50	---
0.3 to 3	75	65	50
3 to 30	100	80	65
>30	125	100	80

NCHRP 573 decreased the required N_{design} and implemented different values based on the grade of asphalt binder. Asphalt mixes with the binder grade lower than PG76-XX required higher values of N_{design}. The Federal Highway Administration (FHWA) conducted a broad evaluation of the NCHRP results and concluded with no general recommendation for the reduction in gyratory levels. The FHWA recommended that agencies should conduct an internal evaluation before adjusting compaction effort from the AASHTO R 35. Currently, only thirteen states use AASHTO R35 compaction effort and only two states use values recommended by NCHRP 573. **Table 2-4** presents N_{design} used by different agencies.

Table 2-4. Compaction effort used by state agencies

No.	State	Gyrations	Notes
1	Alabama	60	All mixtures
2	Alaska	---	Marshall Mix Design
3	Arizona	50, 75, 100, 125	Special Provisions
4	Arkansas	50, 75, 100, 125	50, 75 (PG64-22), 100 (PG70-22), 125 (PG76-22)
5	California	85	
6	Colorado	75, 100	100-mm diameter specimens
7	Connecticut	75, 100	Towns/municipalities use 50
8	Delaware	75	
9	Florida	50, 65, 75, 100	Mostly 75 and 100
10	Georgia	50, 65	4.75 mm mixtures use 50
11	Hawaii		Marshall Mix Design
12	Idaho	50, 75, 100	
13	Illinois	30, 50, 70, 90	SMA 80, 50
14	Indiana	75, 100	SMA 75
15	Iowa	50, 60, 65, 68, 76, 86, 96, 109, 126	Original N_{design} level +3 for low traffic volume mixes
16	Kansas	75, 100	Want to switch to 60, or 3% air voids at 75.

17	Kentucky	50, 75, 100	
18	Louisiana	55, 65	
19	Maine	50, 75	50 more common
20	Maryland	50, 65, 80, 100	SMA 100
21	Massachusetts	50, 75, 100, 125	75 and 100 more common
22	Michigan	45, 50, 76, 86, 96, 109, 126	Original N_{design} level +2 for low traffic volume mixes
23	Minnesota	40, 60, 90, 100	Depends on ESAL
24	Mississippi	50,65, 85	65 used more frequently
25	Missouri	50, 75, 80, 100, 125	
26	Montana	75	Before 100
27	Nebraska	40, 65, 95	40 for shoulders, 65 low volume traffic, 95 high volume traffic
28	Nevada		Hveem Mix Design
29	New Hampshire	50, 75	3.0% to 3.5% air voids on 9.5 mm 75 gyrations
30	New Jersey	50, 75	50 rarely used
31	New Mexico	75, 100, 125	75 low volume traffic, 125 urban interstates, 100 the rest
32	New York	50, 75, 100	50 and 100 rarely used
33	North Carolina	50, 65, 75, 100	50 for low traffic volume fine 9.5 mixes
34	North Dakota	75	
35	Ohio	65	
36	Oklahoma	50, 60, 80	50 (PG64-22), 60 (PG70-28), 80 (PG76-28)
37	Oregon	65, 80, 100	
38	Pennsylvania	50, 75, 100	
39	Rhode Island	50	
40	South Carolina	50, 75	Depends on the type of the road
41	South Dakota	40, 50, 60, 70, 80	
42	Tennessee		Marshall Mix Design
43	Texas	50	Can be reduced by the Engineer to 35
44	Utah	50, 75, 100, 125	75 used more frequently
45	Vermont	50, 65, 80	50 and 80 rarely used
46	Virginia	65	Researching 50
47	Washington State	50, 75, 100, 125	

48	Washington DC	100	
49	West Virginia	50, 65, 80, 100	If PG76-XX is used: 80 is decreased to 65, and 100 to 80
50	Wisconsin	40, 75, 100	SMA 65
51	Wyoming	50, 75, 100	

From **Table 2-4** it can be concluded that most of the states developed their own values of the compaction effort. Some states use just one value for all the mixtures, whereas others use different values for different traffic load, mix type, or asphalt binder grade.

CHAPTER 3 INTRODUCTION OF SUPERPAVE DESIGN

Superpave is a comprehensive asphalt mix design and analysis system, including a Performance Grade (PG) asphalt binder specification, a series of aggregate tests and specifications, a hot mix asphalt (HMA) design and analysis system. The section briefly introduced the Superpave correlated tests performed in this study.

3.1 Consensus and Source Properties of Aggregates

The Superpave mix design includes guidance on aggregate selection. There are two types of properties: consensus and source. Consensus properties are the outcome of the SHRP program and include fine aggregate angularity, coarse aggregate angularity, flat and elongated particles and sand equivalent. Source properties are properties considered as inherent and include Los Angeles abrasion, soundness and clay lumps. Source properties requirements are set by local agencies.

The coarse aggregate angularity is tested following ASTM D5821, which measures the participation of particles with fractured faces. The sample is divided in three groups: zero fractured faces, one fractured face, and two fractured faces. The fine aggregate angularity (ASTM C1252) is determined by the value of uncompacted voids in the material poured into the specific cylinder. The higher number of voids represents higher angularity of fine aggregate. The number of uncompacted voids (1) is calculated as follows:

$$Uncompacted\ voids = \frac{V-\frac{W}{G_{sb}}}{V}100 \tag{1}$$

Where, V - volume of cylinder, W - weight of loose fine aggregate, G_{sb} - bulk specific gravity of fine aggregate,

The flat and elongated particles test (ASTM D4791) is important to assure good performance of asphalt mixture. A high content of flat and elongated particles can result in a mixture with low voids in mineral aggregate. Also, these particles are easier to break down during compaction resulting in lower performance, workability and compactability. The test is performed by dividing the longest side of particle by the shortest side and the result should be lower than five.

Sand equivalent test (ASTM D2419) determines the clay content in the aggregate. The high clay content can prevent a good bonding between aggregates and asphalt binder and result in stripping.

3.2 Other test methods of Aggregates

Specific gravity of the aggregates is an important property, which permits easy volume-weight conversion and allows determination of void content in compacted asphalt mixture. Specific gravity is the ratio of the weight of a specific volume of aggregate to the weight of the same volume of water at approx. 25°C. There are different specific gravities utilized for asphalt mixture: apparent specific gravity, bulk specific gravity and effective specific gravity.

- Aggregate Bulk Specific Gravity (G_{sb})

$$G_{sb} = \frac{W_s}{(V_s+V_{pp})\gamma_w} \tag{2}$$

Where, G_{sb} is a bulk specific gravity, W_s is a weight of solids, V_s is a volume of solids, V_{pp} is a volume of water permeable pore, and γ_w is density of water.

- Aggregate Apparent Specific Gravity (G_{sa})

$$G_{sa} = \frac{W_s}{(V_s)\gamma_w} \tag{3}$$

Where, G_{sa} is apparent specific gravity, W_s is a weight of solids, V_s is a volume of solids, and γ_w is density of water.

- Aggregate Effective Specific Gravity (G_{se})

$$G_{se} = \frac{W_s}{(V_s+V_{pp}-V_{ap})\gamma_w} \tag{4}$$

Where, Gsb is bulk specific gravity, Ws is a weight of solids, Vs is a volume of solids, Vpp is a volume of water permeable pore, Vap is a volume of pore absorbing asphalt, and γw is density of water.

There are two tests needed to calculate the specific gravity of aggregates. One is for coarse aggregates (retained on sieve No. 4) and the other is for fine aggregates (passing sieve No. 4). Procedure and equipment required for testing coarse specific gravity is specified in ASTM C127. It can be summarized as taking 5 kg of material retained on sieve No. 4, drying the material to constant weight, soaking for 24 hours, emptying water and using towel to get saturated superficially dry (SSD) condition, determining mass of soaked superficially dry aggregates (B), determining mass under water (C), determining oven dry mass (A). Coarse specific gravity can be calculated as follow:

$$G_{sb} = \frac{A}{(B-C)} \tag{5}$$

$$G_{sa} = \frac{A}{(A-C)} \tag{6}$$

$$Absorption\ \% = \left[\frac{(B-A)}{A}\right] 100\% \tag{7}$$

The ASTM C128 standard can be used to determine the specific gravity of the fine aggregate and the procedure can be summarized as following: adding 500 g of SSD material (S) to pycnometer of known volume, filling the pycnometer to line and determining the mass of pycnometer with aggregate and water (C), determining oven dry mass (A), determining the weight of pycnometer filled with water (B). The fine aggregate specific gravities and absorption are calculated as follow:

$$G_{sb} = \frac{A}{(B+S-C)} \tag{8}$$

$$G_{sa} = \frac{A}{(B+A-C)} \tag{9}$$

$$absorption\ \% = \left[\frac{(S-A)}{A}\right] 100\% \tag{10}$$

The combined specific gravity of aggregate (11) is calculated based on percentage of course and fine material:

$$G = \frac{P_c + P_f}{\frac{P_c}{G_c} + \frac{P_f}{G_f}} \tag{11}$$

Where, G is combined specific gravity of aggregates, P_c is a percentage of coarse aggregate, P_f is a percentage of fine aggregate G_c is specific gravity of coarse aggregate, G_f is specific gravity of fine aggregate

Similar formula is utilized to calculate specific gravity of the blend of aggregates:

$$G = \frac{P_1 + P_2 + \dots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \dots + \frac{P_n}{G_n}} \tag{12}$$

Where, G is combined specific gravity of the blend of aggregates, $P_1, P_2, \ldots, P_n$ are percentage of fractions 1,2,…,n, and $G_1, G_2, \ldots, G_n$ are specific gravity for fractions 1,2,…,n.

Effective specific gravity (G_{se}) of aggregates is determined from asphalt mix test and can be calculated as follow:

$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}} \quad (13)$$

Where, P_b is asphalt binder content, G_{mm} is theoretical maximum specific gravity, and G_b is specific gravity of asphalt binder.

3.3 Superpave Gyratory Compactor

The Superpave Gyratory Compactor (SGC) is a key device of the Superpave mix design. The specific number of gyrations is selected to obtain specimens with similar density as field samples. The advantage of gyrations over impact (Marshall hammer) is that gyration leave the aggregate particle with similar orientation as the field compaction. Three parameters control the compaction: number of gyrations, angle of gyration, and vertical pressure. Superpave mix design defines two angles of gyrations, internal (1.16°) and external (1.25°), and a standard vertical pressure of 600 kPa (87 psi). The number of gyrations should supply the compaction effort similar to the field situation after certain years of traffic. The compaction effort should be applied at the rate of 30 gyrations per minute. The SGC utilizes two types of molds, 150 mm (~6 inches) and 100 mm (~4 inches). The most common used mold is 150 mm because it can accommodate asphalt mixtures with large stones. The disadvantage of larger mold is the high amount of material required to prepare samples.

3.4 Asphalt Mixture Design Requirements

Superpave mix design has similar design requirements as Marshall mix design. However, Superpave does not use stability and flow criteria. To determine the optimum asphalt content a series of specimens containing different asphalt content are compacted. The Superpave defines optimum asphalt content as the asphalt content at 4% of air voids in total mix (VTM) at specific N_{design}. Additional properties such as voids in mineral aggregate (VMA) and voids filled with asphalt cement (VFA) should be determined and fulfill the specification. If all the volumetric properties meet specification, the moisture susceptibility should be tested by one of the methods. Superpave recommends AASHTO T283 method.

The three volumetric parameters, VTM, VMA, and VFA, are important to the quality of asphalt mixture. **Table 3-1** summarizes the Superpave volumetric requirements.

Table 3-1. Superpave volumetric requirements

ESAL (Million)	VTM (%)	MIN. VMA (%)						VFA (%)	Dust to binder Ratio
		37.5 mm	25.0 mm	19.0 mm	12.5 mm	9.5 mm	4.75 mm		
<0.3	4	11	12	13	14	15	16	70 - 80	0.6 - 1.2
0.3 - 3.0	4	11	12	13	14	15	16	65 - 78	0.6 - 1.2
3.0 - 10.0	4	11	12	13	14	15	16	65 - 75	0.6 - 1.2
10.0 - 30.0	4	11	12	13	14	15	16	65 - 75	0.6 - 1.2
>30.0	4	11	12	13	14	15	16	65 - 75	0.6 - 1.2

VTM is the total volume of air pockets in the asphalt mixture expressed as a percent of the bulk volume of the compacted asphalt mixture. It is calculated as follows:

$$G_{mb} = \frac{W_D}{W_{SSD} - W_{Sub}} \tag{14}$$

Where, G_{mb} is bulk specific gravity of compacted asphalt mixture specimen, W_D is dry weight of specimen, W_{SSD} is saturated surface dry weight, W_{Sub} is saturated surface dry weight in water.

$$G_{mm} = \frac{1}{\left(\frac{1-P_b}{G_{se}}\right) + \left(\frac{P_b}{G_b}\right)} \tag{15}$$

Where, G_{mm} is maximum theoretical specific gravity, P_b is asphalt content by weight of mixture, G_{se} is effective specific gravity of aggregates, and G_b is specific gravity of asphalt.

$$VTM = \left(1 - \frac{G_{mb}}{G_{mm}}\right) 100 \tag{16}$$

Where, VTM is air void, G_{mm} is maximum theoretical specific gravity, and G_{mb} is bulk specific gravity of compacted asphalt mixture specimen.

Voids in the mineral aggregate (VMA) is a total volume of voids in the mass of aggregate. This volumetric parameter has important impact on performance of the asphalt mixture. Too small value of VMA can affect durability of the mixture and too high value can provoke stability problems. The VMA has two parts, the voids filled with asphalt and the voids filled with air. In Superpave mix design, the VMA is calculated as follow:

$$VMA = 100 - \frac{G_{mb} P_s}{G_{sb}} \tag{17}$$

Where, VMA is voids in mineral aggregate, G_{mb} is bulk specific gravity of compacted asphalt mixture specimen, P_s is percent of aggregate by total weight of mix, G_{sb} is bulk specific gravity of aggregate.

This calculation procedure is different from the procedure currently utilized by TDOT. TDOT utilizes following formula to calculate VMA:

$$VMA_{TDOT} = 100 - \frac{G_{mb}P_s}{G_{se}} \tag{18}$$

Where, VMA is voids in mineral aggregate, G_{mb} is bulk specific gravity of compacted asphalt mixture specimen, P_s is percent of aggregate by total weight of mix, G_{sb} is bulk specific gravity of aggregate.

The effective specific gravity of aggregates can facilitate the calculation of VMA, because it does not require a long and complicated procedure to test the bulk specific gravity of the aggregates. It is successfully utilized in Tennessee for many years by combining design experience with existing materials. However, the effective specific gravity does not allow for a precise VMA calculation. Also, the bulk specific gravity of aggregates is utilized to calculate the percent of absorbed and effective binder. Once Superpave mix design is implemented in Tennessee, it is recommended to utilize the bulk specific gravity to calculate VMA, and further study is necessary to evaluate the procedure of calculating VMA in Tennessee. The Voids filled with asphalt (VFA) is additional volumetric criterium utilized by Superpave and represents the percentage of VMA filled with asphalt binder. VFA is calculated as follows:

$$VFA = 100\left(\frac{VMA - VTM}{VMA}\right) \tag{19}$$

Where, VFA is voids filled with asphalt, VMA is voids in mineral aggregate, and VTM is air voids.

CHAPTER 4 PLANT ASPHALT MIXTURES

4.1 Introduction

A total of nine plant mixtures (three BM-2 mixes and six D-mixes) were collected at different asphalt plants across state of Tennessee. The TDOT staff helped to decide these mixtures which contain various aggregates and binders. The locations of asphalt plants are presented in Figure 4-1. Four mixtures were collected in Region 1 (one BM-2 and three D), one BM-2 mix was collected in Region 3, and in Region 4, four mixtures were collected (one BM-2 and three D). **Table 4-1** presents the summary of aggregates and asphalt binders for each of nine mixtures. The job mix formulas are included in Appendix B.

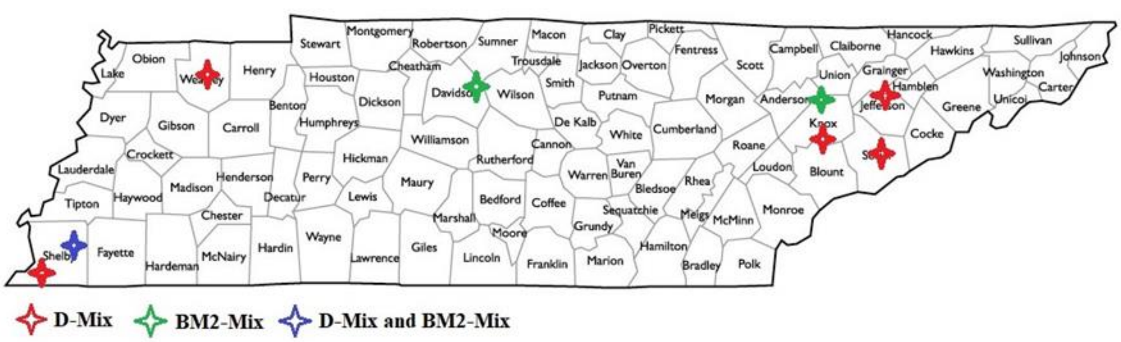


Figure 4-1. Sampling sites of plant asphalt mixtures

Table 4-1. Composition of plant hot mix asphalt

REGION 1				
No.	**TDOT No.**	**Mix Type**	**PG**	**Material**
1	1160371	411 D	64-22	Granite D-Rock

No.	TDOT No.	Mix Type	PG	Material
				Soft Limestone
				Natural Sand
				Baghouse Fines
2	1160463	411 D	76-22	Hard Limestone #7
				Slag
				Hard Limestone #10
				Soft Limestone #10
				Natural Sand
				RAP ½
3	1160307	307 BM-2	76-22	Soft Limestone #57
				Soft Limestone #7
				Soft Limestone #10
				RAP ½
4	1160315	411 D	70-22	Gravel
				Soft Limestone #10
				Natural Sand
REGION 3				
No.	**TDOT No.**	**Mix Type**	**PG**	**Material**
5	3160011	307 BM-2	64-22	Soft Limestone BM-2
				Soft Limestone #7
				Natural Sand
				RAP ¾
				RAS ⅜
REGION 4				
No.	**TDOT No.**	**Mix Type**	**PG**	**Material**
6	4160125	411 D	64-22	Gravel
				Soft Limestone
				Natural Sand
7	4160056	307 BM-2	76-22	Gravel BM-2 Rock
				Soft Limestone #57
				Natural Sand
				Soft Limestone #10
				RAP ½
				RAP 5/16
8	4160010	411 D	76-22	Gravel
				Soft Limestone
				Natural Sand
				RAP ½
				RAP 5/16

9	4160049	411 D	76-22	Gravel
				Soft Limestone
				Natural Sand
				RAP ½
				RAP 5/16

4.2 Marshall Hammer Validation

Five plant mixtures from different region of Tennessee were used to validate Marshall Hammer located at the research laboratory of the University of Tennessee, Knoxville. Different type asphalt mixtures (BM2 and D) and different materials were utilized in this validation process. Mixtures collected by TDOT Inspectors were sent to the TDOT Laboratory in Nashville and compacted utilizing Marshall compactor, 4-in molds and temperatures indicated in the Job Mix Formula. For verification, the same asphalt mixtures were compacted at the UTK Laboratory utilizing Marshall compactor, 4-in molds and temperatures indicated in the Job Mix Formula. The compacted samples were tested for bulk specific gravities (G_{mb}) and the results were compared. The summary of validation results is shown in **Table 4-2**. The test results indicated that the G_{mb} of specimens from two labs were close with G_{mb} from UTK Hammer slightly higher than that from TDOT's.

Table 4-2. The Marshall hammer validation data

Design Number	Region	Mix	Asphalt Cement	G_{mb} TDOT	G_{mb} UTK	Difference G_{mb}TDOT - G_{mb}UTK
1160307	1	307 BM2	PG 76-22	2.424	2.425	-0.001

3160011	3	307 BM2	PG 64-22	2.453	2.457	-0.004
1160463	1	411 D	PG 76-22	2.535	2.540	-0.005
1160371	1	411 D	PG 64-22	2.467	2.475	-0.008
4160125	4	411 D	PG 64-22	2.245	2.255	-0.010

4.3 Test Plan for equivalent N_{design}

Nine plant mixtures from different regions of Tennessee were collected and initially compacted with Marshall hammer at the temperature indicated in the Job Mix Formula. The same compaction effort of 75 blows on each side of the sample was applied to all the samples. At the same time theoretical maximum specific gravities were tested at the UTK Laboratory. The test results indicated that two mixtures could reach 4% of air voids using the temperature indicated in the Job Mix Formula. However, for the other seven mixtures using the temperature indicated in the Job Mix Formula could not satisfy 4% of air voids. To obtain the required compaction temperature to achieve 4% of air void content, the samples were compacted at different temperatures. The summary of corrected temperatures can be found in **Table 4-3**. The adjustment in compaction temperatures ranged from 5 °F to 15 °F.

Table 4-3. Temperature correction required to reach 4% of air voids

No	1	2	3	4	5	6	7	8	9
TDOT Design No.	1160371	1160463	1160307	1160315	3160011	4160125	4160056	4160010	4160049
Mix Type	411D	411D	307B M2	411D	307B M2	411D	307B M2	411D	411D

Compaction Temperature JMF	270°F	275°F	290°F	300°F	270°F	290°F	300°F	320°F	300°F
Compaction Temperature UTK	280°F	265°F	305°F	295°F	270°F	290°F	290°F	315°F	305°F

Once adjustments to compaction temperatures were defined, three sets of samples for each mixture were prepared. The first set of five samples were compacted in 4-in. mold with Marshall Hammer, followed by the second set of four samples compacted in 6-in. (150-mm) mold using Superpave Gyratory Compactor. The third set of three samples was compacted in 4-in. (100-mm) mold using Superpave Gyratory Compactor. All the samples in one set were compacted utilizing the same corrected temperatures. Densification curves obtained from Superpave Gyratory Compactor and Theoretical Maximum Specific Gravities were obtained. The results of air void from Marshall samples were compared to the same air void content obtained from gyratory compaction and equivalent N_{design} were back calculated. Based on this, the relation between 75 blows (each side) of Marshall Hammer and the number of gyrations of SGC at the corresponding amount of air voids was established.

4.4 Test Results of Plant Asphalt Mixtures

The back calculated equivalent N_{design} obtained at specific air void are presented in **Table 4-4** and Figure 4-2. The range of air voids obtained from Marshall compaction was from 3.71% for mixture 3 to 4.08% for mixture 7.

Table 4-4. Equivalent N_{design} for 150-mm and 100-mm molds

No	1	2	3	4	5	6	7	8	9
TDOT Design No.	1160371	1160463	1160307	1160315	3160011	4160125	4160056	4160010	4160049
Mix Type	411D	411D	307B M2	411D	307B M2	411D	307B M2	411D	411D
Marshall Air Voids	3.84	3.89	3.71	3.94	3.86	3.95	4.08	4.00	3.92
Superpave Gyrations @ air voids (150-mm mold)	77	38	75	39	39	66	45	60	54
Superpave Gyrations @ air voids (100-mm mold)	86	32	73	48	43	59	42	55	45

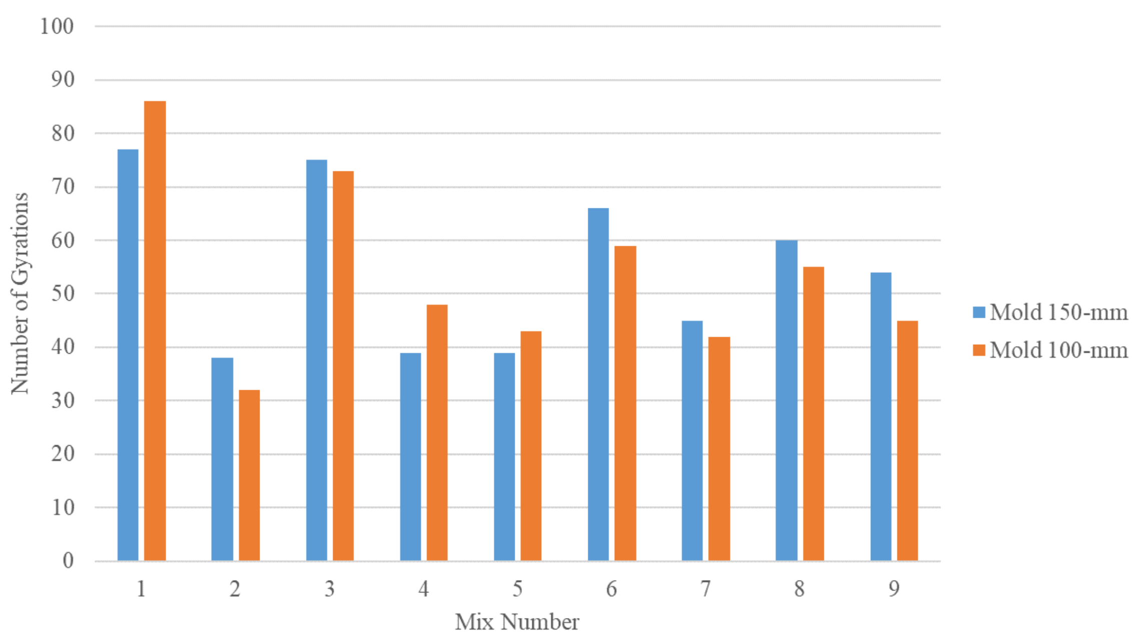


Figure 4-2. Summary of equivalent N_{design} for 100-mm and 150-mm molds.

The range of equivalent N_{design} for D mixtures compacted in 150-mm mold was from 38 to 77 gyrations, and for mixtures compacted in 100-mm mold the range was from 32 to 86 gyrations. As for BM-2 mixtures, the range of N_{design} was from 39 to 75 gyrations for 150-mm mold, and from 42 to 73 gyrations for 100-mm mold. There were three mixtures obtained higher values of equivalent N_{design} when compacted in 100-mm mold compared to 150-mm mold, including one BM2 and two D mixtures. The other six mixtures obtained higher equivalent N_{design} when compacted in 150-mm mold. The results from both molds showed a similar trend of equivalent N_{design} with a gap of 2 to 9 gyrations.

4.5 Influence of Baghouse Fines

Based on the discussion with TDOT engineers and the feedback received from project progress report, the research team investigated the aggregate gradation and asphalt content of each candidate mixture. It was indicated that plant mixtures can be affected by baghouse fines. To evaluate the influence of baghouse fines, the samples of nine plant mixtures were washed with Trichloroethylene and the sieve analysis test was performed. The purpose of this test was to estimate the variability of components (aggregates and asphalt) for each mixture compared to Job Mix Formula. The results of sieve analysis and asphalt content are presented in **Table 4-5**.

Table 4-5. Comparison between aggregate passing sieve No. 200, asphalt content of plant mixtures and the job mix formula

1160307			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	5.5	5.7	-0.2 (4%)
AC content (%)	4.45	4.5	-0.05 (1%)
3160011			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	5.6	4.7	0.9 (19%)
AC content (%)	4.39	4.2	0.19 (5%)
4160056			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	6.4	6.0	0.4 (6%)
AC content (%)	4.9	4.8	0.1 (2%)
1160315			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	6.1	5.3	0.8 (16%)
AC content (%)	6.01	5.8	0.21 (4%)
1160371			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	4.9	4.7	0.2 (4%)
AC content (%)	5.61	5.7	-0.09 (2%)
1160463			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	6.2	5.1	1.1 (22%)
AC content (%)	5.92	5.7	0.22 (4%)
4160010			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	6.0	5.6	0.4 (7%)
AC content (%)	5.96	5.9	0.06 (1%)
4160049			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	6.6	6.0	0.6 (10%)
AC content (%)	5.92	5.8	0.12 (2%)
4160125			
	Plant Mix	JMF	Difference
Passing No. 200 (%)	5.7	5.5	0.2 (4%)
AC content (%)	5.9	5.9	0.0 (0%)

Five mixtures (1160307, 1160371, 4160010, 4160056 and 4160125) had results close to the job mix formula regarding asphalt content and sieve No. 200 passing (no more than 10% difference), and four mixtures (1160315, 1160463, 3160011 and 4160049) had a difference greater than 10% when compared to the job mix formula. The difference in asphalt content and the percent passing sieve No. 200 could affect the results of equivalent N_{design} obtained in this study. Therefore, it is necessary to repeat the equivalent N_{design} validation process using laboratory mixtures.

4.6 Conclusions

In this section a total of nine plant mixtures (three BM-2 mixes and six D-mixes) were collected at different asphalt plants in Tennessee for the equivalent N_{design} validation. The Marshall Hammer devices were compared first between the TDOT Laboratory and the UTK Laboratory, and the test results indicated that the G_{mb} of specimens from two labs were close. By using the corrected temperature, the air void from Marshall samples were compared to the same air void content obtained from gyratory compaction and equivalent N_{design} were back calculated for nine plant mixtures. Based on the findings on baghouse fines, it is necessary to repeat the equivalent N_{design} validation process using laboratory mixtures.

CHAPTER 5 LABORATORY ASPHALT MIXTURES

5.1 Introduction

The aggregates and asphalt binders collected with plant mixtures (chapter 3) were used to prepare laboratory mixtures. As it can be seen in **Table 4-1**, the collected materials covered a wide range of aggregates such as granite, gravel, limestone, slag, natural sand, recycled materials (RAP and RAS), as well as different types of asphalt binder including PG 64-22, PG 70-22, and PG 76-22.

5.2 Gradation and Specific Gravity of Aggregates

As one of the most important property of aggregates, gradation is a distribution of different particle sizes represented as percentage of total weight, and it is the first step of the mix design process. Gradation is obtained by sieve analysis following ASTM C136 and ASTM C117 standards. ASTM C136 is utilized to perform dry sieve analysis and ASTM C117 is required to determine the percent passing sieve No. 200 by washing aggregates.

Currently, TDOT utilizes the grading tables for each type of mixes. Superpave introduced the less strict way to define gradation, which is call the Control Points. These points are defined in similar way as grading tables; however, they are limited only to key sieves like No. 8, No. 200, and the sieves utilized to determine the nominal maximum aggregate size. By comparing the required control points with TDOT grading tables, it can be observed that the grading D is in close relation to 12.5 mm Superpave control points and the grading BM2 is close to 25 mm. **Table 5-1** and **Table 5-2** present the comparison between TDOT grading and Superpave control point. TDOT Grading tables require a small modification to satisfy Superpave control points. For Grading D, the modification includes reducing maximum passing percentage for sieve 3/8” from 93% to 90% and increasing minimum passing from 0% to 2% for sieve No. 200. For Grading BM2, a modification is required for sieve ¾” and also it needs to decrease the maximum passing percentage from 93% to 90%.

Table 5-1. Comparison between TDOT Grading D and Superpave 12.5 mm

Sieve		TDOT Grading D		Superpave 12.5 mm		Modified TDOT	
inch	mm	Min	Max	Min	Max	Min	Max
5/8"	15.9	100	100	100*		100	100
1/2"	12.5	95	100	90	100	95	100
3/8"	9.5	80	93		90	80	**90**
No. 4	4.75	54	76			54	76
No. 8	2.36	35	57	28	58	35	57
No. 30	0.6	17	29			17	29
No. 50	0.3	10	18			10	18
No. 100	0.149	3	10			3	10
No. 200	0.075	0	6.5	2	10	**2**	6.5

Table 5-2. Comparison between TDOT Grading BM2 and Superpave 25.0 mm

Sieve		TDOT Grading BM2		Superpave 25.0 mm		Modified TDOT	
inch	mm	Min	Max	Min	Max	Min	Max
1¼"	31.8	100	100	100*		100	100

3/4"	19	81	93		90	81	**90**
3/8"	9.5	57	73			57	73
No. 4	4.75	40	56			40	56
No. 8	2.36	28	43	19	45	28	43
No. 30	0.6	13	25			13	25
No. 50	0.3	9	19			9	19
No. 100	0.149	6	10			6	10
No. 200	0.075	2.5	6.5	1	7	2.5	6.5

Based on the implementations of the Superpave mix design in other states, it can be suggested that TDOT can choose any of the three options for aggregate gradation: 1) keeping current grading table, 2) implementing Superpave control points, or 3) making small modification to TDOT grading tables.

Table 5-3 presents the specification for consensus properties of aggregates. For most of the values the requirements increase as the predicted traffic increases. Many states modified the consensus aggregate specifications based on experience with local materials. TDOT has general specifications (**Table 5-4**) that cover most of the consensus and source properties included in Superpave mix design. TDOT specifications are divided into two groups as surface and base mixtures rather than by predicted traffic like Superpave. TDOT does not specify sand equivalent limits but includes other properties that limit clay content in the aggregates like maximum 5% passing sieve No. 200 for natural sand or maximum 0.5% of clay lumps. Also, the fine aggregate angularity is not included in current TDOT specifications. The comparison between **Table 5-3** and **Table 5-4** indicate that TDOT has lower requirements for the minimum of one and two fractured faces, and the maximum for flat and elongated particles is double comparing to Superpave.

Table 5-3. Superpave Consensus Properties of Aggregates

Traffic million ESALS	Coarse Aggregate Angularity (minimum) Depth from Surface (mm)		Fine Aggregate Angularity (minimum) Depth from Surface (mm)		Flat and Elongated Particles (maximum)	Sand Equivalent (minimum)
	<100	>100	<100	>100		
< 0.3	55/--	--/--	--	--	--	40
0.3 - 3.0	75/--	50/--	40	40	10	40
3.0 - 10.0	85/80	60/--	45	40	10	45
10.0 - 30.0	95/90	80/75	45	40	10	45
>30.0	100/100	100/100	45	40	10	50

Table 5-4. **TDOT Aggregate Specifications**

Test	Surface Mix	Base Mix
One Fractured Face	40% Minimum	40% Minimum
Two Fractured Faces	70% Minimum	70% Minimum
Flat and Elongated Particles	20% Maximum	20% Maximum
Resistance to Sodium Sulfate	9%/12% Maximum	9%/12% Maximum
Clay Lumps	0.5% Maximum	0.5% Maximum
Los Angeles Abrasion	40% Maximum	50% Maximum

Table 5-5 presents the volumetric specifications for the mixtures utilized by TDOT. It

can be observed that the specifications are based on the type of mixture rather than the expected traffic as the Superpave mix design. Both TDOT and Superpave include air void, voids in mineral aggregate, and dust to binder ratio specification. However, VMA and dust to binder ratio is calculated differently since TDOT does not utilize bulk specific gravity of aggregates. As mentioned before, TDOT VMA is calculated utilizing the effective specific gravity of aggregates and dust to binder ratio is calculated utilizing the total asphalt content instead of the effective asphalt content.

Table 5-5. **TDOT volumetric requirements**

MIX	VTM (%)	MIN. VMA (%)	Dust to binder Ratio
411D	4	14.0	0.6 - 1.2
307BM2	4	13.5	0.6 - 1.5

5.3 Determination of Equivalent N_{design} for Lab Mixtures

In this study, the following steps were followed to determine the equivalent Ndesign for laboratory mixtures.

1) Design Consideration

The Marshall Mix Designs are evaluated based on Job Mix Formulas (JMF) provided by TDOT. The design requirements established by TDOT for Marshall Mix Design are compared to Superpave guidelines. First, the sieve analysis is performed on raw materials to determine if aggregate composition fulfilled Superpave control points. Then the specific gravities of aggregates are tested. The mixing and compaction temperatures are determined based on data provided on the JMF.

2) Design procedures

Since mix designs are based on actual JMF, the optimum asphalt content from JMF is taken as the initial optimum asphalt content, then four more samples are prepared with different AC contents: two samples with higher asphalt contents (+0.5% and +1%) and two samples with lower AC contents (-0.5% and -1%). Based on that, the final optimum AC content is determined. At the same time, the samples for Superpave are prepared with the same aggregate composition and AC content.

3) Comparison of two mix design methods

The first step in comparing Marshall (AASHTO T245) and Superpave mix designs (AASHTO M323) is to review gradation. For TDOT`s D-mixes, the corresponding Superpave gradation is 12.5 mm. For BM-2 mixes the corresponding Superpave gradation is 25.0 mm. The second step is to determine equivalent N_{design} by comparing optimum asphalt content obtained from Marshall mix design to data obtained from Superpave samples, which is the number of gyrations required to obtain mixes with the same properties as those from the Marshall mix design. Based on the densification curve from Superpave gyratory compactor and different AC contents of Superpave samples, the equivalent N_{design} is determined by comparing with the Marshall samples with optimum AC content.

5.4 Results and discussion

5.4.1 Materials

The combined gradations of the mixtures are presented in Figure 5-1 (surface mixtures) and Figure 5-2 (base mixtures). These gradations were integrated from the sieve analysis of individual aggregates and followed the job mix formulas of plant mixtures. The original gradations are included in appendix B. Small modifications were applied to fulfill the control points of Superpave mix design. Therefore, the gradations of laboratory mixtures satisfied both the TDOT Specifications and Superpave control points.

Table 5-6 presents the summary of the various aggregates utilized in this research, as well as the results of specific gravities and absorption for each aggregate and the combined specific gravity and absorption for each mixture. These were the most commonly used materials in Tennessee such as granite, hard limestone, soft limestone, gravel, slag, natural sand, RAP/RAS, and baghouse fines. **Table 5-6** includes summary of specific gravities and absorptions for each material and for each size of aggregate (coarse and fine) with the sieve No. 4 as boundary. The specific gravities for the baghouse fine were tested as filler material (100% passing sieve No. 200) and calculated as apparent specific gravity with absorption 0. The dosage of baghouse fines in the mixture is 1% and its specific gravity has no significant impact on the combined specific gravity and absorption.

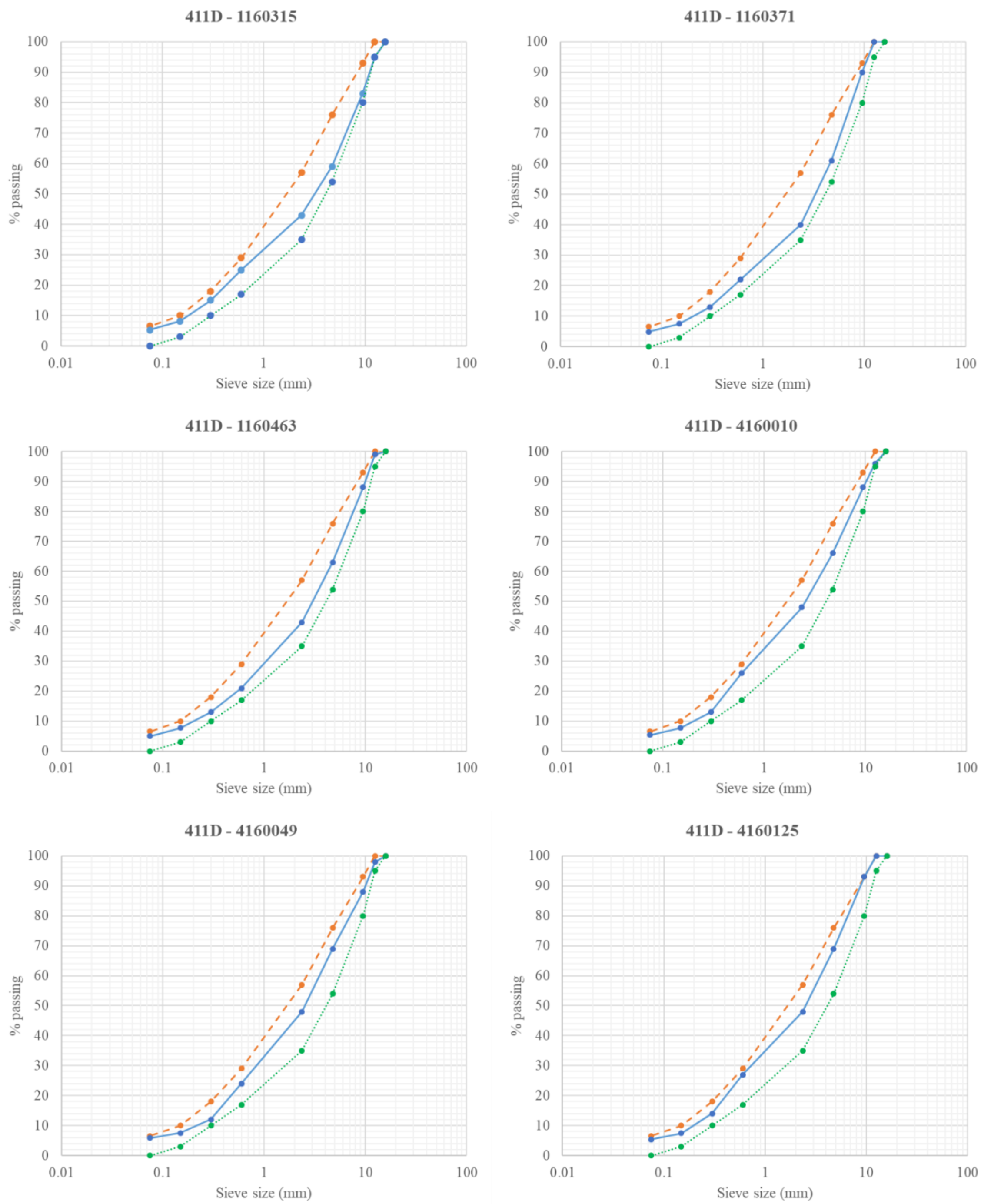


Figure 5-1 Combined gradations of the surface (D) mixtures

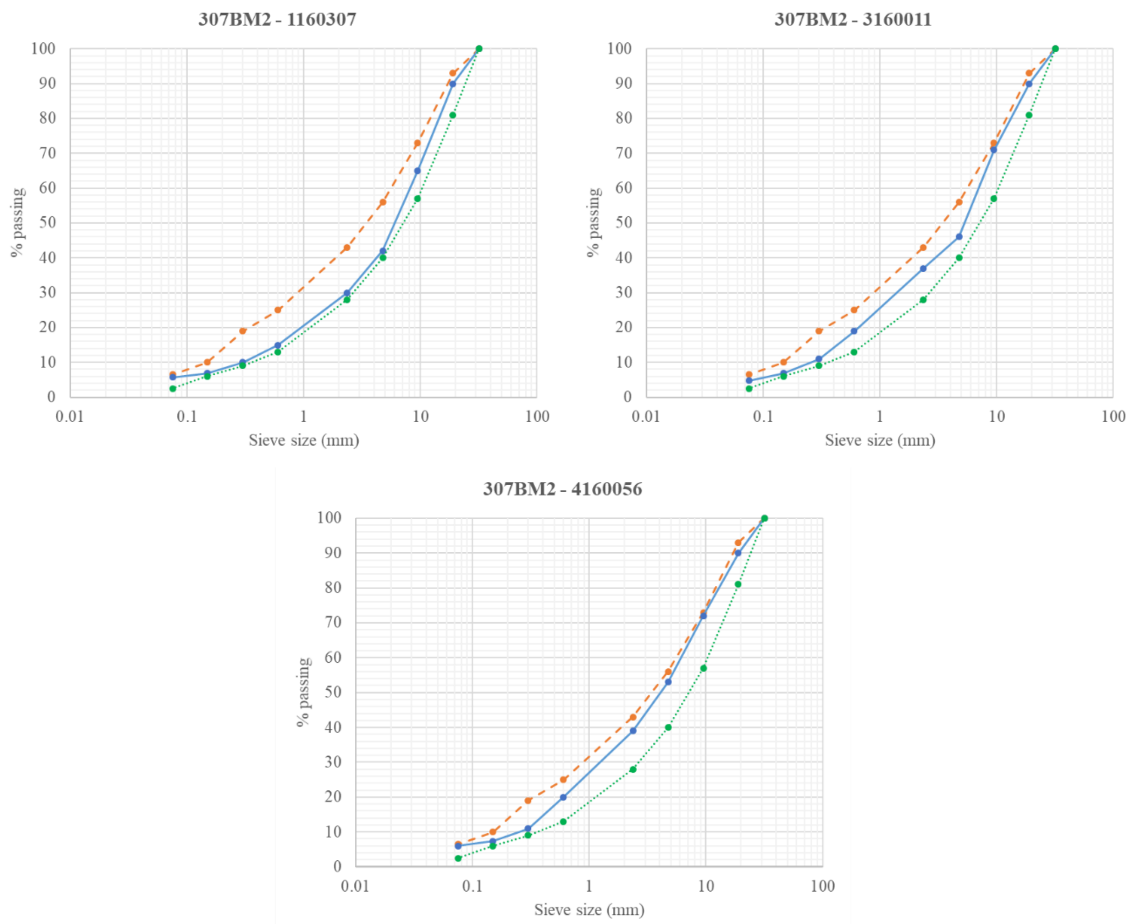


Figure 5-2 Combined gradations of the base (BM2) mixtures

Table 5-6. Summary of specific gravities and absorption for aggregates

No.	TDOT No.	Mix Type	PG	Material	Bulk Specific Gravity	Combined SG	Absorption (%)	Combined Absorption (%)
1	1160307	307 BM-2	76-22	Soft Limestone #57	2.767	2.678	2.0	2.2
				Soft Limestone #7	2.777		1.5	
				Soft Limestone #10	2.738		2.4	
				RAP ½	2.394		2.3	
2	3160011	307 BM-2	64-22	Soft Limestone BM-2	2.695	2.548	1.2	2.3
				Soft Limestone #7	2.696		2.3	
				Natural Sand	2.577		1.8	
				RAP ¾	2.309		3.2	

				RAS ⅜	1.941		4.2	
3	4160056	307 BM-2	76-22	Gravel BM-2 Rock	2.428	2.497	3.3	2.0
				Soft Limestone #57	2.641		0.9	
				Natural Sand	2.631		1.6	
				Soft Limestone #10	2.600		1.7	
				RAP ½	2.443		2.2	
				RAP 5/16	2.334		2.3	
4	1160315	411 D	70-22	Gravel	2.508	2.570	2.1	1.6
				Soft Limestone #10	2.730		0.6	
				Natural Sand	2.583		1.5	
5	1160371	411 D	64-22	Granite D-Rock	2.775	2.709	1.0	1.6
				Soft Limestone #10	2.699		2.2	
				Natural Sand	2.591		1.6	
				Baghouse Fines	*2.835**		0.0	
6	1160463	411 D	76-22	Hard Limestone #7	2.781	2.729	0.8	1.7
				Slag	2.806		2.1	
				Hard Limestone #10	2.774		1.1	
				Soft Limestone #10	2.697		1.8	
				Natural Sand	2.586		1.8	
				RAP ½	2.644		1.0	
8	4160010	411 D	76-22	Gravel	2.452	2.512	3.0	2.6
				Soft Limestone #10	2.632		0.9	
				Natural Sand	2.598		0.8	
				RAP ½	2.491		2.8	
				RAP 5/16	2.490		2.1	
9	4160049	411 D	76-22	Gravel	2.456	2.499	3.1	2.7
				Soft Limestone #10	2.600		1.7	
				Natural Sand	2.631		0.3	
				RAP ½	2.443		2.2	
				RAP 5/16	2.334		2.3	
6	4160125	411 D	64-22	Gravel	2.335	2.449	4.1	2.9
				Soft Limestone	2.591		1.6	
				Natural Sand	2.559		1.9	

5.4.2 Marshall Mix Design

Marshall Mix designs were based on the job mix formulas provided by TDOT. As a starting point, the optimum asphalt content from the job mix formula was taken and 12 samples were compacted for each mixture at different asphalt content. The mixing and compacting temperatures adopted the job mix formulas. The bulk specific gravity (G_{mb}) and theoretical maximum specific gravity (G_{mm}) were tested. For Marshall mix design, VMA were calculated utilizing bulk specific gravity of aggregates (G_{sb}) as well as utilizing TDOT standard procedure with effective specific gravity of aggregates (G_{se}). **Table 5-7** presents the summary of the Marshall mix design. It can be observed that the calculated VMA utilizing G_{sb} was lower than the calculated VMA utilizing G_{se}. The optimum asphalt contents were defined at 4% of air voids. The results of stability were generally high for the mixtures containing modified asphalts, especially for mixture 1 (1160307). Also, the flow for this mixture is very high as 29.0. The stability and flow tests were reliable for mixtures with unmodified asphalts, whereas for modified asphalts the results might be out of range.

Table 5-7. The Summary of Marshall mix designs

No.	Mix Type	Design No.	AC Type	Optimum AC (%)	VTM (%)	VMA (%)		VFA (%)	Unit Weight (pcf)	Stability (lb)	Flow (0.01")
						G_{sb}	G_{se}				
1	BM2	1160307	PG76-22	4.60	4.0	11.1	13.6	67	155.6	5770	29.0
2	BM2	3160011	PG64-22	4.45	4.0	10.9	13.5	70	149.9	2834	11.9
3	BM2	4160056	PG76-22	4.65	4.0	12.2	14.6	64	146.0	3600	12.0
4	D	1160315	PG64-22	5.60	4.0	15.2	17.1	73	144.2	2851	14.5

5	D	1160371	PG64-22	5.60	4.0	15.3	17.3	75	151.7	4170	11.3
6	D	1160463	PG76-22	5.60	4.0	15.5	17.4	74	152.8	4060	12.0
7	D	4160010	PG76-22	5.90	4.0	14.3	17.3	72	142.5	3500	14.1
8	D	4160049	PG76-22	5.90	4.0	14.1	17.2	75	143.0	3957	15.8
9	D	4160125	PG64-22	5.90	4.0	14.2	16.8	72	139.5	3125	12.7

Table 5-8 presents the optimum asphalt content results obtained in the laboratory compared to the results from the job mix formulas. The difference ranged from 0% to 0.25%. Among nine mixtures, two mixtures showed no difference in optimum asphalt content. Three mixtures had lower optimum asphalt content in the job mix formulas than obtained in the lab, while four mixtures had higher asphalt content in the job mix formulas than obtained in the lab.

Table 5-8. **Comparison of the optimum asphalt content**

Mix No.	Design No.	Mix Type	JMF Opt. AC	Lab Opt. AC	Difference
1	1160307	BM2	4.50	4.60	0.10
2	3160011	BM2	4.20	4.45	0.25
3	4160056	BM2	4.80	4.65	-0.15
4	1160315	D	5.80	5.60	-0.20
5	1160371	D	5.70	5.60	-0.10
6	1160463	D	5.70	5.60	-0.10
7	4160010	D	5.90	5.90	0.00
8	4160049	D	5.80	5.90	0.10
9	4160125	D	5.90	5.90	0.00

5.4.3 Back calculation of equivalent N_{design} in 150-mm Superpave mold

Based on the aggregate composition and binder content from the Marshall Mix Design, Superpave specimens were compacted at the same temperature. Each sample was gyrated 200 times to permit construction of complete densification curve. Figure 5-3 represents relation between percent of air void and percent of asphalt content for different number of gyrations. This figure allows determining air void or asphalt content at different compaction effort. It can be utilized as well to determine equivalent N_{design} at required air void content. Figure 5-4 to Figure 5-12 are simplified versions of Figure 5-3 that include relations between the optimum asphalt content and the compaction effort. The ranges of equivalent N_{design} were from 64 gyrations for mix 4160049 to 75 gyrations for mix 1160307. **Table 5-9** summarizes the equivalent N_{design}. The results indicate that BM2 mixes had higher average equivalent N_{design} than D mixes as 73 gyrations and 68 gyrations respectively.

Table 5-9. **Summary of equivalent N_{design}**

<table>
<tr><th>Mix No.</th><th>Design No.</th><th>Mix Type</th><th>Eqv. N_{des}</th><th>Avg. Eqv. N_{des}</th></tr>
<tr><td>1</td><td>1160307</td><td>BM2</td><td>75</td><td rowspan="3">73</td></tr>
<tr><td>2</td><td>3160011</td><td>BM2</td><td>74</td></tr>
<tr><td>3</td><td>4160056</td><td>BM2</td><td>71</td></tr>
<tr><td>4</td><td>1160315</td><td>D</td><td>65</td><td rowspan="6">68</td></tr>
<tr><td>5</td><td>1160371</td><td>D</td><td>70</td></tr>
<tr><td>6</td><td>1160463</td><td>D</td><td>72</td></tr>
<tr><td>7</td><td>4160010</td><td>D</td><td>66</td></tr>
<tr><td>8</td><td>4160049</td><td>D</td><td>64</td></tr>
<tr><td>9</td><td>4160125</td><td>D</td><td>69</td></tr>
</table>

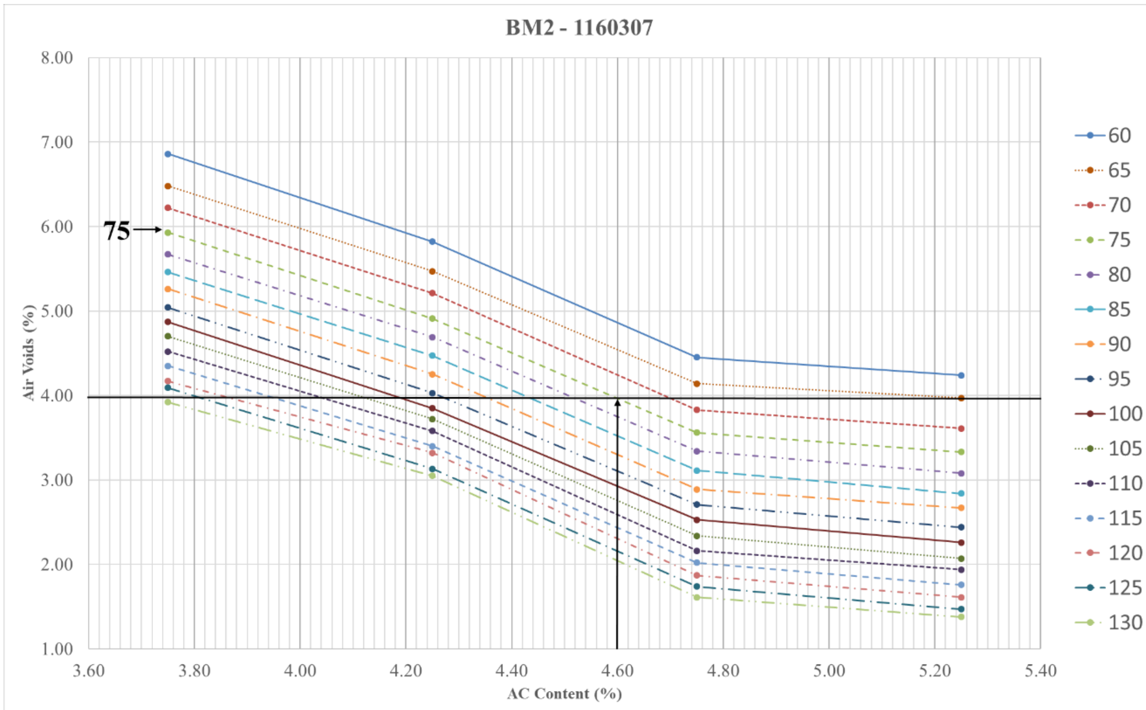


Figure 5-3 Example of back calculation of equivalent N_{design} for mixture 1160307

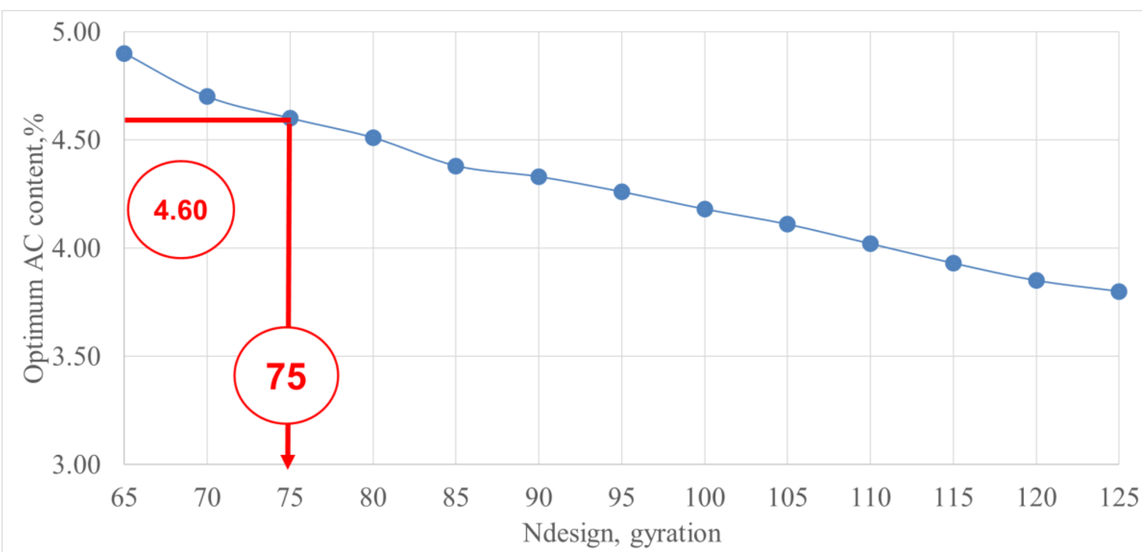


Figure 5-4 Back calculation of equivalent N_{design}, mixture 1160307

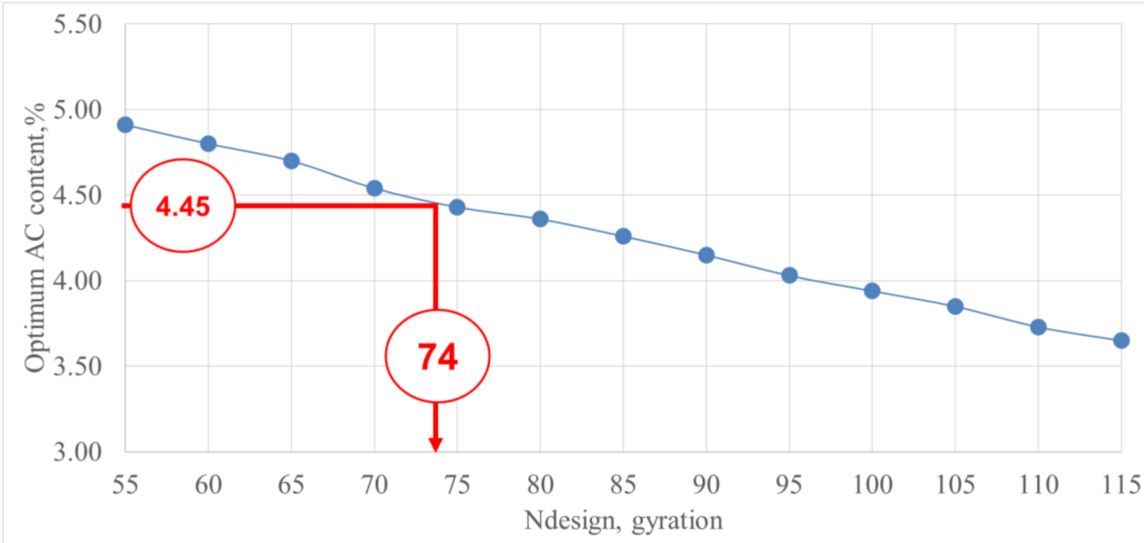


Figure 5-5 Back calculation of equivalent N_{design}, mixture 3160011

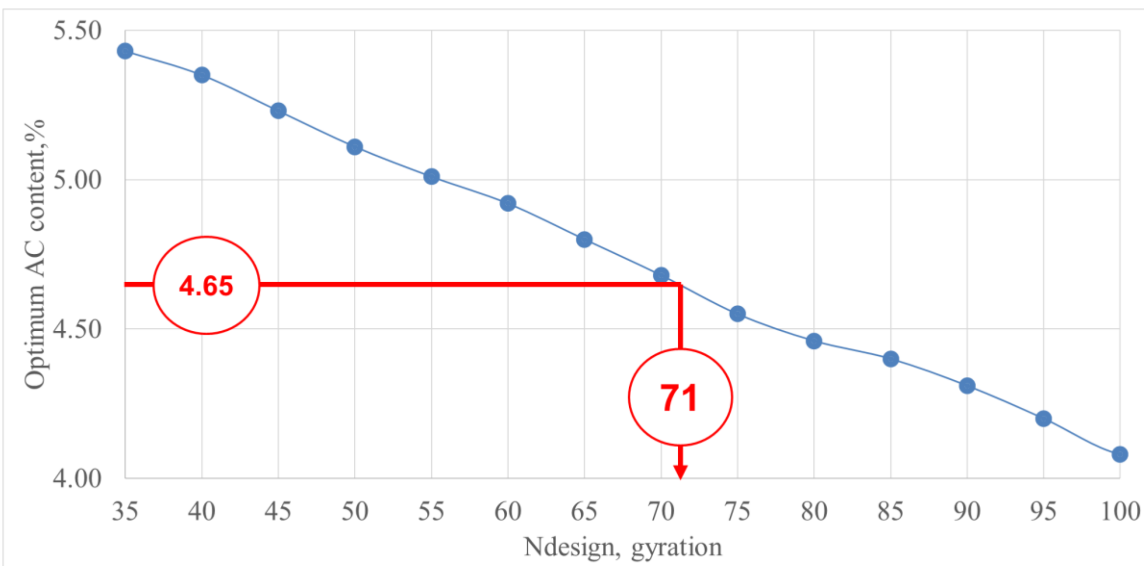


Figure 5-6 Back calculation of equivalent N_{design}, mixture 4160056

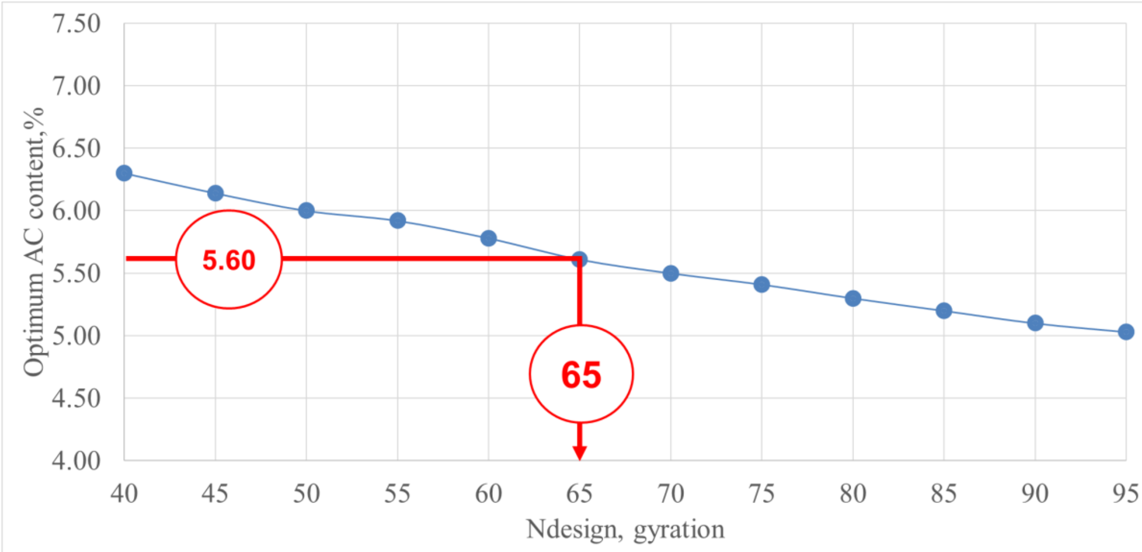


Figure 5-7 Back calculation of equivalent N_{design}, mixture 1160315

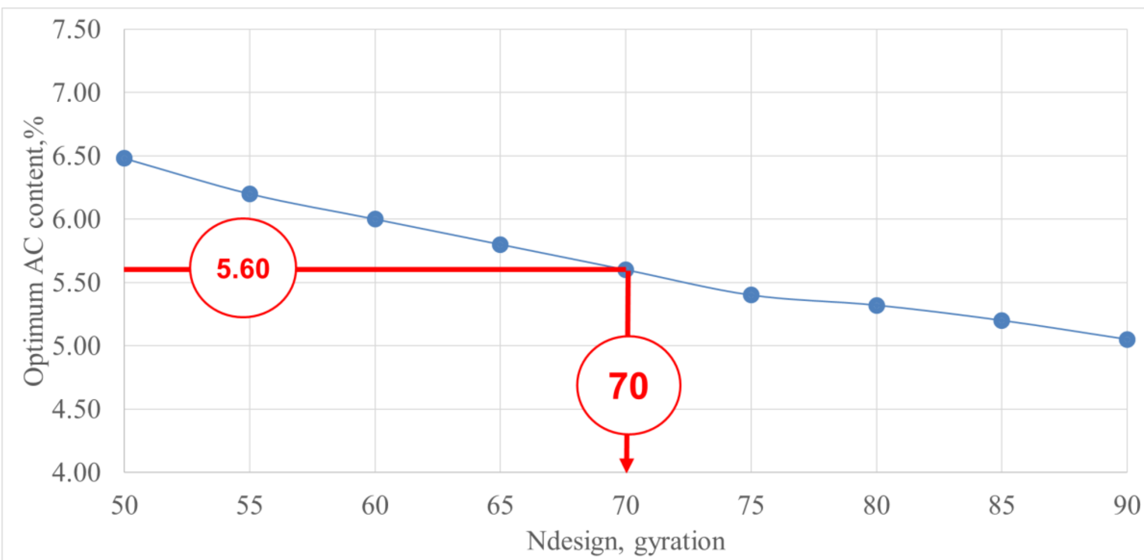


Figure 5-8 Back calculation of equivalent N_{design}, mixture 1160371

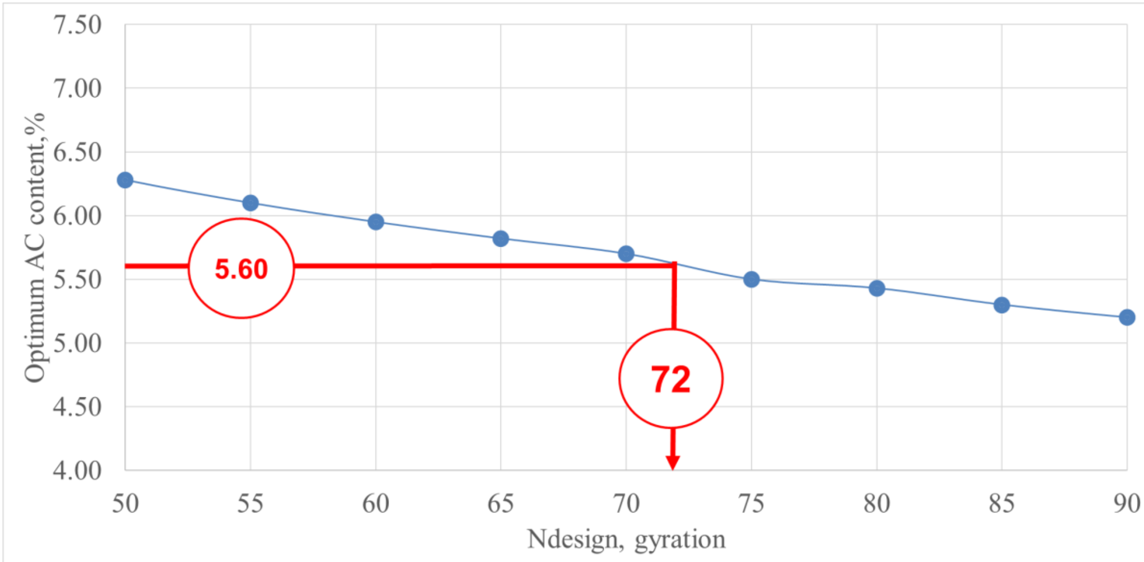


Figure 5-9 Back calculation of equivalent N_{design}, mixture 1160463

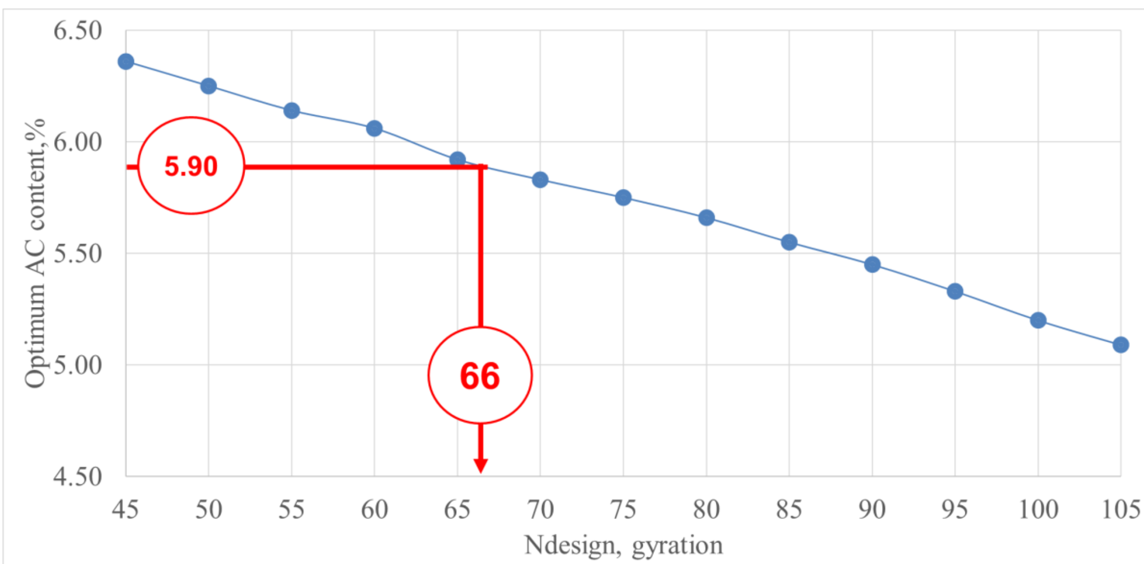


Figure 5-10 Back calculation of equivalent N_{design}, mixture 4160010

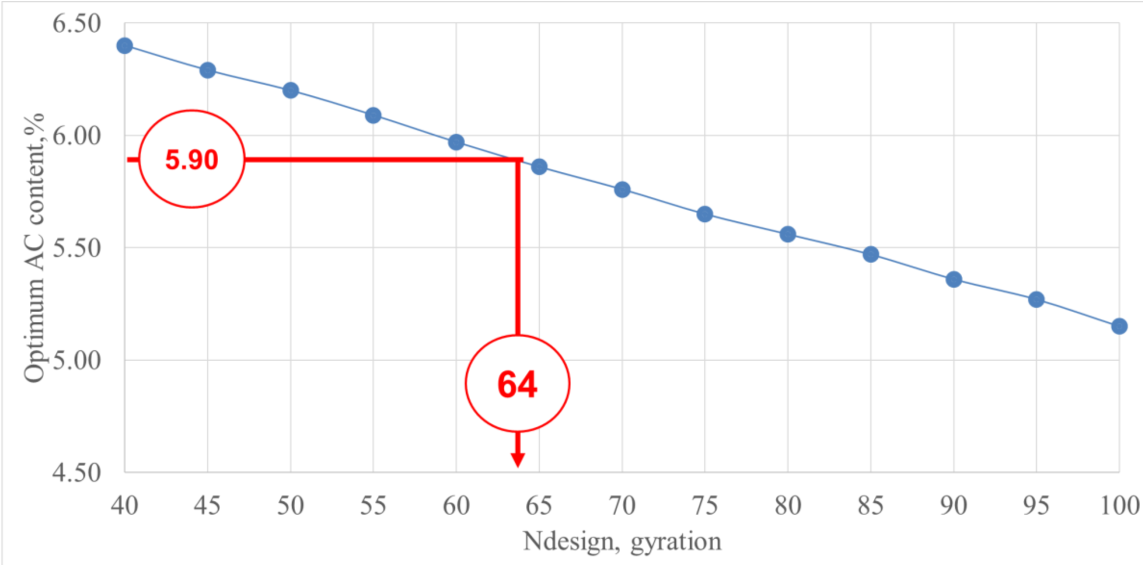


Figure 5-11 Back calculation of equivalent N_{design}, mixture 4160049

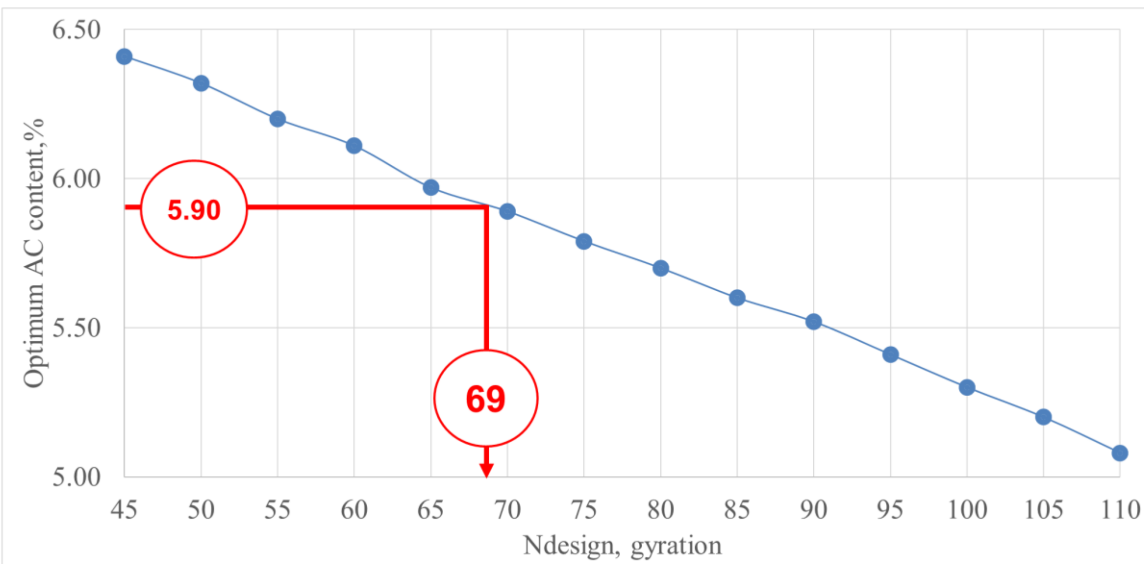


Figure 5-12 Back calculation of equivalent N_{design}, mixture 4160125

Figure 5-13 compares the equivalent N_{design} between plant and laboratory mixtures. The results of equivalent N_{design} from the laboratory mixtures were more consistent than the results from the plant mixtures. Only one mix (1160307) obtained the same results of 75 gyrations and one mix (1160371) had a lower result for laboratory mixtures. The rest of the mixtures had higher results for laboratory mixtures.

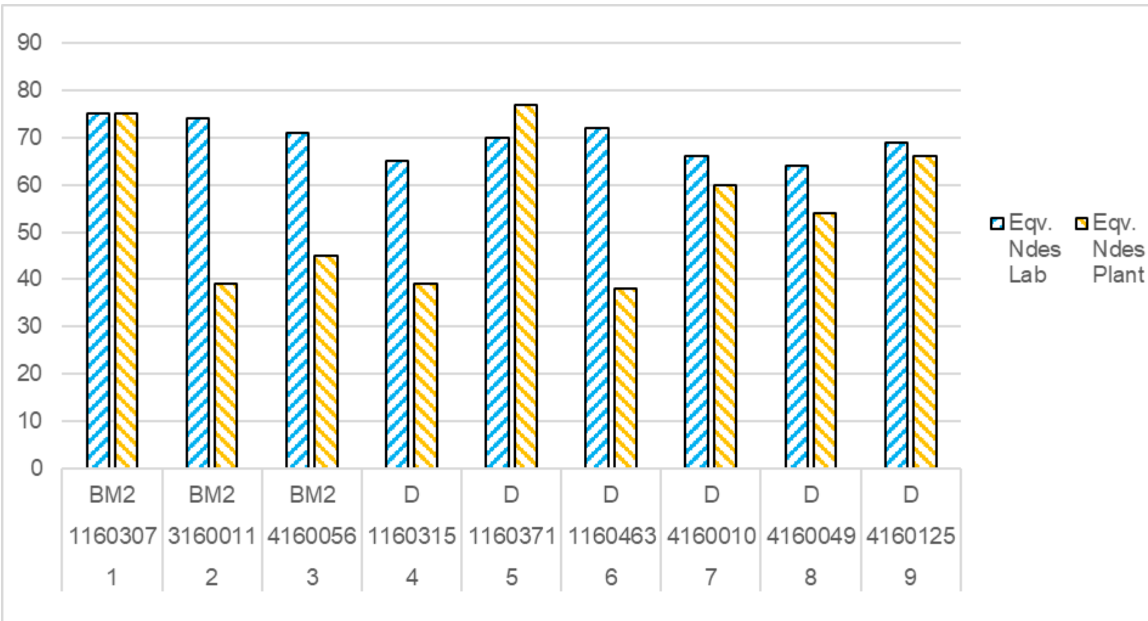


Figure 5-13 Comparison in Equivalent N_{design} between Plant and Laboratory Mixes

The results of equivalent N_{design} were close to the parameters recommended by AASHTO R35 (75 gyrations) and NCHRP 573 (65 gyrations) for twenty years design traffic of 0.3 to 30 million ESAL. Based on the results of equivalent N_{design} the following numbers of gyrations were chosen to prepare mix design for performance tests: 70 and 75 for BM2 mixes and 65 and 70 for D mixes. To determine the asphalt content at determined number of gyrations, similar graphics were utilized as for the equivalent N_{design}

determination. Figure 5-14 to Figure 5-22 presents the determination of asphalt content at different N_{design}, 70 and 75 gyrations for BM2 mixes and 65 and 70 gyrations for D mixes.

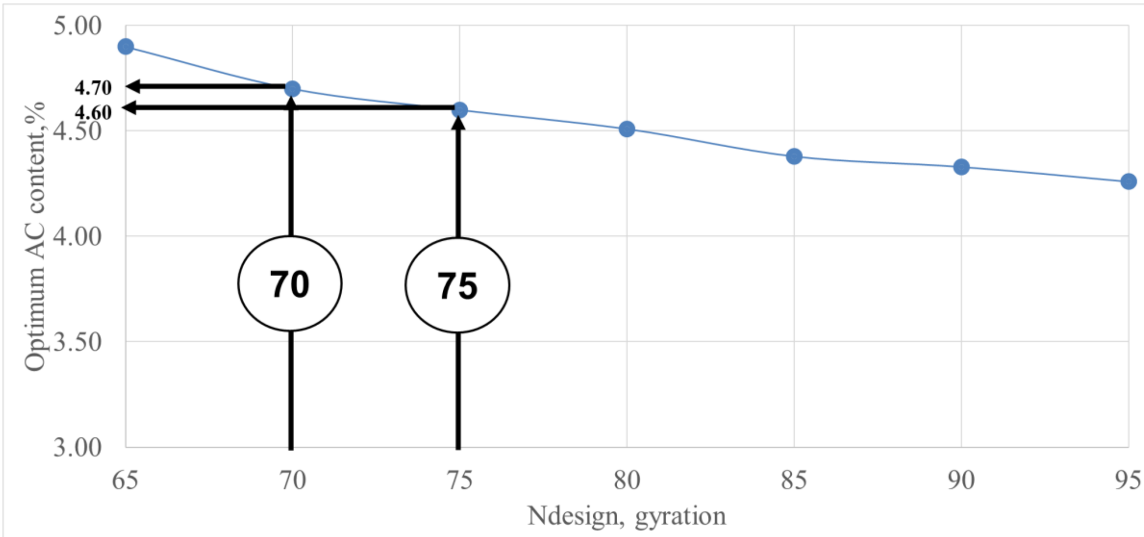


Figure 5-14 Determination of optimum asphalt content for mix 1160307 at 70 and 75 gyrations.

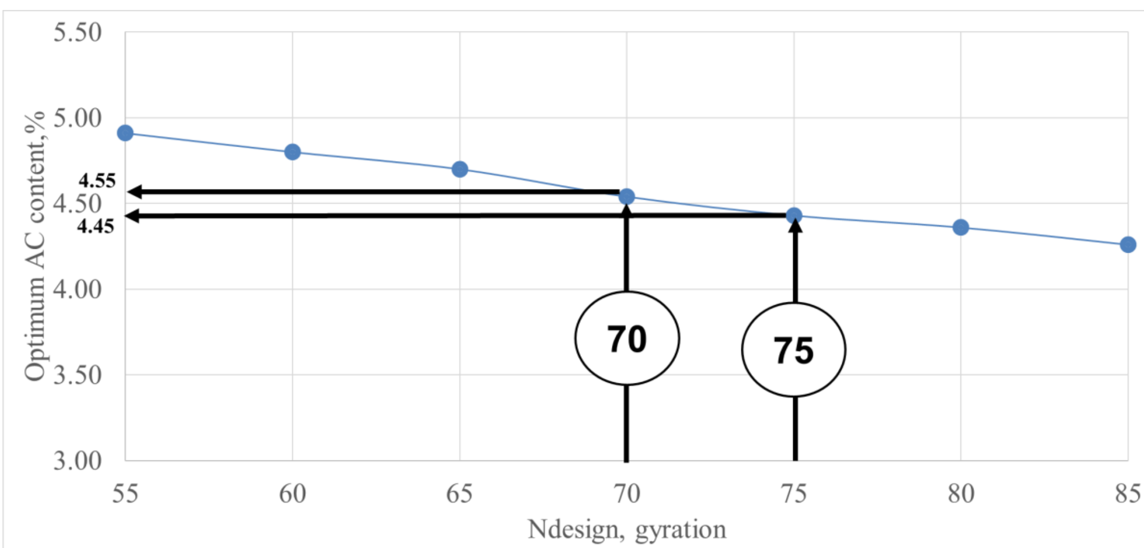


Figure 5-15 Determination of optimum asphalt content for mix 3160011 at 70 and 75 gyrations.

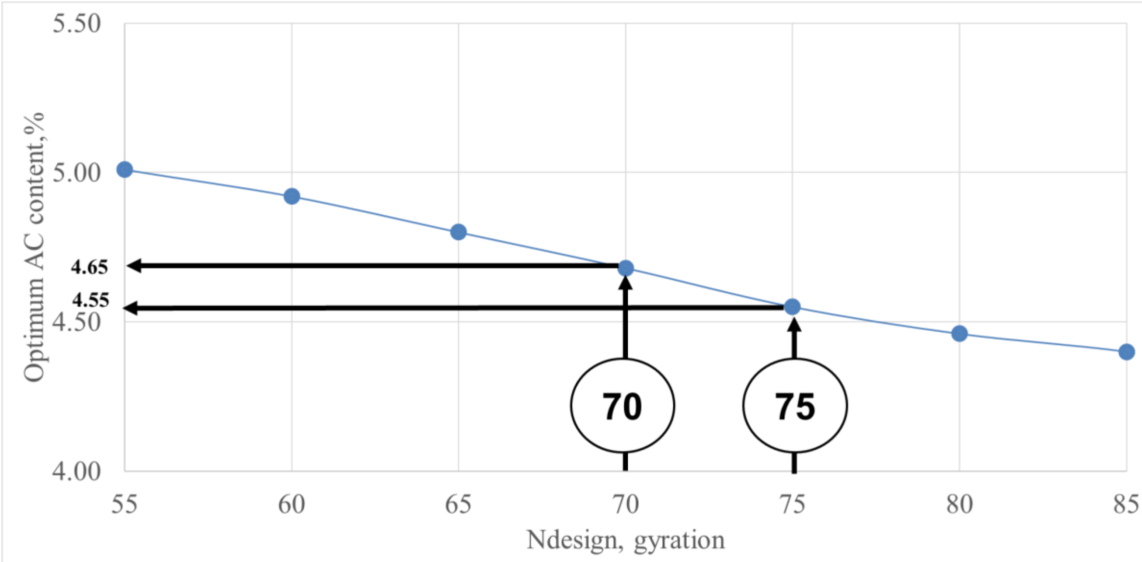


Figure 5-16 Determination of optimum asphalt content for mix 4160056 at 70 and 75 gyrations.

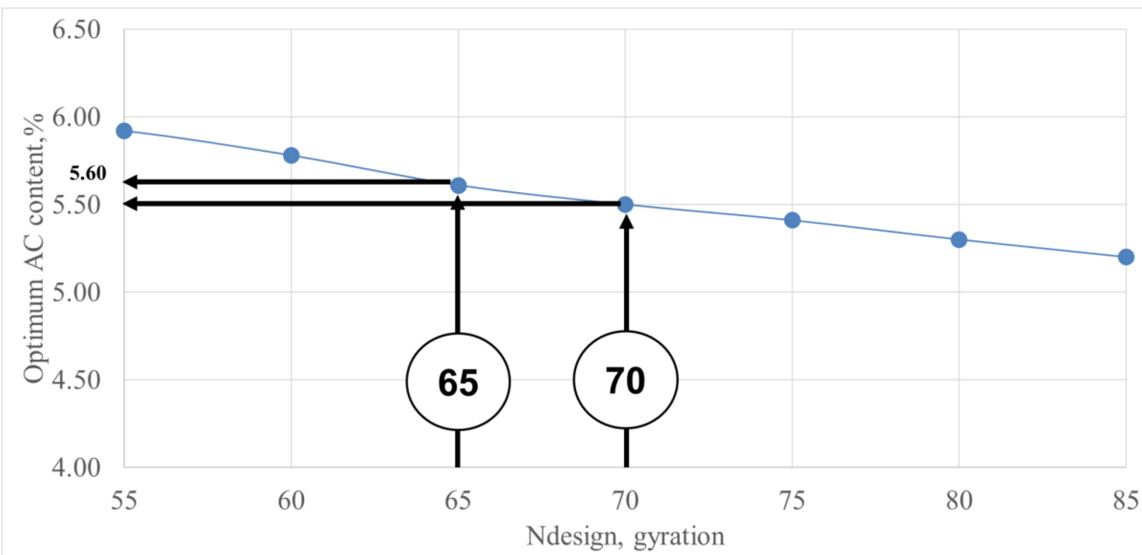


Figure 5-17 Determination of optimum asphalt content for mix 1160315 at 65 and 70 gyrations.

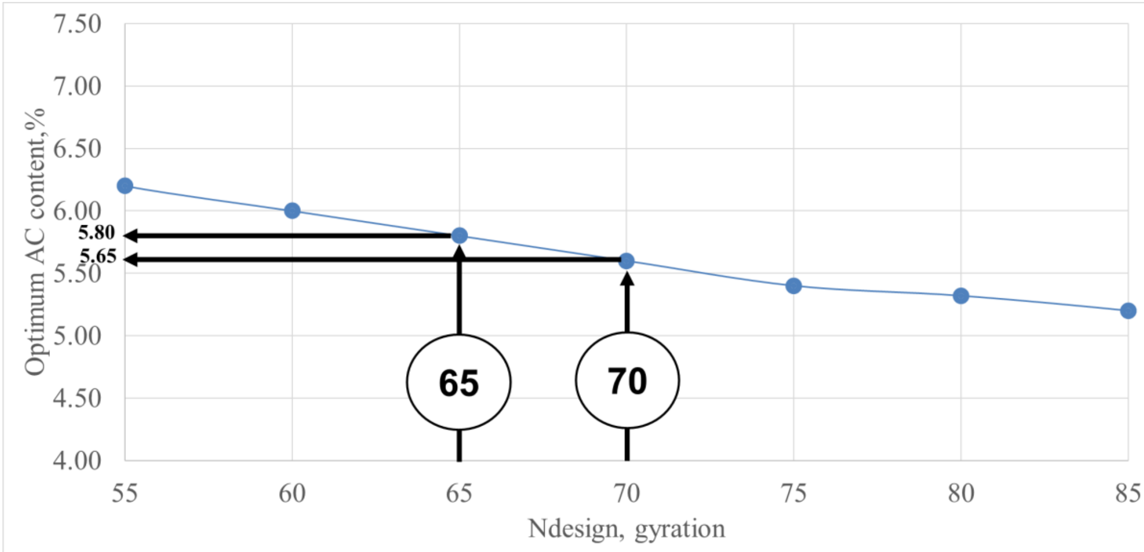


Figure 5-18 Determination of optimum asphalt content for mix 1160371 at 65 and 70 gyrations.

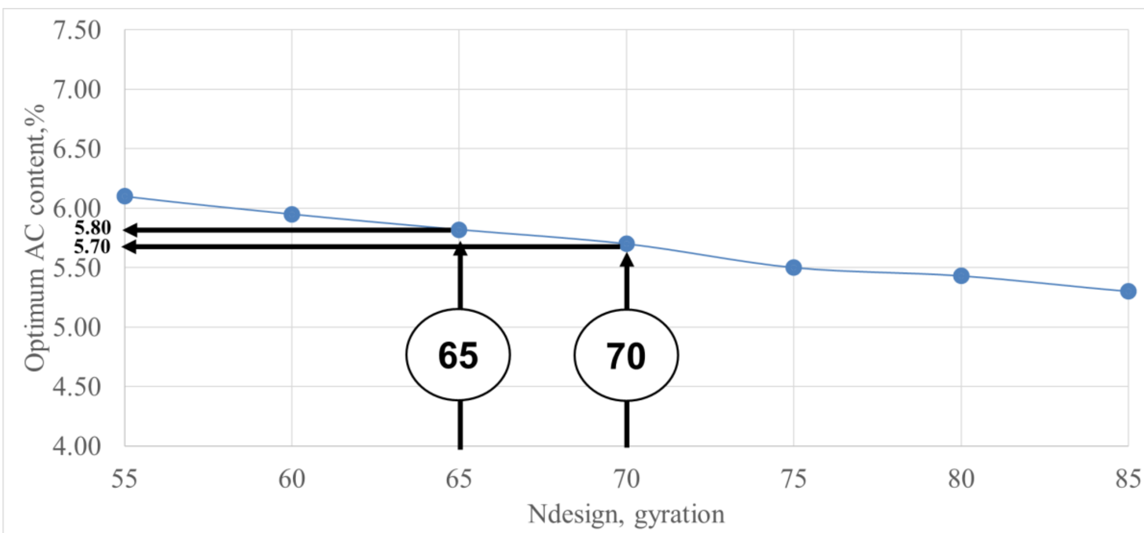


Figure 5-19 Determination of optimum asphalt content for mix 1160463 at 65 and 70 gyrations.

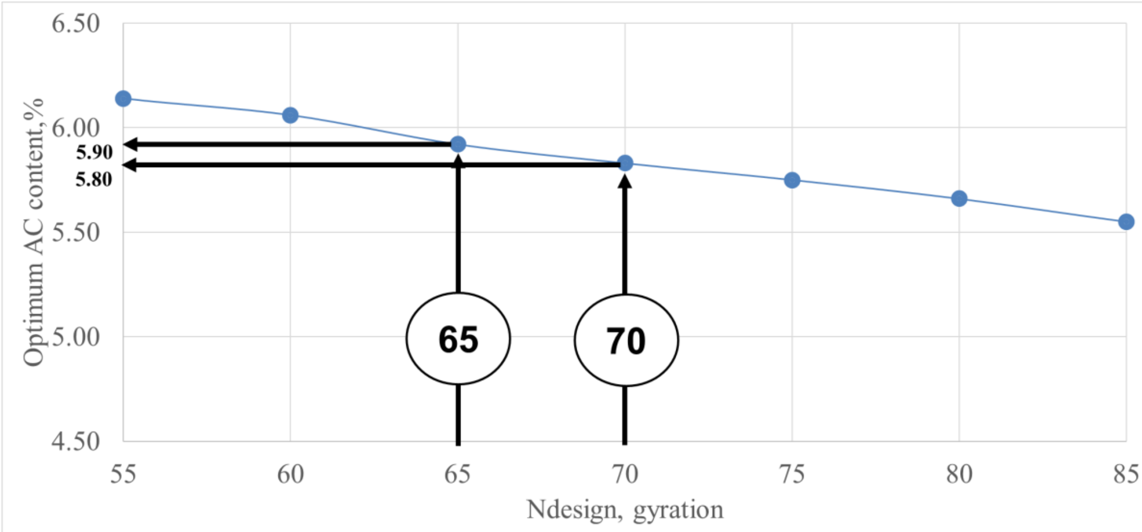


Figure 5-20 Determination of optimum asphalt content for mix 4160010 at 65 and 70 gyrations.

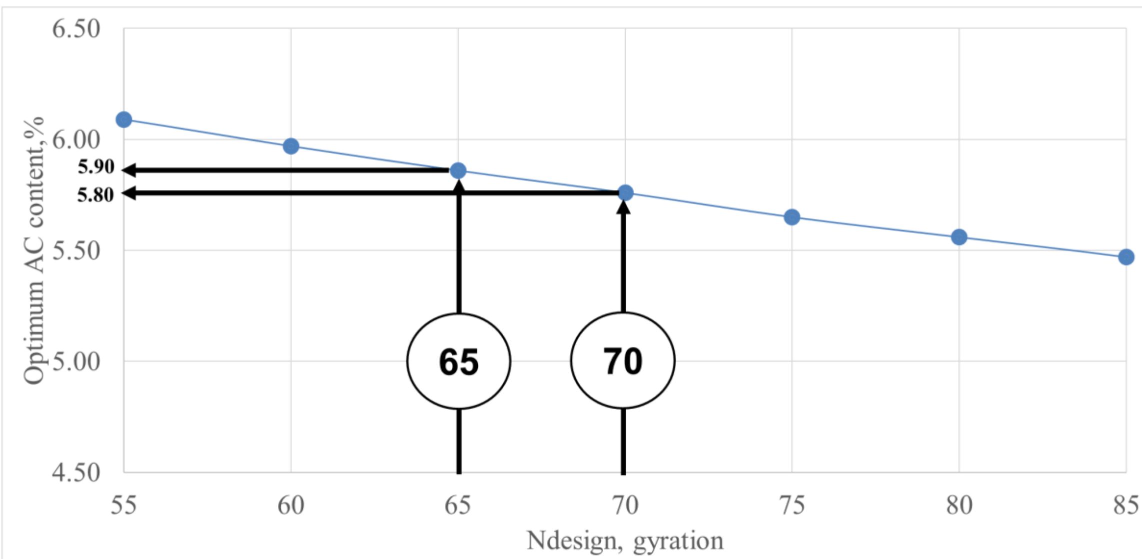


Figure 5-21 Determination of optimum asphalt content for mix 4160049 at 65 and 70 gyrations.

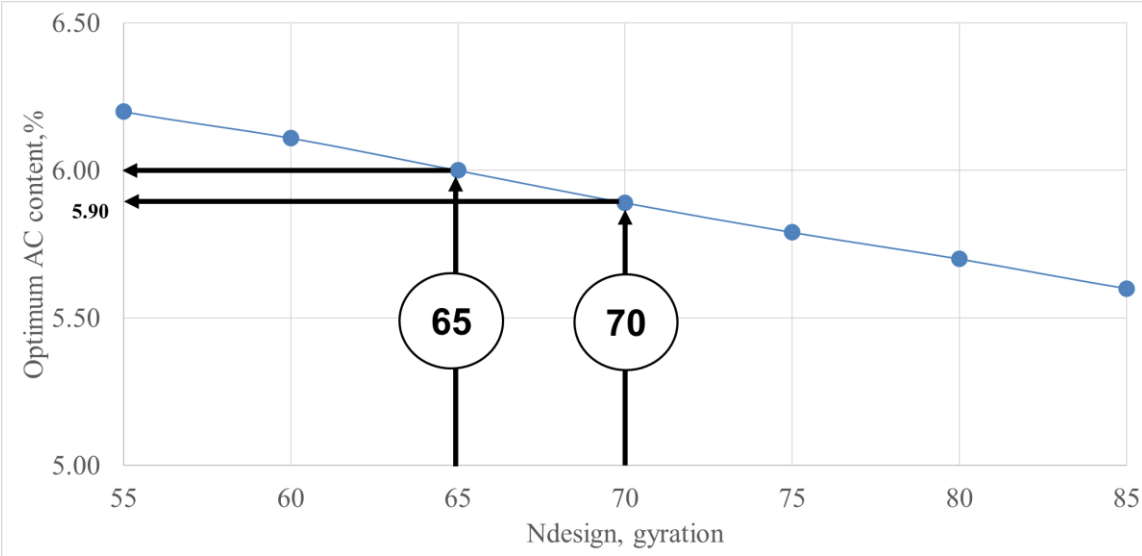


Figure 5-22 Determination of optimum asphalt content for mix 4160125 at 65 and 70 gyrations.

Table 5-10 summarizes the asphalt content for the BM2 mixtures at 70 and 75 gyrations and for D mixtures at 65 and 75 gyrations. For one D mixture 1160371, the difference in asphalt content was 0.15% and for the rest of the mixtures the difference was 0.10%. After the optimum asphalt content was determined for each mixture at two gyration numbers, the trial samples were prepared to confirm the results of air void obtained from the graphs. Three samples for each mixture were compacted at two gyrations previously determined. The bulk specific gravity of compacted specimens (G_{mb}) were determined as well as the theoretical maximum specific gravity (G_{mm}). **Table 5-11** presents the results of air void content at different gyrations. The results were all close to 4% of air voids required by Superpave. The highest deviation from 4% air voids was 0.12% for mixture 4160010. Based on the results of equivalent N_{design}, the optimum asphalt content at chosen gyrations and the revision of air voids, the specimens for performance tests were prepared.

Table 5-10. **Summary of the asphalt content at different gyrations.**

No.	TDOT No.	Mix Type	PG	N_{design}	AC cont.
1	1160307	307 BM-2	76-22	70	4.70
				75	4.60
2	3160011	307 BM-2	64-22	70	4.55
				75	4.45
3	4160056	307 BM-2	76-22	70	4.65
				75	4.55
4	1160315	411 D	70-22	65	5.60
				70	5.50
5	1160371	411 D	64-22	65	5.80
				70	5.65
6	1160463	411 D	76-22	65	5.80
				70	5.70
7	4160010	411 D	76-22	65	5.90
				70	5.80
8	4160049	411 D	76-22	65	5.90
				70	5.80
9	4160125	411 D	64-22	65	6.00
				70	5.90

Table 5-11. Summary of the air voids content at different gyrations.

No.	TDOT No.	Mix Type	PG	N_{design}	Air Void
1	1160307	307 BM-2	76-22	70	4.02
				75	4.10
2	3160011	307 BM-2	64-22	70	3.94
				75	4.01
3	4160056	307 BM-2	76-22	70	3.89
				75	3.92
4	1160315	411 D	70-22	65	4.11
				70	3.99
5	1160371	411 D	64-22	65	3.93
				70	4.07

6	1160463	411 D	76-22	65	4.00
				70	4.11
7	4160010	411 D	76-22	65	3.88
				70	3.95
8	4160049	411 D	76-22	65	4.05
				70	4.08
9	4160125	411 D	64-22	65	4.06
				70	3.98

5.4.4 Back calculation of equivalent Ndesign in 100-mm Superpave mold

The objective of utilizing different mold size was to analyze if the smaller 100-mm diameter mold can be used to design Tennessee surface mixtures (smaller than 1" maximum aggregate size). Previous research efforts focused on whether the differences in mold size affected the compaction and properties of the asphalt mixture. Hall et al. (1996) investigated how the size of the sample affected the compaction and volumetric properties of asphalt mixtures by using the same 150 mm mold for all the samples. The weight of the samples varied from 2000 g to 6500 g. Hall concluded that the compaction characteristics and volumetric properties change with the size of the sample. McGennis et al. (1996) presented the hypothesis that for the same mixture and the same number of gyrations, the resulting compaction should be the same for both molds. Two 12.7 mm (½") and five 19.1 mm (¾") nominal maximum size aggregate mixtures were tested. Specimens were prepared at the optimum asphalt content in the 150-mm and the 100-mm gyratory molds. However, more than half of the results rejected this hypothesis. Jackson and Czor (2003) evaluated the use of the 100-mm mold to prepare test samples in the laboratory by collecting different mixes and comparing relative density obtained with the 100-mm mold and the 150-mm mold. This work found the significant statistical differences between results obtained from two sizes of molds.

In this current study, the specimens compacted in 100-mm mold had the same aggregate and binder properties as samples compacted with the Marshall Hammer and 150-mm SGC molds. For four surface mixtures (1160463, 1160315, 4160125, and 4160049) the same procedure was applied as the equivalent N_{design} calculation for 150-mm mold. Figure 5-23 to Figure 5-26 show the back calculation of equivalent N_{design} for 100-mm mold. The summary of the equivalent N_{design} for 100-mm mold is presented in **Table 5-12**.

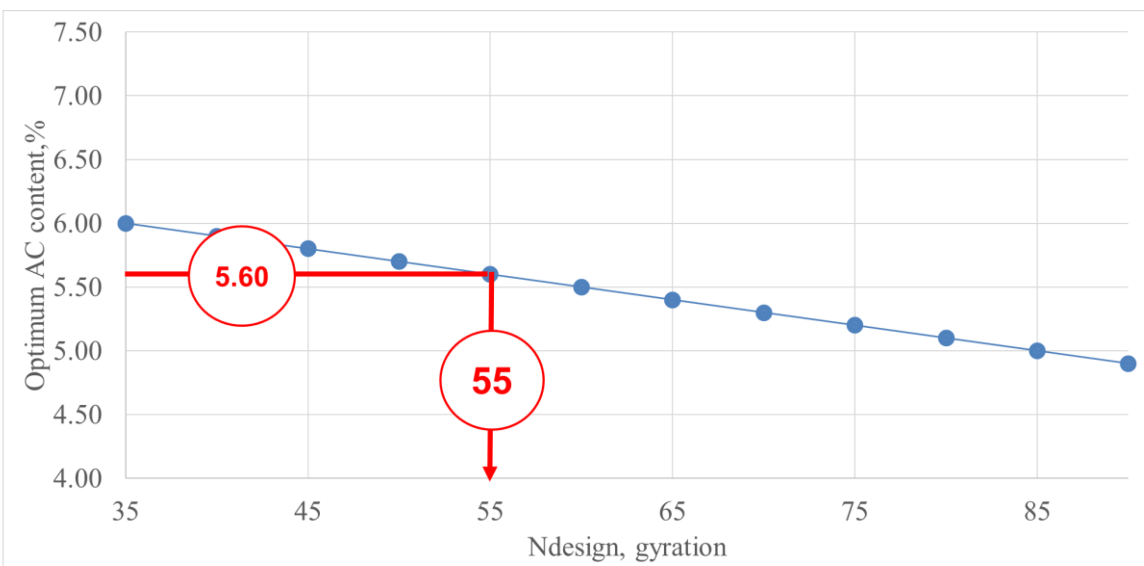


Figure 5-23 Back calculation of equivalent N_{design}, mixture 1160463, 100-mm mold

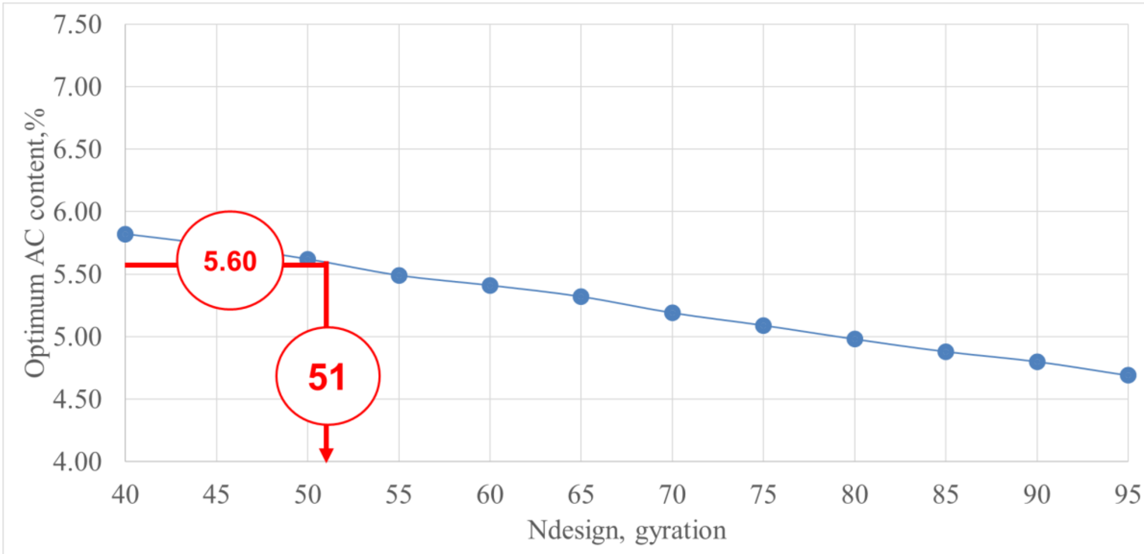


Figure 5-24 Back calculation of equivalent N_{design}, mixture 1160315, 100-mm mold

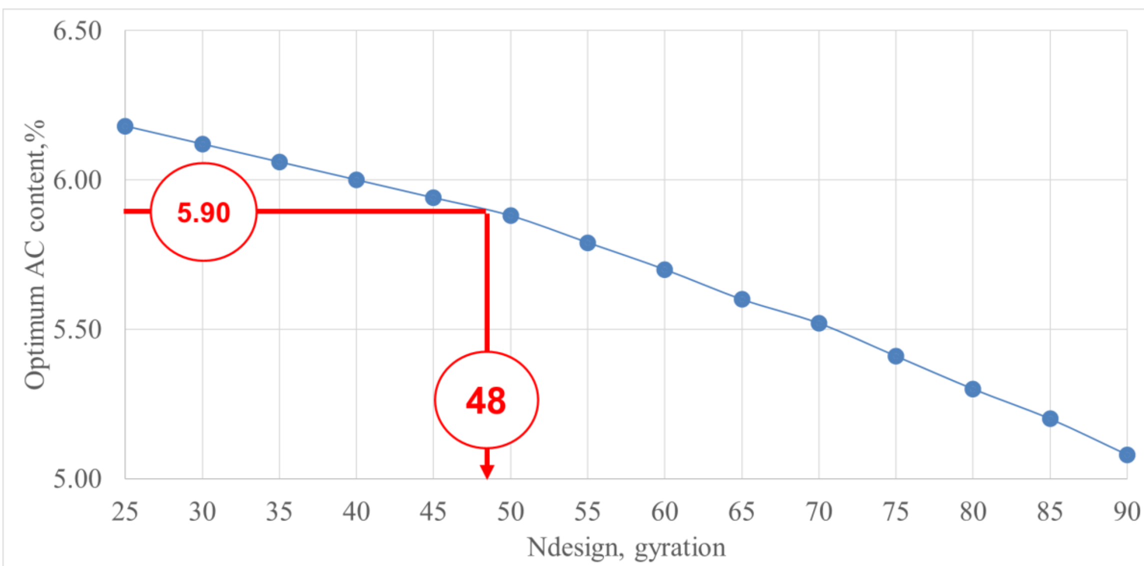


Figure 5-25 Back calculation of equivalent N_{design}, mixture 4160125, 100-mm mold

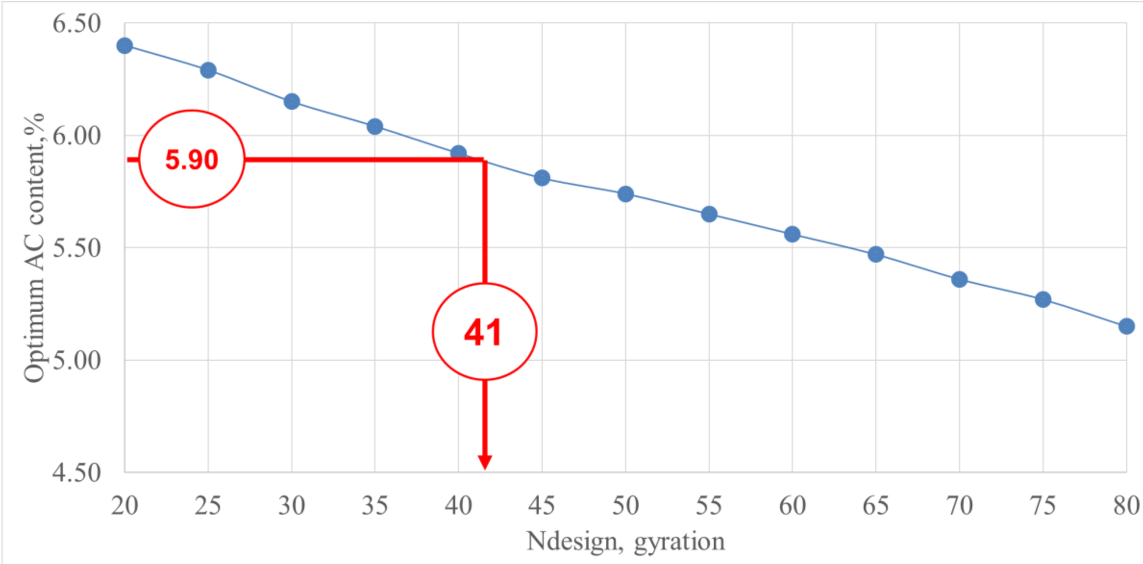


Figure 5-26 Back calculation of equivalent N_{design}, mixture 4160125, 100-mm mold

Table 5-12. Summary of the equivalent N_{design} for 100-mm mold

D-Mix - 100 mm mold				
1160463	1160315	4160125	4160049	**Average**
55	51	48	41	**49**

From **Table 5-12** it can be observed that the equivalent N_{design} for the 100-mm molds was considerably lower than that for the 150-mm mold. The average equivalent N_{design} for 100-mm mold was 49 gyration and for 150-mm mold was 68 gyrations, which resulted in a difference of 19 gyrations. Also, the range of the results was greater for 100-mm mold from 55 gyrations to 41 gyrations, when for 150-mm mold the range was from 72 gyrations to 64 gyrations. This study was mainly focused on the standard Superpave procedure involving the 150-mm mold. Therefore, the results obtained from 100-mm mold were limited in the number of mixtures and samples. Further studies might include the change in applied force, gyration angle, and gyration speed.

5.4.5 Voids in Mineral Aggregate (VMA)

Voids in Mineral Aggregate (VMA) is the volume of voids in compacted mixture and is represented as percentage of total volume of the mixture. The VMA consists of effective asphalt and air voids, as the ingredients of asphalt mixture that are not aggregates and absorbed asphalt (Chadbourn et al. 1999). Figure 5-27presents the graphic representation of VMA.

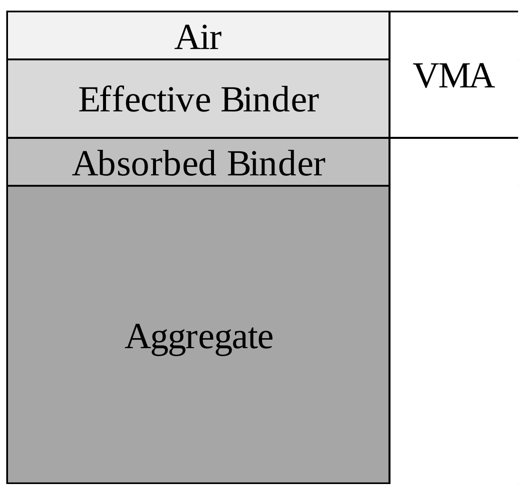


Figure 5-27 Representation of VMA

Table 5-13 summarizes some of the known factors that might affect the VMA. It can be noticed that the VMA can be affected at every stage of asphalt mix service life, starting from aggregate characteristics, production and handling, to hauling time and paving temperature of the mixture. When most of the states utilize the bulk specific gravity of aggregates to calculate the VMA, TDOT uses the effective specific gravity of aggregates. Both methods have the advantages and the disadvantages. The advantage of utilizing bulk specific gravity is the precision of the VMA calculation, while the disadvantage is the complicated and time-consuming process of testing the bulk specific gravity, especially for fine aggregates. The advantage of effective specific gravity is the time required to obtain the effective specific gravity from the maximum theoretical specific gravity, while the disadvantage is a lack in knowledge of aggregate absorption and the percentage of absorbed and effective binder in the mixture. The long experience and relatively unchanged aggregates permit TDOT to utilize the effective specific gravity to determine VMA.

Table 5-13. Summary of the factors that affect the VMA (Chadbourn et al. 1999).

Factor	**Effect on VMA**
Aggregate Gradation	Dense gradations decrease VMA
Aggregate Texture	Smooth or polished aggregates decrease VMA
Aggregate Shape	Less angular aggregates decrease VMA
Asphalt Absorption	Increased absorption results in lower VMA
Dust Content	Higher dust contents increase surface area, decrease film thickness and tend to lower VMA
Baghouse Fines/ Generation of Dust	Increase surface area, decrease film thickness and tend to lower VMA
Plant Production Temperature	Higher plant production temperature decreases asphalt binder temperature, results in more asphalt absorption and lower VMA
Temperature during Paving	Higher temperature during paving create soft mixtures, lower air voids, and lower VMA
Hauling Time	Longer hauling time allow increased absorption, and lower VMA

Aggregate Handling	More steps in aggregate handling increases potential for aggregate degradation, resulting in increase in fines and lower VMA

In the process of migration from the Marshall mix design to the Superpave mix design, it is recommended a statewide implementation of the bulk specific gravity testing for VMA calculations. The Superpave will be a new method in Tennessee, where most of the mix designers have long experience with existing materials and they are used to utilizing effective specific gravity to calculate VMA. When preparing the Superpave mix design, the material knowledge might be not enough to correctly estimate the absorbed and effective binder. It can lead to several problems caused by incorrect determination of asphalt absorption such as error in calculations of the VFA, VMA and air voids, which might result in a mixture lacking stability or durability. Stripping, cracking, or raveling can be caused by insufficient effective binder. Construction problems such as tender mixtures and segregations may also happen. The absorbed asphalt binder may wary between the lab and the field due to extended mixing, storage, hauling or variation in temperatures during production and construction. It is important to have the precise values of the VMA during mix design, plant production and compaction. Before implementing Superpave mix design in Tennessee, it is recommended to conduct statewide workshops and interlaboratory test program to determine the precision of each laboratory that will conduct Superpave mix designs. The new specifications that indicate the minimum VMA should be established based on research program.

Table 5-14 presents the comparison of the VMA for nine mixtures utilized in this study calculated by utilizing the bulk specific gravity of aggregates (G_{sb}) and the effective specific gravity of aggregates (G_{se}). The difference between the results of VMA calculated with different specific gravities varied from 1.9 to 3.1. This significant difference can affect the decision about accepting or rejecting the mixture during mix design process or later by quality assurance during plant production.

Table 5-14. Summary of the VMA data calculated with G_{sb} and G_{se}

No.	Mix Type	Design No.	AC Type	Optimum AC (%)	VTM (%)	VMA (%)		Difference VMA (G_{se}-G_{sb})
						G_{sb}	G_{se}	
1	BM2	1160307	PG76-22	4.60	4.0	11.1	13.6	2.5
2	BM2	3160011	PG64-22	4.45	4.0	10.9	13.5	2.6
3	BM2	4160056	PG76-22	4.65	4.0	12.2	14.6	2.4
4	D	1160315	PG64-22	5.60	4.0	15.2	17.1	1.9
5	D	1160371	PG64-22	5.60	4.0	15.3	17.3	2.0
6	D	1160463	PG76-22	5.60	4.0	15.5	17.4	1.9
7	D	4160010	PG76-22	5.90	4.0	14.3	17.3	3.0
8	D	4160049	PG76-22	5.90	4.0	14.1	17.2	3.1
9	D	4160125	PG64-22	5.90	4.0	14.2	16.8	2.6

5.5 Conclusions

In this section, a wide range of aggregates from Tennessee were collected and tested and a total of nine laboratory mixtures (three BM-2 mixes and six D-mixes) were designed to determine the equivalent N_{design}. The testing results of the aggregates indicated that the gradation of TDOT`s D and BM2 mixes are close to Superpave 12.5 mm and 25 mm. Based on the implementations of the Superpave mix design in other states it can be suggested that TDOT can choose any of the three options for aggregate gradation: 1) keeping current grading table, 2) implementing Superpave control points, or 3) making small modification to TDOT grading tables.

The N_{design} for the laboratory mixes was defined based on the nine well-performing mixtures provided by TDOT. The results of back calculations showed that for BM2 mixes the range of equivalent N_{design} was from 71 to 75 gyrations (average 73 gyrations), while for D mixtures the range was from 64 to 72 gyrations (average 68 gyrations).

CHAPTER 6 LABORATORY PERFORMANCE TESTS

The following laboratory performance tests were conducted to evaluate the properties and performance of the asphalt mixtures compacted with different compaction effort:

- Tensile Strength Ratio (TSR)
- Asphalt Mixture Performance Test
- Superpave Indirect Tension (IDT) Tests
- APA Hamburg Test

6.1 Performance Test Methods

6.1.1 Tensile Strength Ratio (TSR)

The Tensile Strength Ratio test is utilized to measure the effect of water on the tensile strength of asphalt mixtures following the AASHTO T283 standard test. The moisture damage susceptibility is determined by compacting a set of samples following the job mix formula. The most common air void content for this test ranges from 6% to 8%. The samples are divided into two sets of approximately the same air void. One set is left in dry condition, while the other set is placed in the water bath and partially saturated. The tensile strength is tested utilizing tensile splitting test. The tensile strength ratio is determined by the ratio of tensile strength of the moisture conditioned set to the dry set.

For each test at least six samples should be compacted, three for dry test and three for partially saturated test. The standard specimens are of 100 mm (4 in.) diameter and 62.5 mm (2.5 in.) height. The set designated to be tested dry should be stored at room temperature. The vacuum should be applied to saturate the second set of specimens from 55 to 80%. The partially saturated samples should be placed in distilled water at 60±1°C (140±1.8°F) for 24 hours.

Test the tensile strength at 25±1°C (77±1.8°F) for both sets. The specimen is placed in loading machine and diametral load is applied at the rate of 50 mm/min (2 in/min) until the maximum load is reached (Figure 6-1). The loading should continue until sample is fractured. The moisture damage should be evaluated visually. The tensile strength is calculated as follows:

$$S_t = \frac{2000P}{\pi t D} \tag{20}$$

Where, St is tensile strength, P is a maximum load, t is height of specimen, and D is diameter of specimen.

Tensile strength ratio is calculated as follows:

$$TSR = \left(\frac{S_{tm}}{S_{td}}\right) 100 \tag{21}$$

Where, TSR is tensile strength ratio, S_{tm} is average tensile strength of moisture conditioned set, S_{td} is average tensile strength of dry set.

Figure 6-1 Testing tensile strength under diametral load

6.1.2 Asphalt Mixture Performance Test

Two types of asphalt mixture performance test, dynamic modulus test and flow number, can be conducted in Asphalt Mixture Performance Test machine (Figure 6-2). The testing procedure for dynamic modulus test is derived from NCHRP 513 Simple performance tester (SPT) for Superpave mix design. Test specimens are placed in an environmental chamber that has been set to the appropriate testing temperature ± 0.5°C. A continuous uniaxial sinusoidal (haversine) compressive stress is applied to the unconfined specimen at a specified test frequency. Three linear variable displacement transducers (LVDT) are used at 120° angles to capture deformation of the specimen during test. The applied stress and the resulting recoverable axial strain response of the specimen are measured and used to calculate the dynamic modulus and phase angle. The stress-to-strain relationship for a linear viscoelastic asphalt mixture specimen is defined by a complex number called complex modulus (E*). The absolute value of the complex modulus, |E*|, is named dynamic modulus. |E*| and calculated as follows:

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \tag{22}$$

where, σ_0, ε_0 – magnitudes of applied loading stress and induced axial strain, respectively.

Figure 6-2 Asphalt Mixture Performance Tester

In this study, the dynamic modulus test was conducted with an uniaxial haversine load inducing approximately 100 microstrain in the specimen. The test was conducted at 30, 20, and 4°C and 25, 20, 10, 5, 2, 1, 0.5, 0.2, and 0.1 Hz.

The flow number was tested on the samples previously utilized to obtain dynamic modulus. Dynamic modulus is considered the non-destructive test. The following test conditions were utilized in this study:

- Repeated axial stress: 600 kPa.
- Temperature: 54°C (129.2°F)
- Air void content: 7.0 ± 0.5%.
- Sample size: 100mm x 150mm

During the flow number test, a sample at specific temperature is tested under repeated haversine axial compressive load of 0.1 second every 1.0 second for a maximum of 10,000 cycles or until deformation of 50,000 microstrains is reached. The point in the permanent strain curve where the rate of accumulation of permanent strain reaches a minimum value has been defined as the flow number (Figure 6-3).

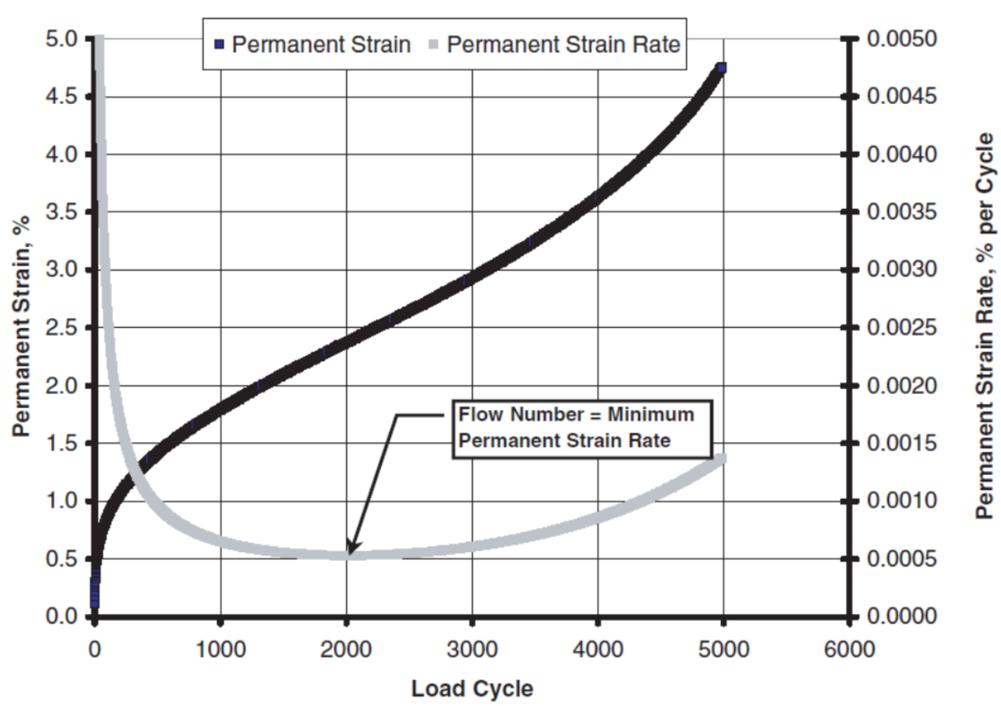


Figure 6-3 Example of flow number determination (NCHRP673)

The test can be conducted with or without confining pressure. The permanent axial strain obtained during test is measured as a function of the load cycles. The flow number is defined as the number of load cycles in relation to minimum rate of change of axial strain and can be related to resistance to permanent deformation (rutting). The flow number test should be performed on the samples that are 150-mm height and 100-mm in diameter. The final sample is cored and cut from larger sample around 170-mm in height and 150-mm in diameter. The test is performed at specific temperature and the testing chamber should equilibrate temperature for at least 1 hour. Follow the AMPT software to start the test and when the test is finished the AMPT will unload itself. The calculation of the flow number is performed by the AMPT software for every specimen. The average should be calculated by the operator.

6.1.3 Superpave Indirect Tension (IDT) Tests

The Superpave IDT tests consist of resilient modulus (M_R), creep compliance, and indirect tensile (IDT) strength tests. For these tests, strain gages are used to obtain vertical and horizontal strain readings. Four strain gages are placed on a sample with the aid of brass gage points which are glued onto the sample prior to testing (NCHRP 530). All tests are conducted at 25˚C. In this study, resilient modulus and indirect tensile strength tests are conducted. Figure 6-4 shows the setup of the Superpave IDT tests.

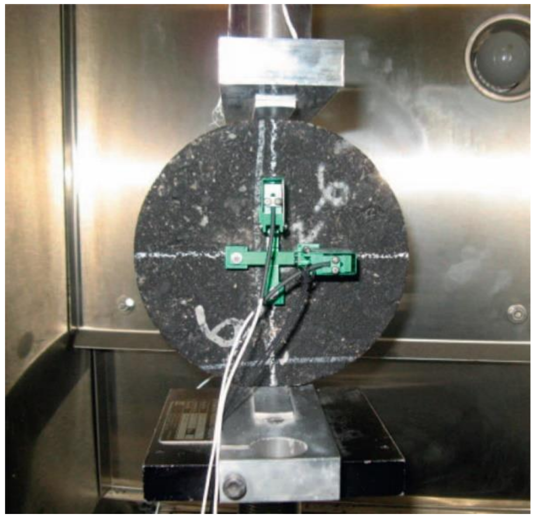


Figure 6-4 Setup of Superpave IDT Tests

Resilient Modulus Test

The resilient Modulus test is conducted by applying a repeated peak-load resulting in horizontal deformations within the range of 200–300 microstrain. Each load cycle consists of 0.1-s load application followed by a 0.9-s rest period. The load and deformation are continuously recorded, and resilient modulus is calculated as follows:

$$M_R = \frac{P \times GL}{\Delta H \times t \times D \times C_{cmpl}} \tag{23}$$

where, M_R - resilient modulus, P - maximum load, GL = gage length, ΔH - horizontal deformation, t - thickness of specimen, D - diameter of specimen, C_{cmpl}- nondimensional creep compliance factor, $C_{cmpl} = 0.6354(X/Y)^{-1} - 0.332$, (X/Y) = ratio of horizontal to vertical deformation.

IDT Strength Test

The same sample tested for resilient modulus is used in the IDT strength test. The IDT strength test is conducted to determine tensile strength and strain at failure of an asphalt mixture. Samples are monotonically loaded to failure along the vertical diametric axis at the constant rate of 3 in/min. During testing the load and deformation are continuously recorded. Maximum load carried by the sample is determined and used to calculate the indirect tensile stress at failure as follows:

$$S_t = \frac{2 \times P \times C_{sx}}{\pi \times t \times D} \tag{24}$$

where S_t - indirect tensile strength, P - failure load, C_{sx} - horizontal stress correction factor, $C_{sx} = 0.948 - 0.01114 \times (t/D) - 0.2693 \times \nu + 1.436 \times (t/D) \times \nu$, ν = Poisson's ratio, $\nu = -0.1 + 1.480 \times (X/Y)^2 - 0.778 \times (t/D)^2 \times (X/Y)^2$, t, D, (X/Y) - same as described above.

Dissipated Creep Strain Energy Threshold (DCSEf)

With the stress-strain response from the IDT strength test, the dissipated creep strain energy threshold (DCSE$_f$) is determined as follows (Figure 6-5):

$$DCSE_f = FE - EE \tag{25}$$

where, FE - fracture energy; it is defined as the area under the stress strain curve to the failure strain ε_f, and EE - elastic energy.

$$FE = \int_0^{\varepsilon_f} S(\varepsilon) d\varepsilon \tag{26}$$

$$EE = \frac{1}{2} S_t (\varepsilon_f - \varepsilon_0) \tag{27}$$

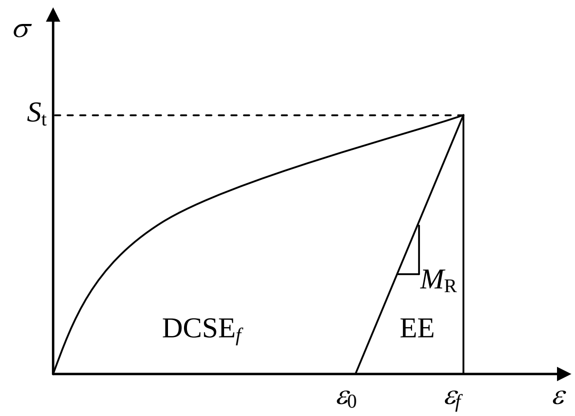


Figure 6-5 Determination of Creep Strain Energy Threshold (DCSE$_f$)

6.1.4 Asphalt Pavement Analyzer Hamburg Test

The latest asphalt pavement analyzer (APA) is capable of running Hamburg wheel tracking test (AASHTO T324) on asphalt samples (Figure 6-6 and Figure 6-7). This test is conducted by rolling a steel wheel, 47 mm wide by 204 mm diameter, across the surface of a sample (150 mm x 75 mm, Figure 6-8) that is submerged in water at 50°C with the load of 685 N (154 lb). The specimens are loaded until the maximum rut value is reached (12 mm), or the maximum number of cycle (10,000) is reached. The stripping inflection point can be determined from the graph of rut depths versus number of cycles. This point defines the number of passes at which moisture damage starts to affect the asphalt mixture. The higher is the stripping inflection point the less likely is the asphalt mixture to strip and to be damaged by moisture.

Figure 6-6 APA Hamburg Test

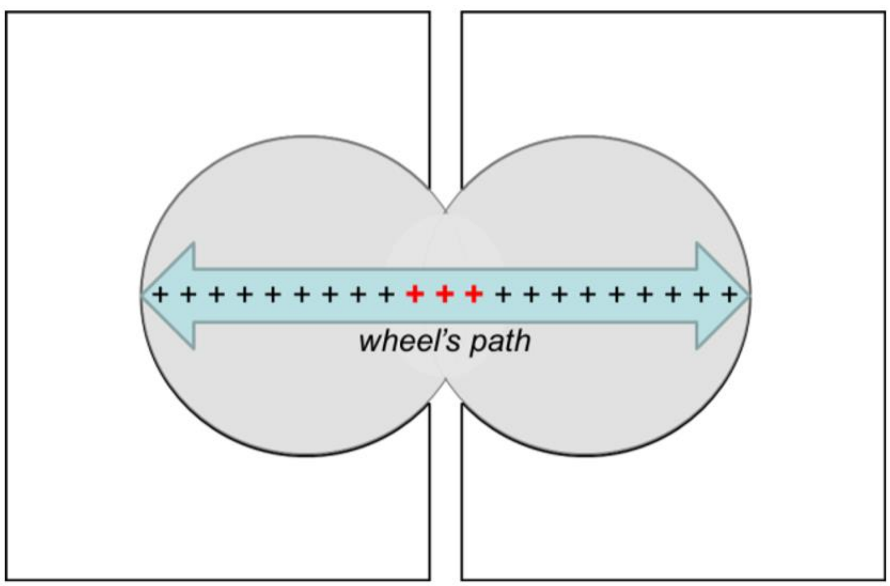


Figure 6-7 Schematic of Rut Depth Measurement Points

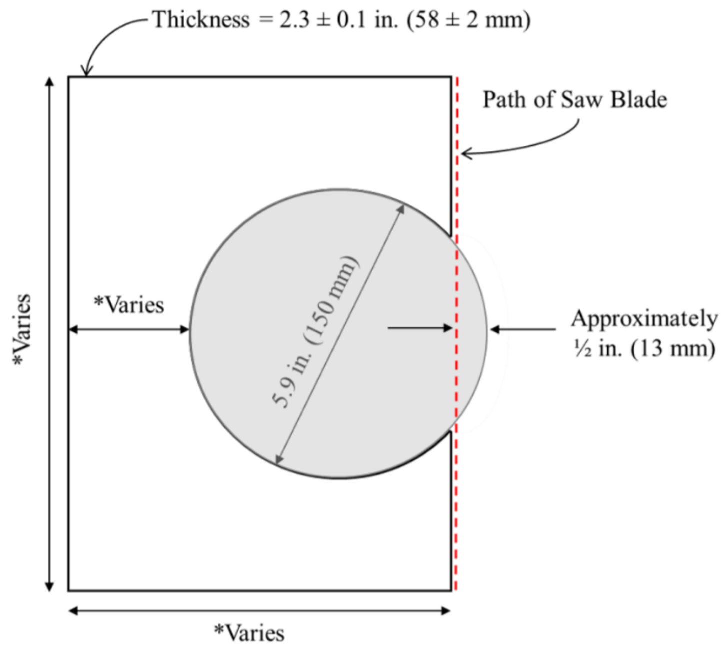


Figure 6-8 Sample cutting setup

6.2 Results and discussion

6.2.1 Tensile Strength Ratio (TSR)

A total of 18 different mixtures were utilized (each of 9 mixtures were prepared at two different N_{design}). The samples were compacted utilizing 100-mm Superpave Gyratory Compactor molds. Initially, a set of trail samples were compacted for each mixture to define the number of gyrations that provides required air void content. Then six specimens for each mixture were compacted with the previously defined number of gyrations. The bulk specific gravity was determined to confirm that the air voids are within the established parameters. Half of the samples were placed in a vacuum chamber for a short time to obtain required degree of saturation. The rest of the samples (dry) were stored at room temperature.

The partially saturated specimens were in moisture conditions by submerging in the water at 60°C for 24 hours. The tensile strength was determined by placing specimens into loading apparatus and the load was applied until maximum load was reached. The summary of the results is presented in **Table 6-1**.

Table 6-1. Summary of the results of tensile strength ratio

No.	TDOT No.	Mix Type	PG	Effective SG	N_{design}	Asphalt Content (%)	TSR (%)
1	1160307	307 BM-2	76-22	2.802	70	4.70	80.3
					75	4.60	79.2
2	3160011	307 BM-2	64-22	2.767	70	4.55	83.5
					75	4.45	79.9
3	4160056	307 BM-2	76-22	2.609	70	4.65	82.8
					75	4.55	81.2
4	1160315	411 D	70-22	2.633	65	5.60	90.0
					70	5.70	89.7
5	1160371	411 D	64-22	2.768	65	5.80	90.4
					70	5.65	88.5
6	1160463	411 D	76-22	2.800	65	5.80	88.6
					70	5.70	85.4
7	4160010	411 D	76-22	2.594	65	5.90	92.7
					70	5.80	91.6
8	4160049	411 D	76-22	2.585	65	5.90	92.1
					70	5.80	91.8
9	4160125	411 D	64-22	2.530	65	6.00	94.6
					70	5.90	92.7

For all the mixtures included in this study, a higher N_{design} yielded lower asphalt binder content and slightly lower moisture resistance. D-mixes had significantly higher moisture resistance (average 91%) than BM-2 mixes (81%). Mixtures 1160307 and 3160011 designed with N_{design} equal to 75 gyrations were outside the TDOT specifications for moisture damage set as 80% minimum.

6.2.2 Asphalt Mixture Performance Test

The results of dynamic modulus are presents in Appendix A. It can be seen that the dynamic modulus of asphalt mixtures followed the general trend as a viscoelastic material. The dynamic modulus decreased with the increase in temperature but increased with the increase in the loading frequency. The phase angle increased with increasing temperature but decreased with increasing loading frequency. There was no significant difference in the results obtained with different N_{design}.

Figure 6-9 and Figure 6-10 present the results of the flow number for D-mixtures and BM-2 mixtures respectively. It can be noticed that the results were highly influenced by the type of asphalt binder. Mixtures that contained PG76-22 binder obtained much higher values of flow number than mixtures with PG64-22 and PG70-22. The difference between PG64-22 and PG70-22 was not as big as the difference with PG76-22. These results indicate that a higher grade of asphalt binder should provide better rut resistance.

The N_{design} influenced the flow number results. For all the mixtures utilized in this study, the flow number increased as N_{design} increased, but the different mixtures had a different magnitude of change in the flow number. A small change in flow number (9 cycles) was recorded for mixture 1. This mixture compacted with N_{design}=65 gyration obtained a flow number of 324 and for N_{design}=70, the flow number was 333. A more pronounced change (398 cycles) in the flow number was observed for mixture 8. For N_{design}=65, the flow number was 2587, and for N_{design}=70, the flow number was 2985.

The results of this study indicate that a higher N_{design} will provide better rut resistance of the mixture. These results were compared to the cracking resistance tests obtained from Superpave IDT and the best combination of rut and cracking resistance were used to determine the recommended N_{design}.

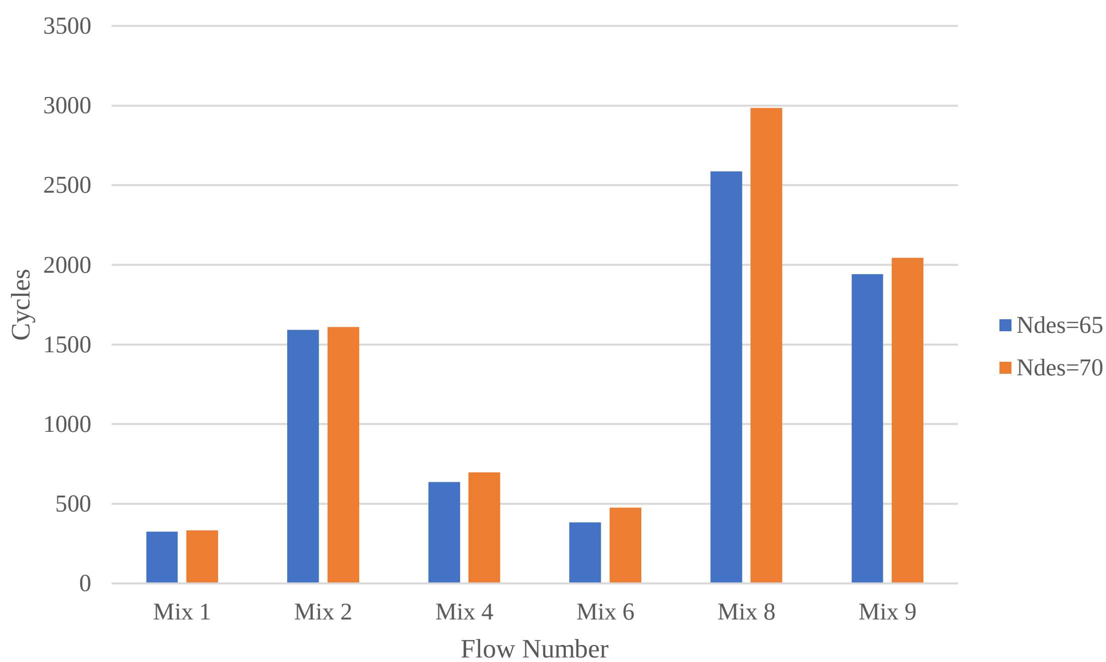


Figure 6-9 Comparison of Flow Number results for D-mixes.

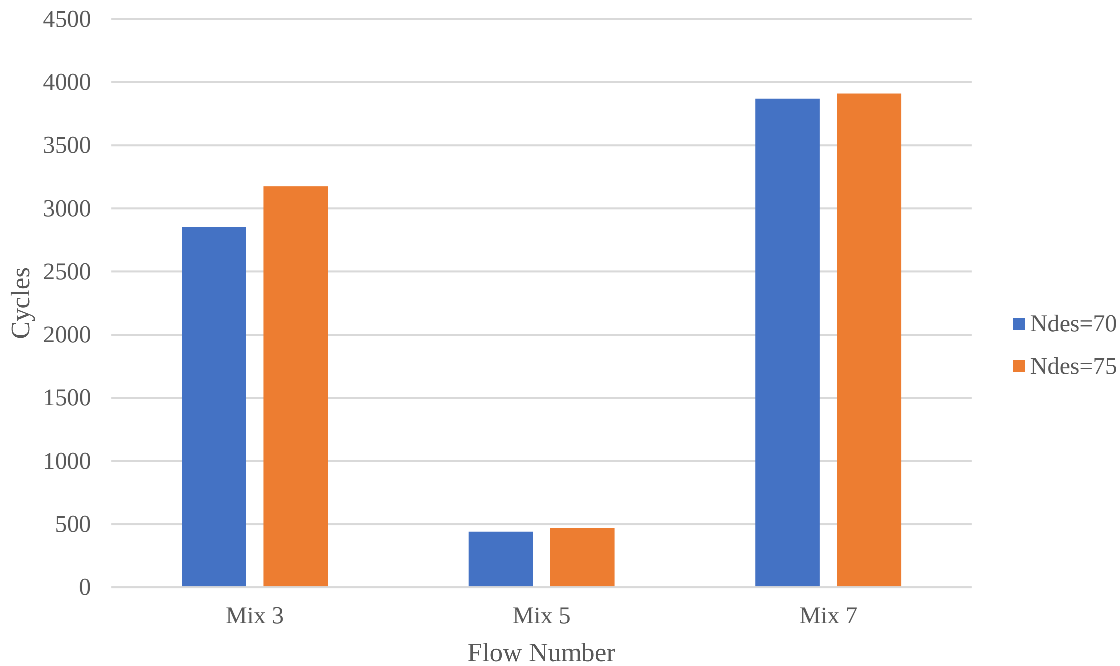


Figure 6-10 Comparison of Flow Number results for BM2-mixes.

6.2.3 Superpave Indirect Tension (IDT) Tests

Figure 6-11 to Figure 6-18 show the results of the Superpave IDT tests. Figure 6-11 and Figure 6-12 presents the results of resilient modulus for BM-2 and D-mixes. The resilient modulus increased with increasing N_{design} for all the mixtures included in this study. It can be attributed to the higher asphalt content for mixes with a lower N_{design}.

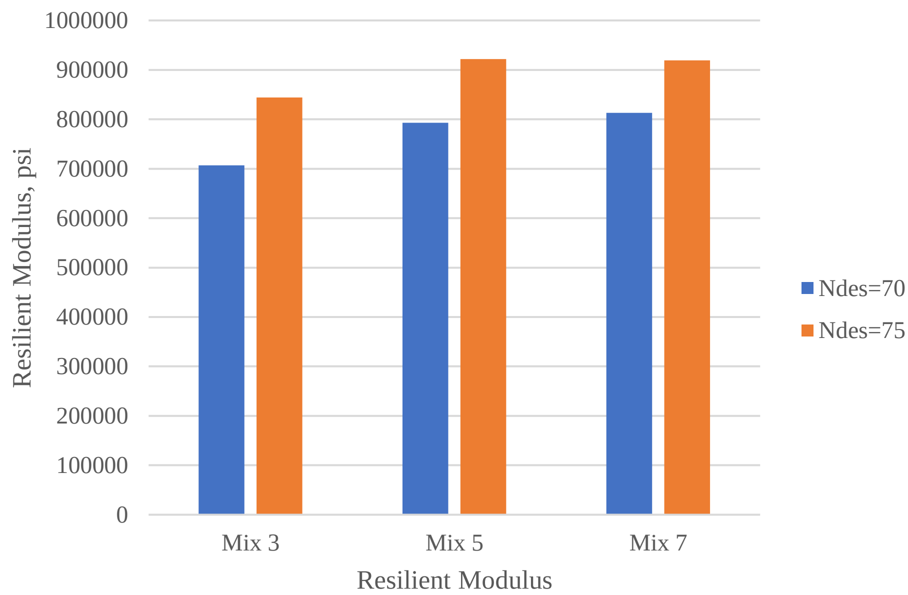


Figure 6-11 Resilient Modulus Results for BM-2 mixes

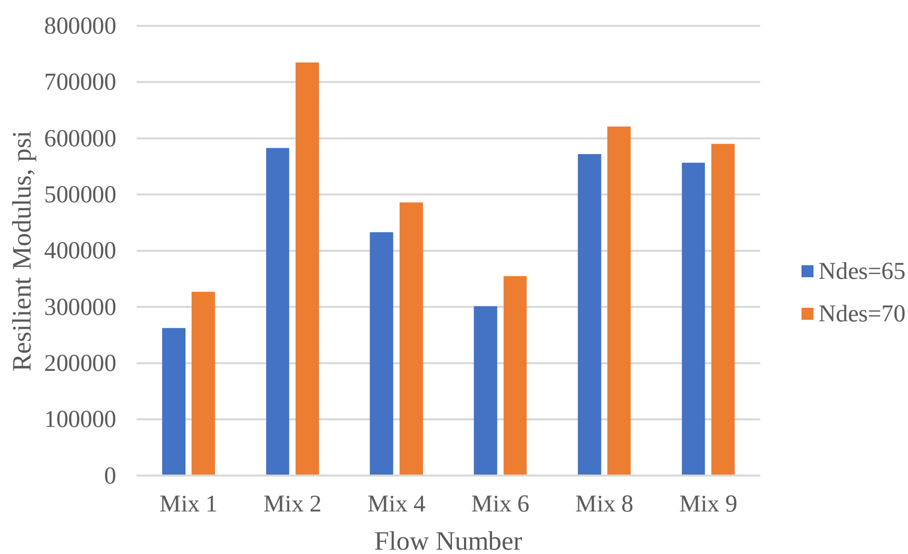


Figure 6-12 Resilient Modulus Results for D mixes

Figure 6-13 and Figure 6-14 present the results of IDT Strength test. Similar to the resilient modulus test, all the mixtures showed a higher failure stress with a higher N_{design}. Also, the difference between different N_{design} was higher for BM-2 mixes.

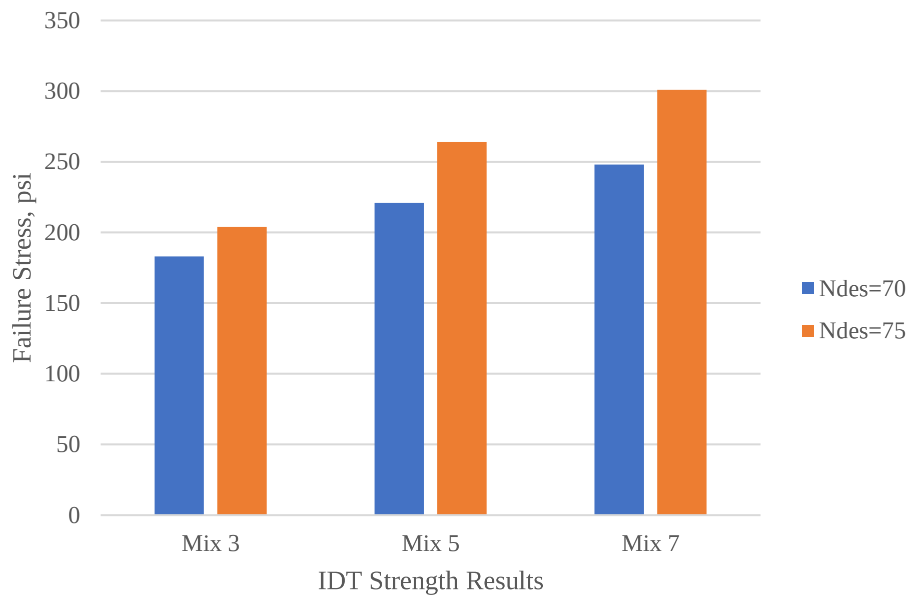


Figure 6-13 IDT Strength Results for BM-2 mixes

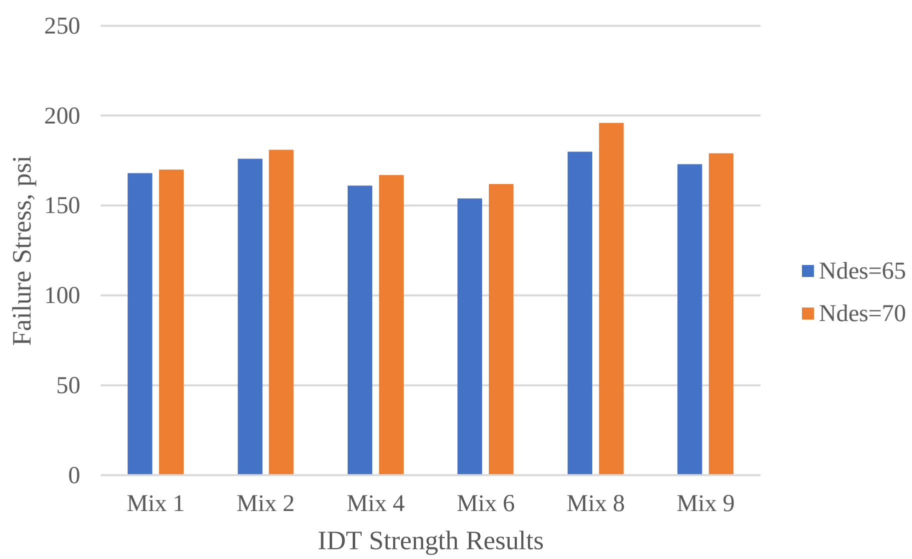


Figure 6-14 IDT Strength Results for D mixes

Figure 6-15 and Figure 6-16 show that the samples prepared with a lower N_{design} represented a larger strain when failed, indicating that higher asphalt content increased the ductility of asphalt mixtures.

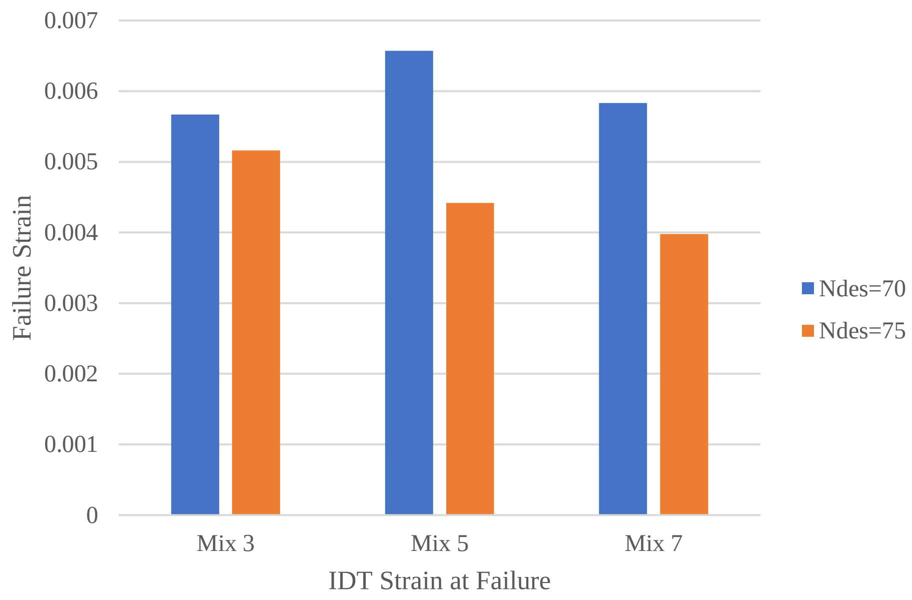


Figure 6-15 IDT Strain at Failure for BM-2 mixes

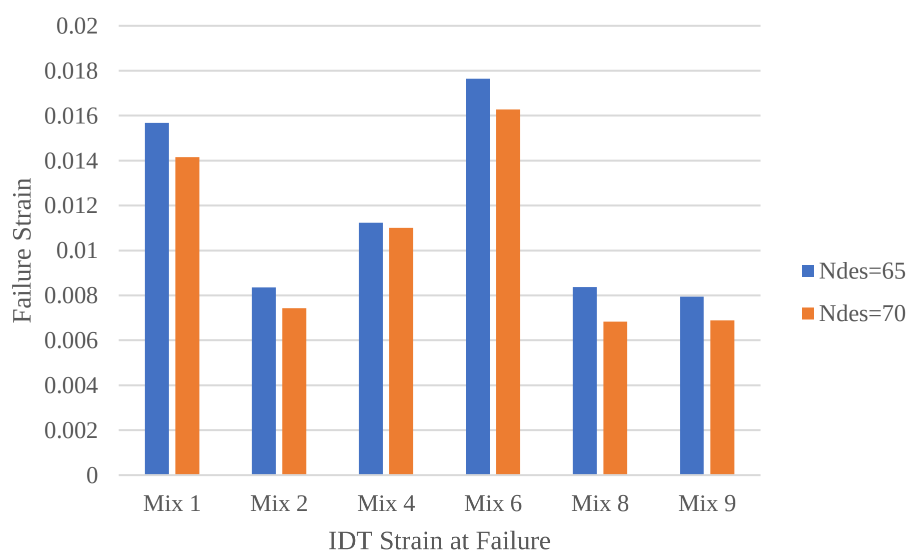


Figure 6-16 IDT Strain at Failure for D mixes

Figure 6-17 and Figure 6-18 show that the samples prepared with a lower N_{design} represented generally a higher $DCSE_f$ than mixtures with lower asphalt content. It indicates that a lower N_{design} provided better resistance to failure as it required more energy to fracture asphalt mixture.

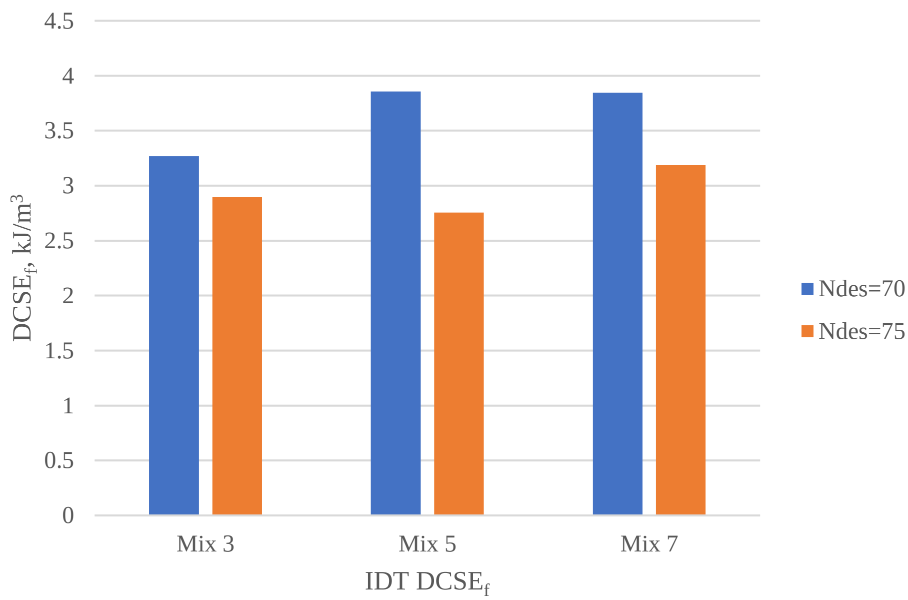


Figure 6-17 IDT $DCSE_f$ Results for BM-2 mixes

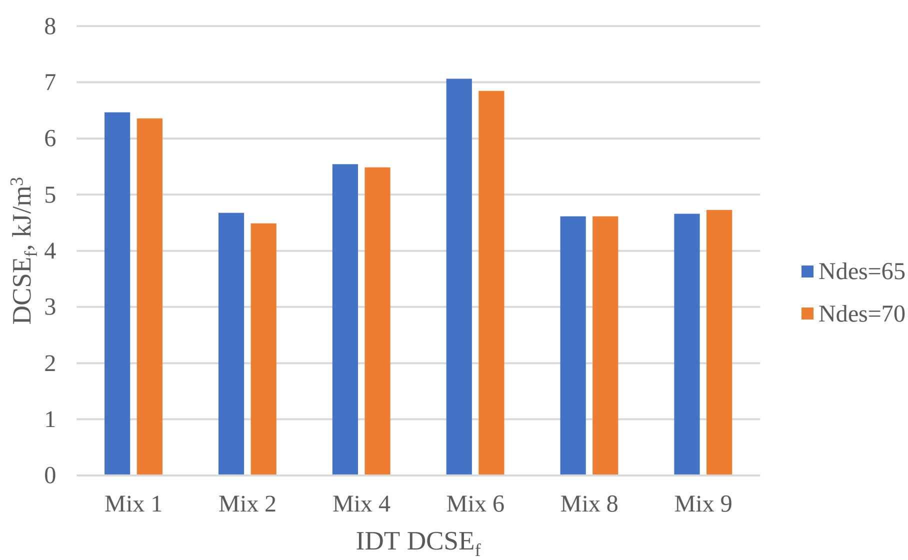


Figure 6-18 IDT $DCSE_f$ Results for D mixes

6.2.4 Asphalt Pavement Analyzer Hamburg Test

Figure 6-19 to Figure 6-27 show the results of rut depths for nine mixtures included in this study at two different N_{design}. All except one mixture presented higher rut depth when using the lower N_{design}. Mixture 1160307 during the first 16,000 passes presented a similar trend with others, but after that the mix with the higher N_{design} obtained higher rut depth. It can be attributed to the moisture damage as stripping inflection point could be observed after the test. It should be noted that stripping inflection points appeared for all three BM2 mixtures designed with 75 gyrations.

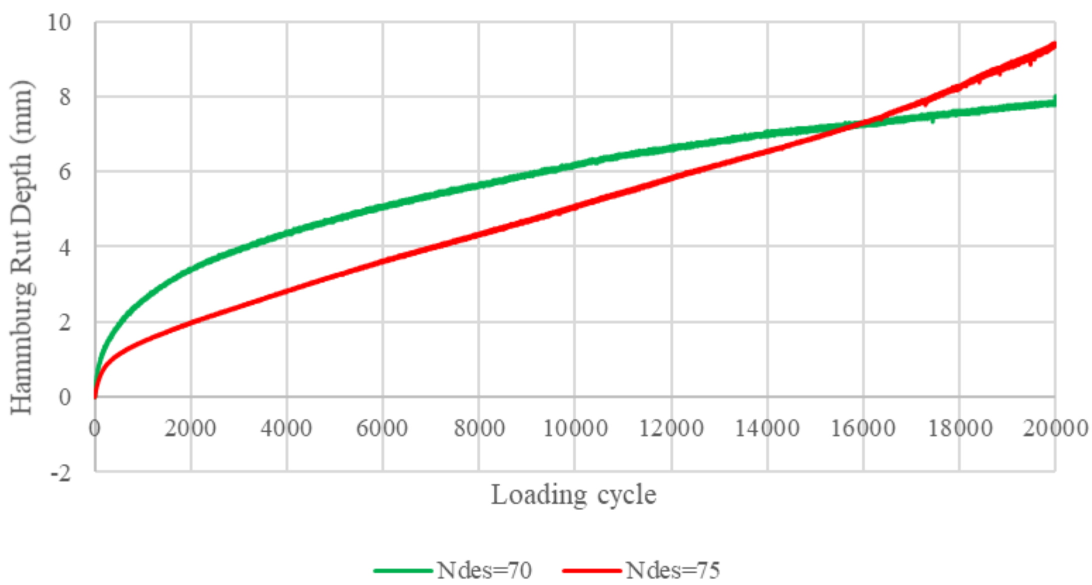


Figure 6-19 Mixture 1160307 – rut depth vs. loading cycle

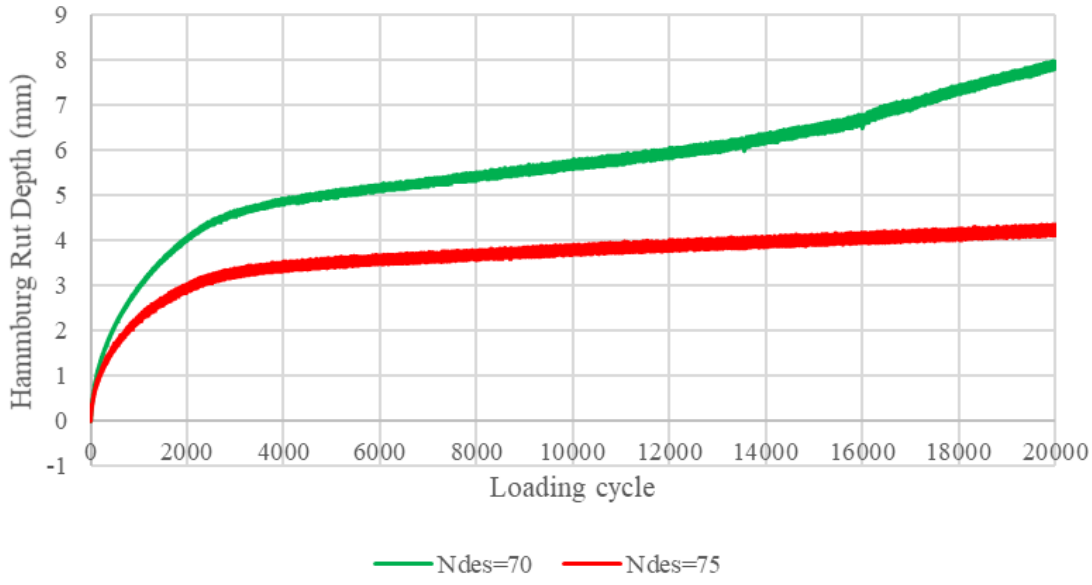


Figure 6-20 Mixture 3160011 – rut depth vs. loading cycle

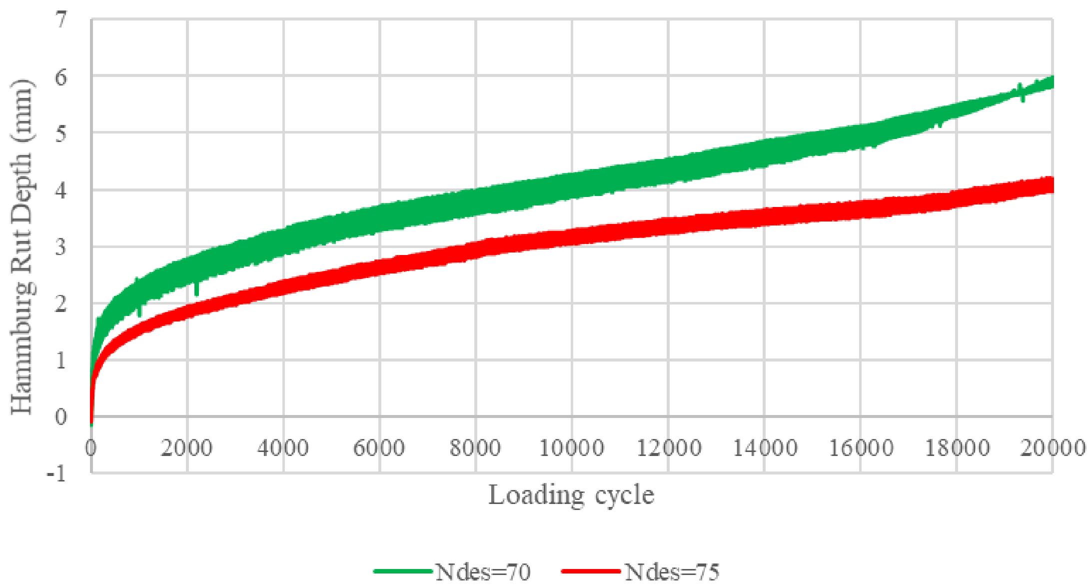


Figure 6-21 Mixture 4160056 – rut depth vs. loading cycle

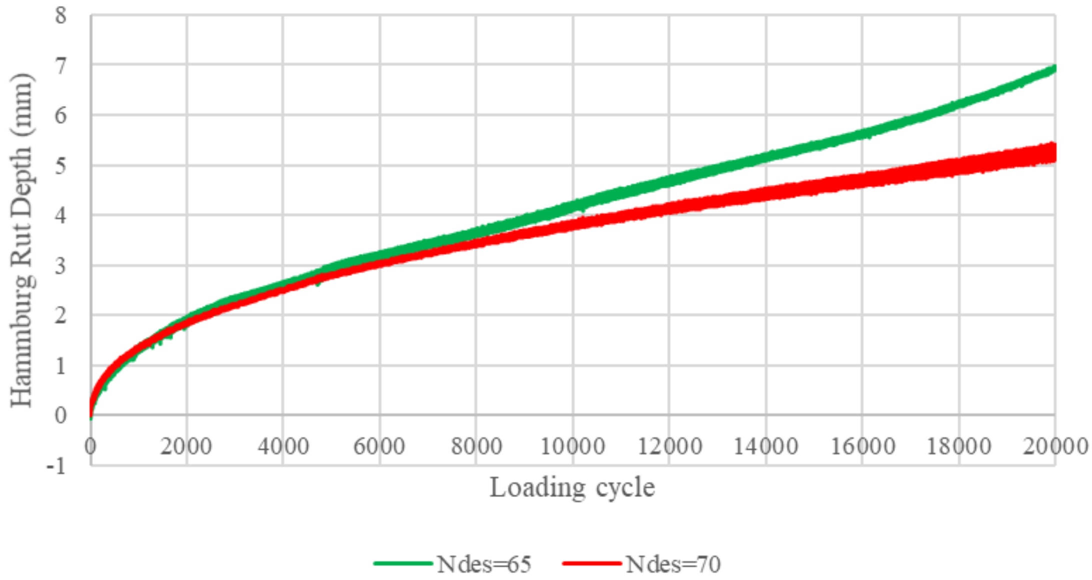


Figure 6-22 Mixture 1160315 – rut depth vs. loading cycle

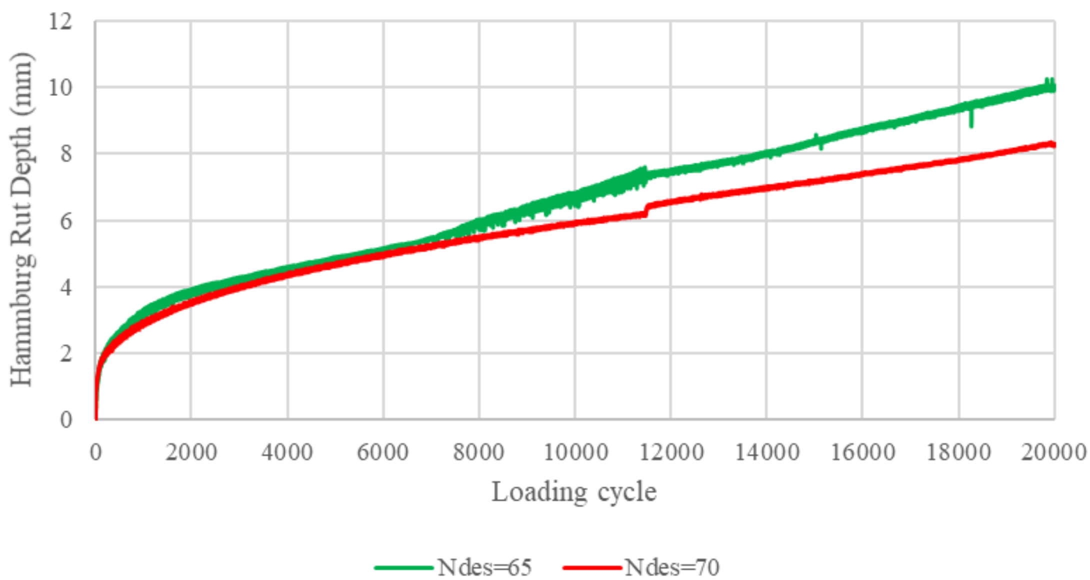


Figure 6-23 Mixture 1160371 – rut depth vs. loading cycle

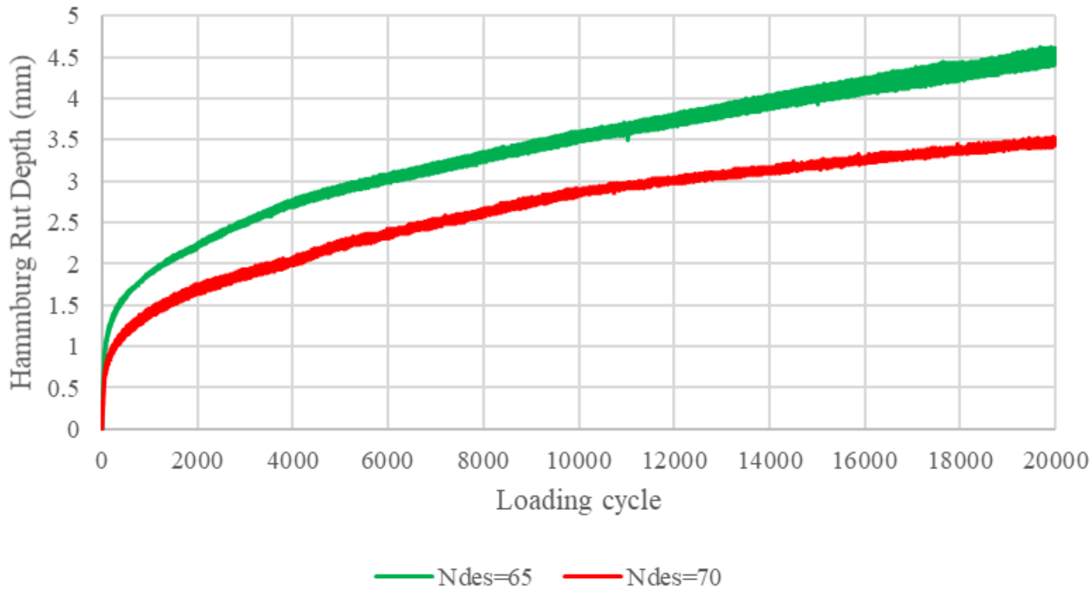


Figure 6-24 Mixture 1160463 – rut depth vs. loading cycle

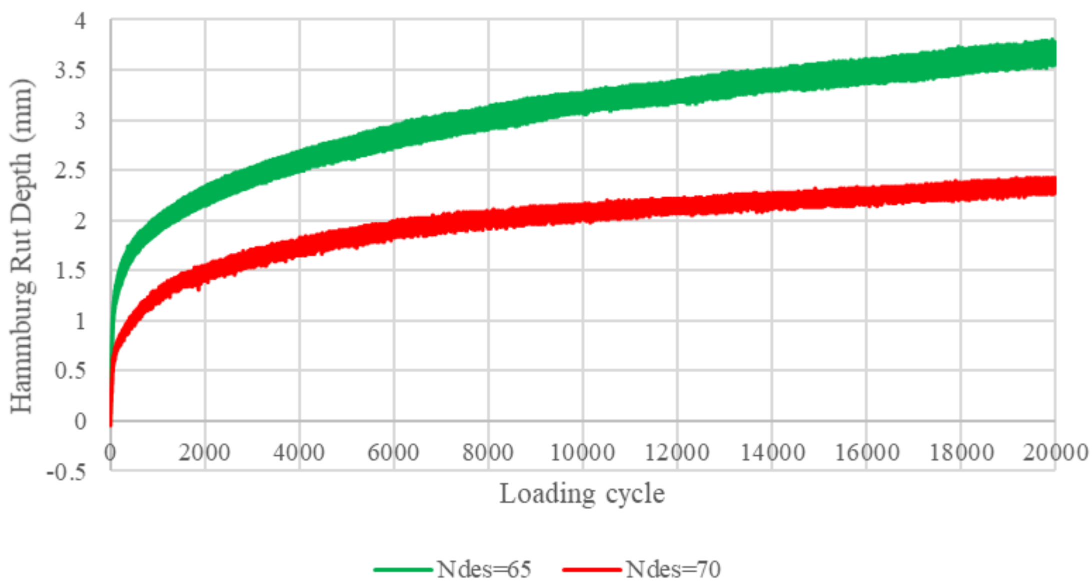


Figure 6-25 Mixture 4160010 – rut depth vs. loading cycle

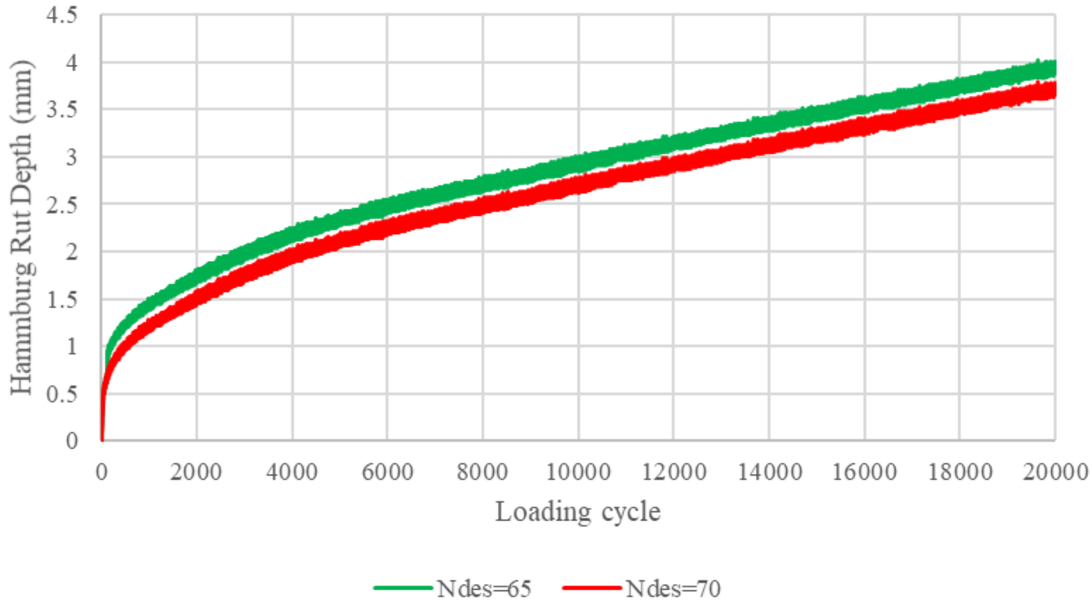


Figure 6-26 Mixture 4160049 – rut depth vs. loading cycle

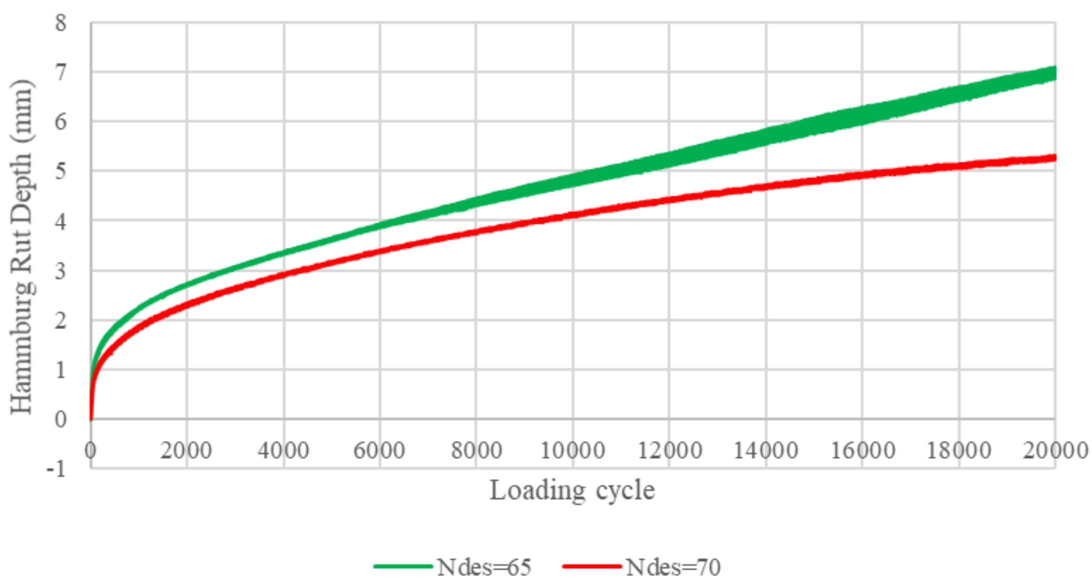


Figure 6-27 Mixture 4160125 – rut depth vs. loading cycle

Figure 6-28 and Figure 6-29 present the summary of the final rut depth at 20,000 passes for different mixtures and N_{design}. It can be observed that as the value of N_{design} increased, the rut depth decreased.

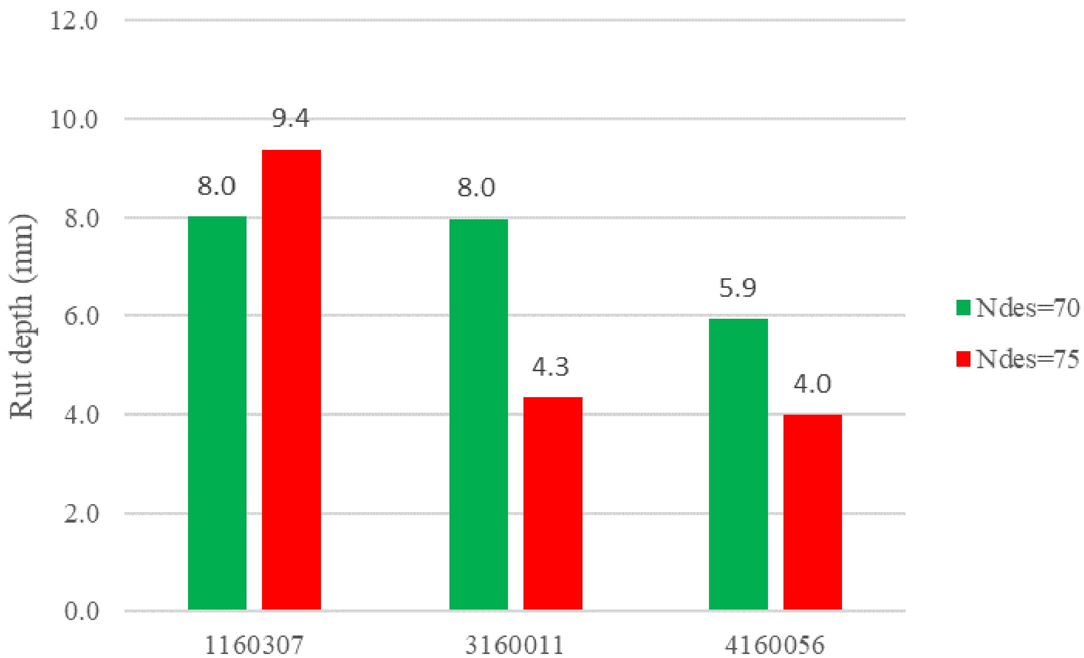


Figure 6-28 Summary of rut depth results for base mixtures

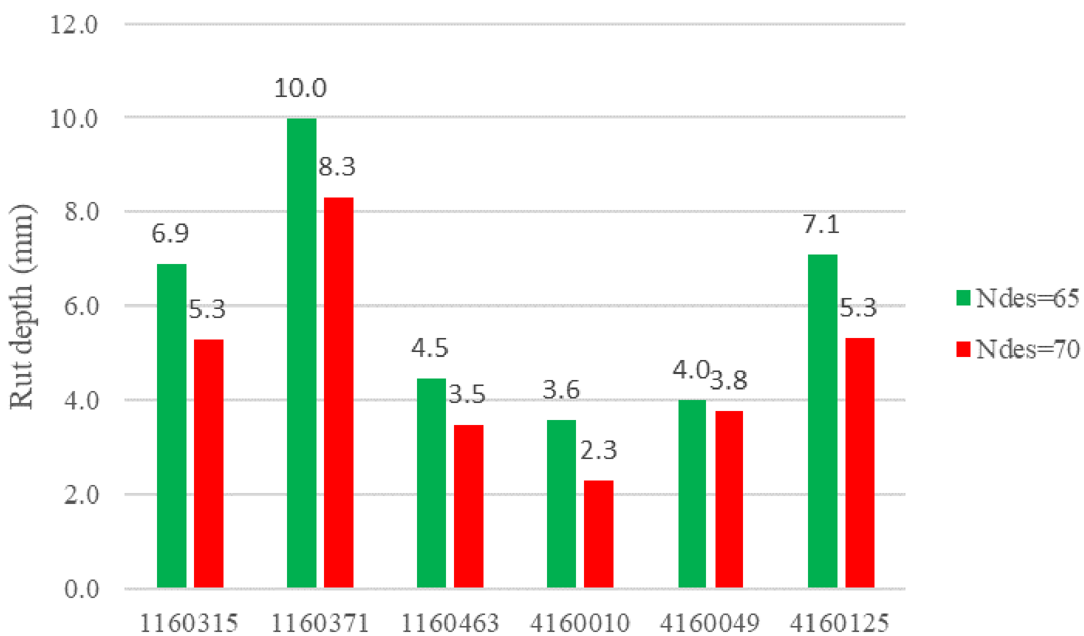


Figure 6-29 Summary of rut depth results for surface mixtures

6.3 Conclusions

In this section, the performance and moisture damage tests were utilized to evaluate the asphalt mixtures at different N_{design}. The following laboratory performance tests were included in this research: Tensile Strength Ratio (TSR), Asphalt Mixture Performance Test (AMPT), Superpave Indirect Tension Tests (IDT) and APA Hamburg Wheel Test. For all the mixtures included in this study, a higher N_{design} yielded lower asphalt binder content and a slightly lower moisture resistance (TSR). D-mixes had a significantly higher moisture resistance (average 91%) than BM-2 mixes (81%). Mixtures 1160307 and 3160011 designed with N_{design} equal to 75 gyrations were outside the TDOT specifications for moisture damage set as 80% minimum. The asphalt mixture designed with different N_{design} followed the general trend as a viscoelastic material; namely, its dynamic modulus decreased with an increase in temperature and increased with an increase in loading frequency. All except one mixture presented higher rutting depth with lower N_{design}. Stripping inflection points appeared for all the three BM2 mixtures designed with 75 gyrations.

CHAPTER 7 IDENTIFYING LOCKING POINT FOR MARSHALL COMPACTION METHOD

7.1 Introduction

Compaction is an important part of the asphalt pavement life. Regardless of the asphalt mix design method, it is a process that uses weight of the rollers to decrease the volume of asphalt mix mass to the required density in relation to the maximum density. During compaction, aggregates are brought together creating skeleton that provides resistance to deformations and at the same time limits permeability by reducing air void content that prolong the life of the pavement. Inadequate compaction can lead to premature damage in the asphalt course and underlying layers (Asphalt Institute 2007).

The compaction process can be affected by many factors such as asphalt cement and aggregates properties, mix type, compaction temperature, lift thickness, base course properties and environmental conditions. Asphalt cement properties change with temperature, which means that there is a specific range where viscosity permits adequate compaction by providing lubrication between particles during the compaction process. Low temperature prevents aggregate particles from moving, and the required density is not possible to achieve (Asphalt Institute 2007).

Another key factor of successful compaction is mix design. The history of asphalt mix design dates to the beginning of the twentieth century when pioneers that worked with asphalt, based on their previous experiences, realized the importance of adequate dosage of mix components. Asphalt mix design is the process of determining the optimum proportions of asphalt cement, coarse aggregates, and fine aggregates, that permits creating well-performing and long-lasting pavements (Asphalt Institute 2007).

The first method to determine optimum binder content in the asphalt mix was the pat test, which was highly imprecise as it was based on visual appraisement, but for the earliest asphalt mix designers, it permitted high advancement in quality and performance (Huber 2016). Around the same time, the bitulithic pavement was developed and patented by Federick Warren. This mix incorporated large stones up to three inches, allowing lower asphalt cement consumption and a lower price (NAPA 2016).

The threshold for development of asphalt industry was Bruce Marshall's invention of a new design method. For over 50 years, Marshall Asphalt Mix Designs dominated the United States and the world paving industry. Internationally, the Marshall method is still the principal choice for designers. Two generations of engineers, specialists and experts utilized this design method without profound understanding of the impact compaction process (Huber 2016; NAPA 2016).

The Superpave mix design method brought new challenges and opportunities to the compaction process. Superpave is permitting better understanding of the compaction process by introducing the Superpave Gyratory Compactor (SGC), which allows monitoring of specimen height after each gyration and provides better simulation of compaction than previous compactors (Asphalt Institute 2017). Therefore, a reasonable compaction effort for mix design can be determined so that over-compaction can be avoided. For the impact hammer compaction, there is no method available to characterize and analyze the behavior of an asphalt mixture during compaction.

In the last 30 years, there were attempts to utilize accelerometers to calibrate the Marshall Hammer, but due to the high level of noise in the signal from the accelerometer, it was not possible to evaluate the data and calibrate the hammer. Siddiqui et al. 1988 conducted research to determine the possibility of using accelerometers to calibrate the Marshall Hammer. Their objective was to eliminate error in the results when different hammers are used. The analysis of the acceleration data indicated limited variability between each blow. The structural ringing made it impossible to analyze the impact of the hammer. The filter applied to the signal can reduce ringing but also alter the signal. Cassidy et al. 1994 made a similar attempt to calibrate the Marshall hammer using an accelerometer, load cell and LVDT. Similarly, like Siddiqui et al. 1988, they encountered strong structural ringing and decided to utilize only the data from the load cell and LVDT. Except for the work cited above, there has been little effort to describe the compaction process of the asphalt mixtures when an impact hammer is used, due to equipment limitation and work scope. The work of Siddiqui et al. 1988 and Cassidy et al. 1994 was focused on finding a calibration method for the Marshall hammer, not on describing compaction properties of asphalt mixtures.

Another approach for utilizing accelerometers was the development of a Clegg Impact Tester. In this approach, at the initial stage of research a standard Proctor-type hammer was equipped with an accelerometer and utilized to measure deceleration of the falling hammer mass. The Clegg Tester can be used to determine hardness of compacted soil and the results can be correlated with a CBR value. The standard procedure consists of dropping a hammer four times in the same place and identifying the highest deceleration value. A higher value of the peak deceleration indicates stiffer material. Currently, the Clegg hammer is not limited to standard Proctor-type hammer. There are several models with different hammer weights. The weight of the hammer is based on the type of soil to be tested. The Clegg Impact Tester provides basic strength values at a relatively small cost and requires low technical abilities (Clegg 1976; Clegg 1978; Clegg 1980; Clegg 1983).

The success of the Clegg Hammer can be attributed to direct contact between the falling mass of the Proctor hammer and the soil during impact. The construction of the Marshall Hammer does not allow direct contact between a falling mass and the asphalt mix because the falling mass hits a metal head before the metal head hits asphalt mix. This impact between the two metal parts of the Marshall Hammer produces structural ringing (noise) that is difficult to eliminate.

One key achievement of the Strategic Highway Research Program (SHRP) was the introduction of the Superpave Gyratory Compactor (SGC). The SGC improves our understanding of the compaction process: the new compactor allows us to monitor the specimen height after each gyration and provides better simulation of compaction than previous compactors. Since the introduction of the gyratory compactor, various researchers have attempted to use a densification curve, which is obtained from specimen height change, to determine compactability of the asphalt mix. Bahia et al. 1998 introduced the concept of the Construction Densification Index (CDI) and the Transportation Densification Index (TDI). The CDI is te area under the SGC densification curve from 88%Gmm to 92%Gmm, where G_{mm} is defined as theoretical maximum specific gravity of the asphalt mix. The TDI is the area under the SGC densification curve from 92%Gmm to 98%Gmm. It is desirable that an asphalt mix possess a low value of CDI, because it represents low effort required in the compaction process. The CDI is meant to represent the energy that is used by a roller during compaction to achieve required compaction. Figure 7-1 presents densification curve, CDI and TDI.

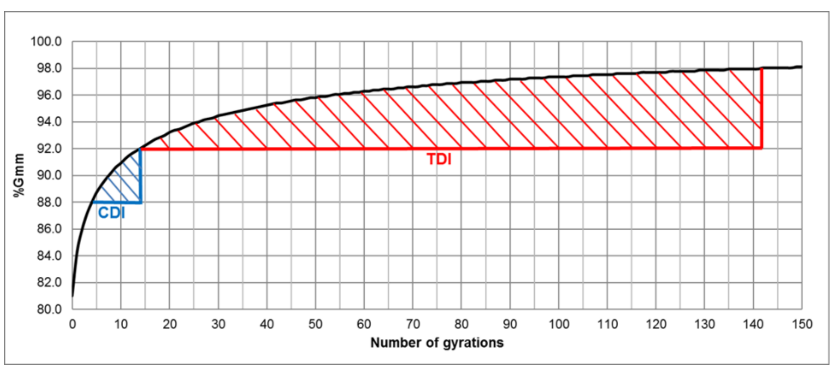


Figure 7-1 Determination of CDI and TDI from a densification curve.

Anderson et al. 2002 introduced a slope of densification curve to describe compactability. The compaction slope can indicate the shear resistance of the aggregates structure in the asphalt mix. Higher slope means higher resistance to compaction. Mallick 1999 indicated that it is commonly accepted and required by most agencies that the target compaction of an asphalt lift in the field should reach 8% of air voids. The value of required air voids can be taken as a borderline between expectations regarding compactibility of the asphalt mix. It is desired for the mix to compact easily until it reaches 92%Gmm and that it should become hard to compact when it exceeds 92%Gmm. In the first case, it is desirable by contractors to have a mix that does not require high compaction effort. In the second case, after the mix reaches 8% of air voids, it should be hard to compact as it permits higher resistance to the traffic and a longer period of service.

The most common concept that is used to evaluate compactability of the asphalt mix is a gyratory locking point. This concept is based on change of the specimen height during the gyratory compaction. Originally, the gyratory locking point was proposed by William J. Pine while working with the Illinois Department of Transportation. The gyratory locking point defines a threshold on the densification curve beyond which the mix structure starts to resist further compaction and aggregates can be fractured. Different mixtures lock up at different number of gyrations and at different air void contents. There are many definitions of the gyratory locking point by different researchers and agencies. Mohammad and Shamsi 2007 reported that the Alabama Department of Transportation defined the locking point "as the point where the sample being gyrated loses less than 0.1 mm in height between successive gyrations". Georgia DOT defined the locking point "as the number of gyrations at which, in the first occurrence, the same height has been recorded for the third time". Louisiana Transportation Research Center (LTRC) denotes "the number of gyrations after which the rate of change in height is equal to or less than 0.05 mm for three consecutive gyrations" as the locking point. In this study, the locking point was defined as the first gyration in the first set of three gyrations at the same height preceded by two sets of two gyrations at the same height. It is the most widely accepted definition presented by Vavrik and Carpenter 1998. Since the Marshall method is still the principal choice for engineers around the world it is important to develop a method to characterize the compaction behavior of asphalt mixtures using Marshall Hammer compaction and to evaluate compactibility of the different asphalt mixtures used in Tennessee. A shock accelerometer was used to determine responses of the HMA at different stages of compaction.

7.2 Laboratory experiments

A total of eight plant hot mix asphalt mixtures were utilized in this study (one mixture was not included, because of not enough material to conclude the study). The preparation of the specimens and testing was conducted with the same equipment as in previous chapters. First, the theoretical maximum specific gravity (G_{mm}) was determined following the AASHTO T 209 specification. Next, asphalt mixtures were reheated for two hours to a temperature that permits air void content at 4% after completing 150 blows (75 blows to each side) with an impact hammer. A standard sample weight of 1,230 grams was used.

HMAs were compacted utilizing the 10-lb. Marshall hammer and 75 blows to each side with the accelerometer placed on the hammer in the vertical direction of hammer drop (Figure 7-2).

Figure 7-2 Accelerometer installed on the impact hammer

The accelerometer was then connected to the National Instrument data acquisition system with a coaxial cable. A LabVIEW System Design Software was used to receive and store the acceleration data. Once the compaction process was concluded, bulk specific gravity (G_{mb}) was determined by AASHTO T 166. Data obtained from the accelerometer were later filtered with five points moving average, and the response from the mix after each blow was plotted in the time domain, analyzed, and compared to data obtained from Superpave Gyratory Compactor (SGC) densification curve (Polaczyk et al. 2018; Polaczyk et al. 2019a; Polaczyk et al. 2019b).

The analysis of the accelerometer data exhibited the existence of a point similar to the SGC locking point: when crossing this point the HMA resists further compaction. In this study, the impact locking point was defined as the number of blows that after which the response of the mix sent to the accelerometer becomes stable with change neither in peak acceleration nor impact time.

7.3 Results and Discussion

7.3.1 Impact Hammer Locking Point

In the light of the studies mentioned in the Literature Review Chapter, the author of this study used acceleration data in the time domain after each of the 150 blows to identify patterns that can allow determination of the impact locking point. The idea of the locking point is based on the assumption that during the compaction process, the skeleton of the asphalt mix is gradually developed until the point where course aggregates interlock and resist further compaction. In this research, the impact locking point is defined as the number of hammer blows at which the acceleration-time history curve stops fluctuating and values of the acceleration and impact time become stable. Figure 3 presents a typical example of the different stages in the compaction process obtained from the acceleration data. The initial stage (Figure 7-3a) is characterized by a relatively long impact time, a low acceleration peak and the existence of more than one peak. As the density of the asphalt mixture increases, the stiffness also increases, causing changes in the acceleration response.

During the middle compaction stage, the impact time becomes shorter, peak acceleration increases and multiple acceleration peaks evolve in to just one peak (Figure 7-3b). The final compaction stage (Figure 7-3c) is taken, in this study, to be the impact locking point. When the asphalt mixture reaches this stage, the peak acceleration and impact time become stable, which means there in no further significant increase in compaction.

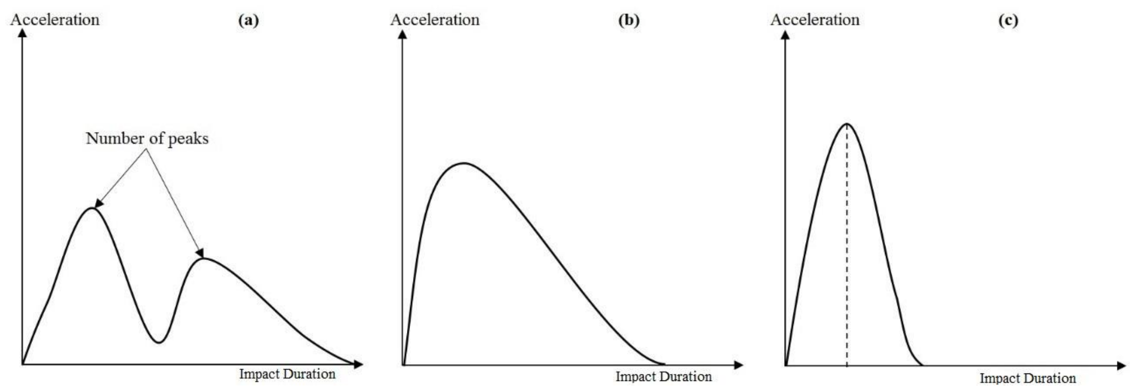


Figure 7-3. Typical shapes of response plots in different compaction stages: (a) initial, (b) middle, (c) at impact locking point

The analysis described above was conducted for all eight mixtures (two samples for each mixture). The three compaction stages were determined for seven of them. One mix (No. 3) reached the second compaction stage after 150 blows, so no impact locking point could be determined. The locking point was marked as >150. For the seven mixes that reached the final compaction stage, the locking point ranged from 108 to 146 blows. The summary of locking points is presented in **Table 7-1**.

Table 7-1.The summary of the impact locking points

Mix No.		1	2	3	4	5	6	7	8
Locking Point (blows)	Sample 1	144	106	>150	112	112	145	117	142
	Sample 2	146	110	>150	113	111	147	116	140
	Average	**145**	**108**	**>150**	**113**	**112**	**146**	**117**	**141**

It can be observed that for two different samples of the same asphalt mixture, the results are varying between one and four blows, which can suggest that the results may be repeatable. However further study should be performed to confirm.

Figure 7-4 presents an example of data analysis and interpretation, in this case, for mixture No. 2, Sample 1. This asphalt mixture sample was evaluated to have the lowest locking point of 106 blows. From Figure 4, the three compaction stages described above can be identified. The initial stage ranges from blow 1 to blow 30 and is characterized by two acceleration peaks. In this stage, the peak acceleration ranged from 250g to 350g, and the impact duration decreased from the initial 50 ms to 20 ms. The second stage began with the 30th blow and lasted until the 106th blow. In this stage, the acceleration peak increased from 350g to 550g, and the impact duration decreased from 20 ms to 15 ms. After the final stage was reached, the acceleration peak was maintained at the same level of 550g and impact duration of 15 ms until the last (150th) blow.

The locking point describes the point during compaction process that is a boundary between easy and difficult compaction. In this study, the impact locking point is defined as a point when the response from the accelerometer becomes stable. It is assumed that when the response becomes stable there will be no further major changes in the stiffness of the asphalt mix, which implies that there will be no major changes in density or air voids.

To validate this line of reasoning, specimens were compacted at various numbers of hammer blows. The first set of samples was compacted with the 150 standard blows to reach 4% air voids. The second set of samples was compacted with the number of blows defined previously as the locking point.

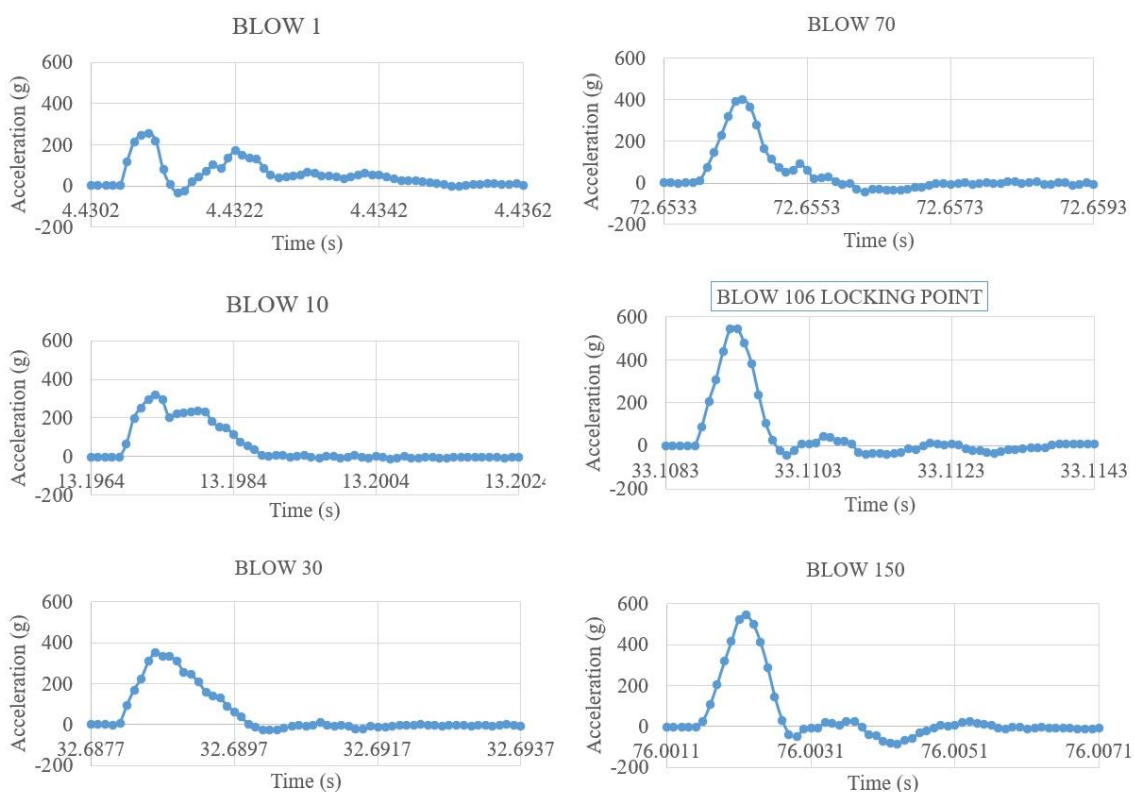


Figure 7-4. Example of data obtained from accelerometer. Mixture 2, Sample 1

The expectation was that the air void content in these two sets would be similar. The last set of samples was compacted at 15 blows below the impact locking point. The expectation was that air voids content would be higher than in two previous sets. The summary of the validation samples is shown in Figure 5. Mixture 3 is not included in the summary because the impact locking point for this mix was not defined in the range of the 150 blows. As presented in Figure 7-5, once the locking point was achieved, further compaction caused minimal change in air voids for all tested mixtures. On average, the air void content was 0.19% higher at the locking point than at the 150th blow. The average difference between air voids at the 150th blow and 15 blows below locking point was 1.2%. From the air void data, it can be concluded that the locking point found with the accelerometer is in fact the point where the asphalt mix changed compaction properties and became more difficult to compact.

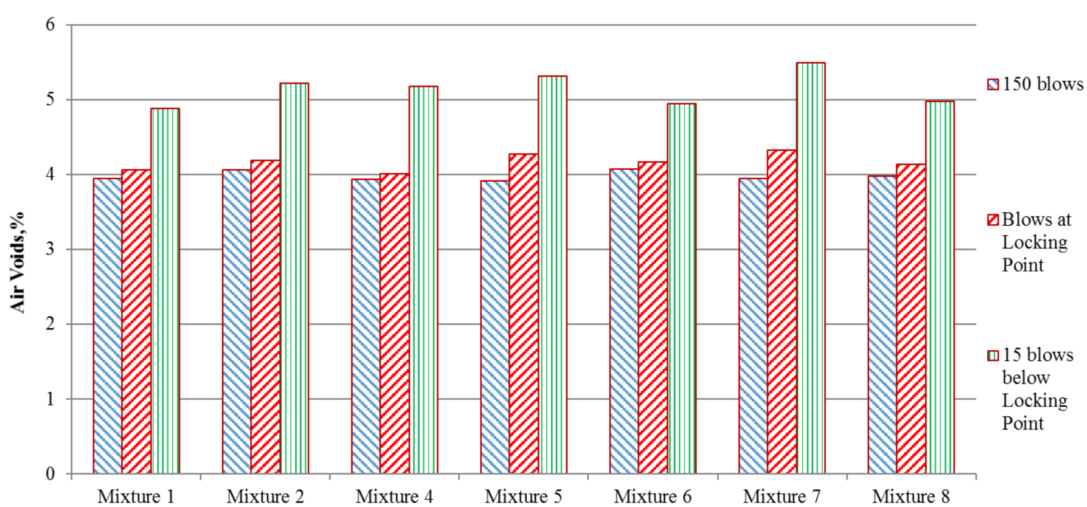


Figure 7-5. Comparison of air voids at 150 blows, at the locking point, and 15 blows below

the locking point.

7.3.2 Validation by Superpave Gyratory Compactor

Since the concept of locking point was developed for gyratory compactor, in this study, this idea was used to compare the results obtained with the impact hammer and the accelerometer. The purpose of this comparison was to evaluate if there is relation between gyratory and impact locking point. Specimens for all eight mixtures (two samples per mix) were compacted using the Pine Instrument Company AFGC125X Superpave Gyratory Compactor (SGC), 150 mm molds, and the same temperatures as previously used with the impact hammer. After the specimens cooled, the bulk specific gravity was determined by AASHTO T 166. Next, utilizing the theoretical-maximum specific gravity and specimen height change data obtained during compaction from the superpave gyratory compactor, densification curves were plotted, and the gyratory locking point was determined for each mix utilizing 2-2-3 method. The summary of the gyratory locking point is presented in **Table 7-2**.

Table 7-2. Comparison of the impact hammer and the gyratory locking point

Mix No.	Impact Locking Point (blows)	Gyratory Locking Point 2-2-3 (gyrations)
1	145	76
2	108	51
3	>150	83
4	113	56

5	112	53
6	146	74
7	117	56
8	141	73

Locking points that were obtained with the SGC have similar trend as the locking points obtained from the impact hammer and the accelerometer. The highest value of 83 gyrations obtained with Mixture 3 also has the highest value of impact locking point, defined as >150 blows. Similarly, Mixture 2 has the lowest gyratory locking point of 51 gyrations and the lowest impact locking point of 108 blows.

Figure 7-6 presents relationship between the locking point results acquired with the gyratory compactor and the impact hammer. The coefficient of determination (R^2) for the set of data obtained is 0.97. It can be concluded that there exists a correlation between the locking points from the different methods of compaction included in this research.

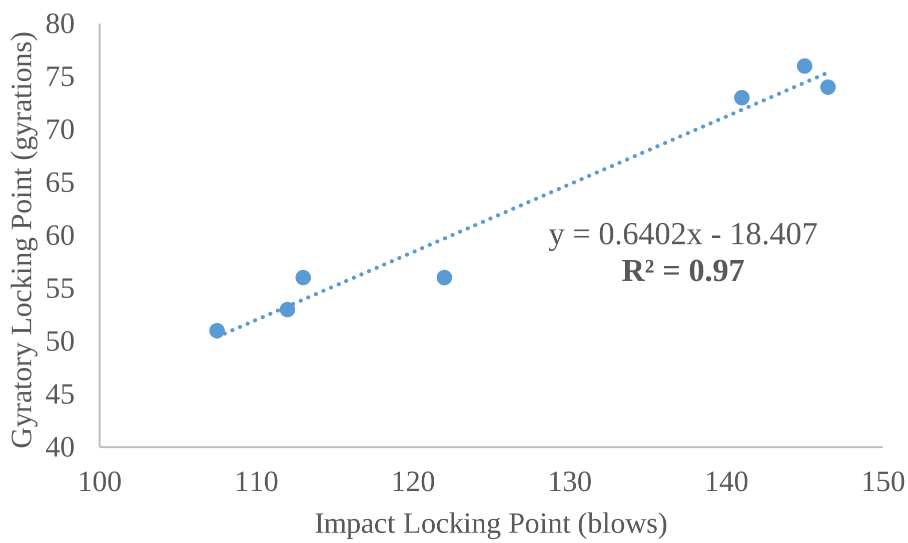


Figure 7-6. Relationship between Gyratory and Impact Locking Point

7.4 Conclusions

In this section, eight different asphalt mixes from different regions of Tennessee were analyzed with purpose to develop a method to characterize the compaction behavior of asphalt mixtures using Impact Hammer compaction and to evaluate the compactibility of the different asphalt mixtures used in Tennessee.

Based on the results it can be concluded that accelerometer can determine different stages in the compaction process. The response received from the asphalt mix via the accelerometer is similar for different samples of the same asphalt mix, which confirms repeatability of this method. The locking point for HMA can be determined as the point where the acceleration and the response time become stable. For researchers, it can be used to evaluate the compactability of asphalt mixture. For contractors, it can provide an easy way to determine the effort required to compact asphalt mixture in the field. The locking point for HMA can be determined as the point where the acceleration and the response duration become stable. For seven out of the eight evaluated mixes the locking point was established between 108th and 146th blow. For one mix, the locking point was higher than 150 blows. The locking points obtained from the Superpave Gyratory compactor confirmed the results obtained with the impact hammer, and the linear relation could be established. In this study, plant mixtures with different properties and materials were evaluated and compared. Further study is needed to evaluate and compare mixtures with limited number of variables that can affect locking point.

CHAPTER 8 LOCKING POINT FOR MARSHALL AND SUPERPAVE COMPACTORS

8.1 Introduction

In this chapter, the locking characteristics of aggregate structure in asphalt mixtures were investigated through routine mixture design procedures. The Marshall Compactor and the Superpave Gyratory Compactor were used to evaluate the impact locking point and the gyratory locking point of different asphalt mixtures. Additionally, the samples for the gyratory locking point were compacted in two mold sizes to evaluate the effect of sample size on the compaction results. A total of ten mixtures was included in this study: seven dense-graded mixes with a maximum size of aggregate from 50.8 mm (2”) to 12.7 mm (1/2”) and three Stone Mastic Asphalt with a maximum size of aggregate from 25.4 mm (1”) to 12.7 mm (1/2”). The maximum size of an aggregate is defined as the smallest sieve opening through which the entire amount of the aggregate is required to pass (AASHTO T 2). The binder used was PG 64-22. The wide range of TDOT mixture types were utilized including 307A, 307B, 307BM2, 307C, 411D, 411E, 411TLD and three different Stone Mastic Asphalt (SMA) mixtures, currently not used in Tennessee, 9.5 mm, 12.5 mm and 19.0 mm.

8.2 Laboratory Experiments

Different limestone aggregates were collected in East Tennessee, including #4 stone, #5 stone, #56 stone, #7 stone, and #10 screenings. Only limestone was used to limit the variability of the results. Aggregates were screened through the same set of sieves: 50.8 mm (2”), 38.1 mm (1.5”), 31.8 mm (1 ¼”), 25.4 mm (1”), 19.1 mm (¾”), 15.9 mm (5/8”), 12.7 mm (½”), 9.5 mm (3/8”), No. 4, No.8, No. 30, No. 50, No. 100 and No. 200. The objective of the screening was to reduce aggregate variability and permit the design of a set of asphalt mixtures that possessed the same aggregate properties and would allow comparison of the locking points of different mixtures with limited influence of abrasion and angularity. For all the mixtures, the same unmodified binder PG 64-22 was used to eliminate the influence of binder properties on the results. The mixing temperature for this binder was 154.4 °C (310 °F); the compaction temperature was 143.3 °C (290 °F). The same equipment was used to prepare and test the samples.

Once the material retained on each of indicated sieves was pre-screened, the process of mix design started. For the seven dense-graded mixtures, the Tennessee Department of Transportation’s specifications were used (Tennessee Department of Transportation 2015). For the three Stone Mastic Asphalt mixtures, the Georgia Department of Transportation’s specifications were used (Georgia Department of Transportation 2013). The granulometric composition of each mixture was chosen as a middle point between the low and high limits provided in the specifications. A summary of the aggregates’ composition is presented in

Table 8-1. For each mixture, coarse and fine specific gravities were determined. The sieve No. 4 was used to sort the aggregates into coarse and fine groups. Next, four samples with different binder content were prepared for all ten mixtures. The specific gravity of each compacted specimen was tested, and the optimum binder content was determined. A summary of mix designs is presented in

Table 8-2. The Marshall Mix Design was utilized to compact all the mixtures with a different number of blows depending on the mix type: dense-graded specimens received 75 blows for each side, and SMA specimens got 50 blows for each side. The same 101.6-mm (4-inch) mold size was used for all specimens. After the mix designs were completed, the production of asphalt mixes started, and around 25 kg of each mixture was prepared.

Table 8-1. Summary of Hot Mix Asphalts Granulometric Composition

Mix	1	2	3	4	5	6	7	8	9	10
Sieve	307 A	307 B	307 BM2	307 C	411 D	411 E	411 TLD	SMA 9.5	SMA 12.5	SMA 19.0
50.8 mm	100	100								
38.1 mm	90	100								
31.8 mm			100							
25.4 mm										100
19.1 mm	60	77	87	100					100	95
15.9 mm					100					
12.7 mm					97	100	100	100	93	57
9.5 mm	42	60	65	80	86	86	95	85	63	42
No. 4	30	42	48	52	65	65	65	39	24	24
No. 8	20	32	35	35	46	46	46	23	20	18

No. 30	12	16	19	18	23	23	25			
No. 50			14	11	14	14	14	13	15	15
No. 100	5	7	8	6	7	10	7			
No. 200	2.3	4.5	4.5	4.5	4.0	7.0	6.0	10.0	10.0	10.0

Table 8-2. Summary of Hot Mix Asphalt Designs

Mix		Optimum AC (%)	Air Void (%)	VMA (%)	VFA (%)	Blows
1	307A	3.5	4.0	14.0	71	75
2	307B	4.0	4.1	14.8	72	75
3	307BM2	4.4	4.0	15.2	74	75
4	307C	4.8	4.1	15.3	73	75
5	411D	5.5	4.0	15.6	74	75
6	411E	5.8	4.0	15.8	75	75
7	411TLD	5.8	4.0	15.7	75	75
8	SMA 9.5	6.8	3.6	18.2	80	50
9	SMA 12.5	6.4	3.5	17.9	80	50
10	SMA 19.0	5.9	3.5	17.2	79	50

The asphalt mixtures produced in the laboratory were placed in the oven for two hours for short-time aging. Each mixture consisted of eleven specimens: five specimens for the impact locking point, three specimens for the gyratory locking point compacted in a 150-mm mold, and three specimens for the gyratory locking point compacted in a 100-mm mold. The weight of the samples for both the impact locking point and the gyratory locking point compacted in the 100-mm mold was 1,230 g, while the weight of samples for a 150-mm mold was 4,000 g. The accelerometer was placed on the falling mass of the Marshall Hammer and connected to the data acquisition system. The software was configurated to record the data with a frequency of 10,000 Hz. The Superpave Gyratory Compactor was set to compact the samples by applying two hundred gyrations.

The samples used to test the impact locking point and the gyratory locking point compacted in the 150-mm mold were prepared for each mixture on the same day. Because it was time-consuming to change the Superpave compactor head from 150-mm to 100-mm, the samples for the gyratory locking point compacted in the 100-mm mold were prepared the next day. All the samples prepared for testing the impact locking point were compacted utilizing 75 blows on each side of the specimen (the samples for the SMA mix design had been compacted with 50 blows on each side). The compaction process was standardized at 75 blows on each side to maintain the same conditions for all the specimens and to allow comparison of the results. Similarly, the samples compacted via the Superpave Gyratory Compactor were all under the same condition of two hundred gyrations.

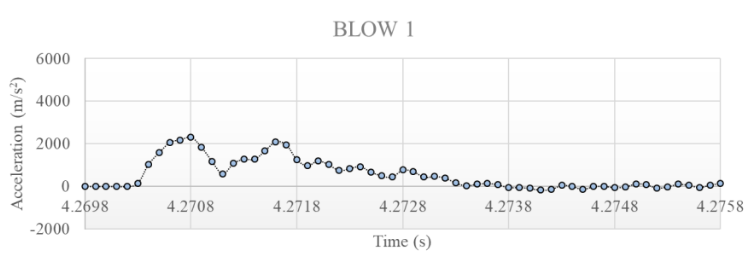
The impact locking point can be defined as the first blow of the impact hammer that produces stable peak acceleration and impact duration, which means the duration of the interaction between the hammer and the mixture. This method is based on the change of specimen stiffness during compaction. Initially, during the first blows applied to the specimen, the mixture is still loose, which implies low stiffness. During this initial stage of compaction, the peak acceleration is low, and the duration of impact is long. In this stage, various acceleration peaks can be observed, mostly limited to two peaks. As the compaction continues with every blow, the stiffness increases, peak acceleration increases and impact duration decreases. After the impact locking point is reached, changes in stiffness become minimal, and peak acceleration and impact duration no longer seems significant changes. The impact locking point can be used to indicate a boundary that separates effortless and effortful compaction. Difficult compaction means that there is no meaningful change in density and further compaction can lead to aggregate damage. Polaczyk et al. 2018 measured change in density to confirm that after crossing the locking point, the change in density is significantly smaller than before the mixture reaches the locking point. Three samples were compacted, first at 150 blows, second at the impact locking point and third at 15 blows below the impact locking point. On average, samples that were compacted at 15 blows below locking point had air voids 1.2% higher than at the final compaction, while samples that were compacted at the locking point had air voids only 0.19% higher than at the final compaction.

The gyratory locking point was determined by following the Georgia Department of Transportation (GDOT) method, which defines the locking point as the number of gyrations at which the same height repeats three times. Initially, the method developed by Vavrik and Carpenter 1998 who introduced commonly used definition that defines the locking point as the first gyration in the first set of three gyrations at the same height that is preceded by two sets of two gyrations at the same height, was utilized in this study. However, it was impossible to obtain the correct result for several specimens, as the sequence 2-2-3 does not necessarily appear in all densification curves.

8.3 Results and Data Analysis

An important improvement over previous study was the consideration of the variability of aggregates, by screening and producing unvarying material for each of the sieves. The materials were screened by sieves to minimize variation of aggregates for each mixture, thereby eliminates aggregate variability as a factor in the results.

Figure 8-1 presents the acceleration data obtained from the mixture 4 (307C) sample 1. The five graphs show the mixture response to compaction at 1, 10, 60, 129 and 150 blows. It can be observed that with every blow applied to the specimen the peak acceleration increased and the impact duration decreased until the locking point was reached at 129 blows. During the initial stage of compaction (blow 1 and 10) there were two acceleration peaks. The acceleration increased from 2305 m/s^2 (235 g) at the first blow up to 4807 m/s^2 (490 g) at the locking point. The duration of impact decreased from 0.0035 s (blow 1) to 0.0013 s (blow 129, i.e., locking point). After that, the peak acceleration varied between 4699 m/s^2 (479 g) and 4817 m/s^2 (491 g), and impact duration between 0.0013 s and 0.0014 s.



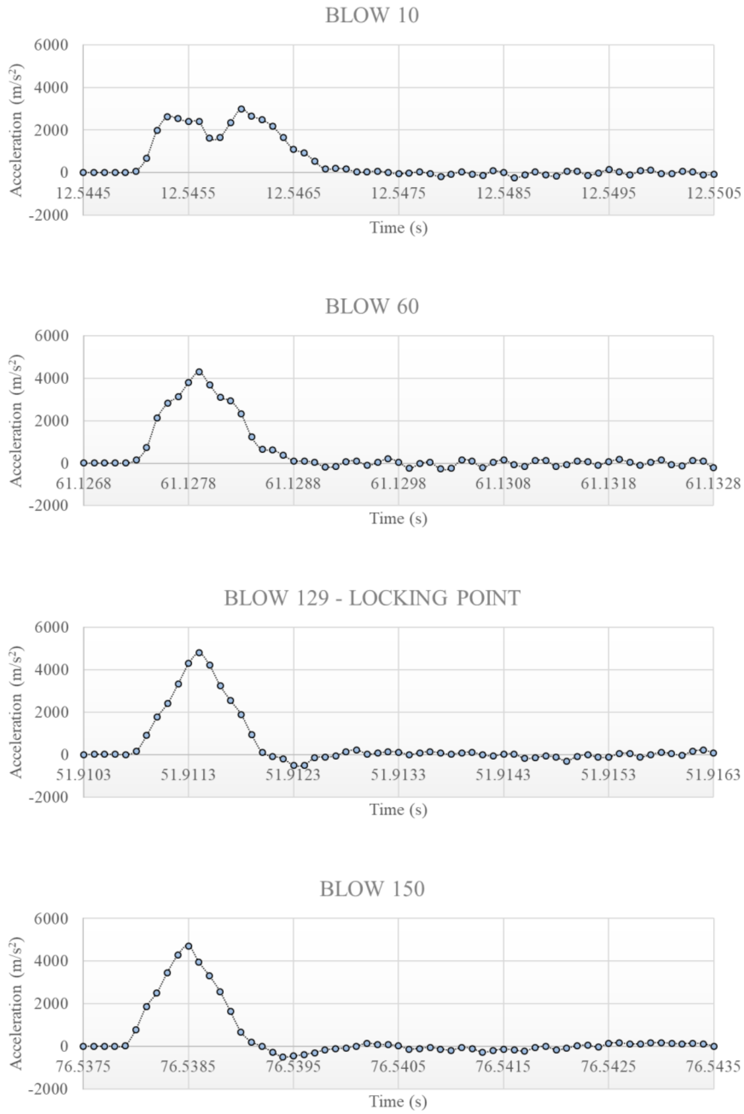


Figure 8-1. Example of the acceleration data and determination of the locking point

Figure 8-2 presents the results of the impact locking point. The number of samples for each mixture was fixed at five, due to the availability of material. The final value of the impact locking point was calculated as the average of the five samples. It was not possible to determine the impact locking point for mixture 1 (307A) or for samples three and four of the mixture 2 (307B), suggesting that the locking point for these mixtures may be higher than 150 blows. However, in the case of mixtures one and two, the failure to reach a locking point may be attributable to the breakage of aggregates, especially the largest coarse aggregates. The specimens of mixture 1 (307A) and 2 (307B) are depicted in Figure 8-3.

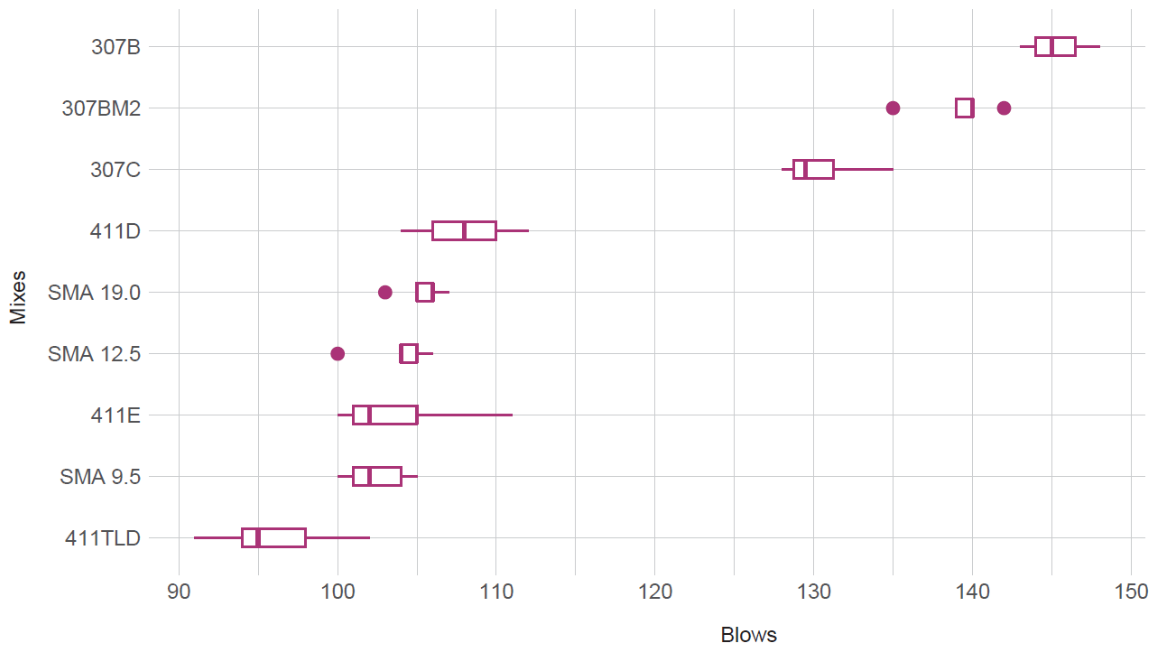


Figure 8-2. The Summary of the Impact Locking Points

Figure 8-3. Example of the Aggregates damage. (A) Mixture One (307A), (B) Mixture two (307B)

The breakage of aggregates, caused by the impact hammer, made it impossible for the aggregates to lock down and yield steady readings of the peak acceleration and impact duration. Once compaction of the specimens was terminated, the samples were warmed and disintegrated with the objective to review the magnitude of damage caused by the impact hammer. To determine breakage, all sides of the aggregates that were not covered with asphalt cement were examined. Post-compaction gradation (sieve analysis) was also performed on aggregate previously washed with the solvent. The comparison of pre and post-compaction sieve analysis is presented in Figure 8-4. The conclusion of the revision was that some of the larger particles suffered a rupture. This phenomenon was particularly visible in mixture 1, which suffered the most extensive damage. The four-inch mold may be inadequate to perform this experiment on mixtures with particles larger than 25.4 mm (1 inch). Also, the limestone aggregates may be too soft to prevent particle damage before compaction is terminated.

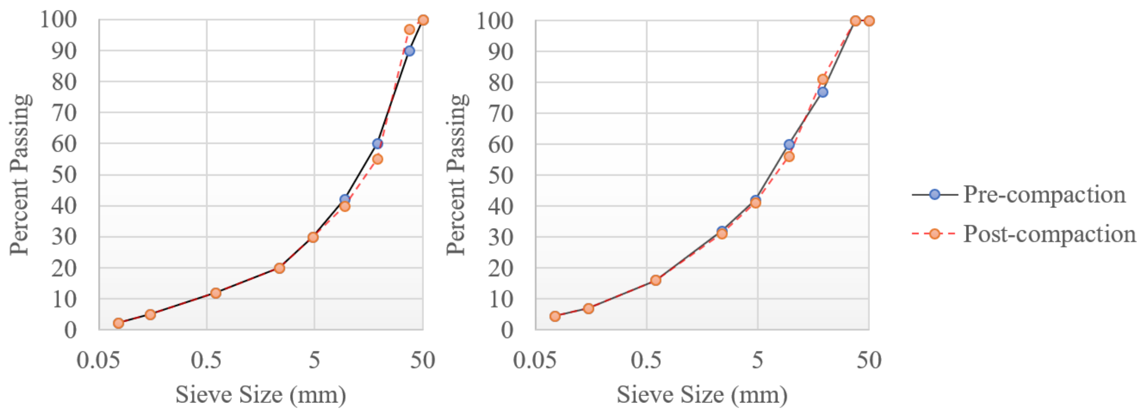


Figure 8-4. Pre and post-compaction sieve analysis. On the left mixture 1 (307A) sample 1. On the right mixture 2 (307B) sample 3

The value of the locking point for mixture 1 has not been concluded. However, for mixture 2, 3 out of 5 samples gave valid results, and the final value was calculated as the average of the three samples. Sample 4 of mixture 4 (307C) was discarded due to human error during the compaction process. For the rest of the mixtures, it was possible to obtain five different samples and calculate average.

The results show that for dense-graded asphalt mixtures, the impact locking point decreases with decreasing maximum aggregate size and range from 145 blows for mixture

2 to 96 blows for mixture 7. Also, it can be concluded that base course asphalt mixtures have higher impact locking point, with an average of 138 blows, while surface asphalt mixtures have lower impact locking points, with an average of 103 blows. The Stone Mastic Asphalt (SMA) mixtures follow the pattern that the impact locking point increases with increasing size of aggregates utilized in the mixture. However, the range of the impact locking point is significantly smaller, from 102 blows for the 9.5 mm mixture to 105 for the 19.0 mm mixture.

The average result of the impact locking point for the base course indicates that it is close to the 150 blows that are commonly used for the Marshall Mix Design of dense-graded mixtures. The difference between the locking point and the final compaction is 12 blows, which means the samples for the mix design are slightly over-compacted. The result obtained for the surface asphalt mixture, 103 blows, indicates that there is a difference of 47 blows between the impact locking point and final compaction. This can lead to significant over-compaction, in extreme cases even damage to aggregates and errors in the mix design. From the results in Figure 8-2, it can be concluded that the average Stone Mastic Asphalt specimen, when compacted at 50 blows for each side, misses only four blows to reach the impact locking point. The results also show that average dense-graded asphalt mixture requires higher compaction energy to achieve the impact locking point than the SMA mixture requires.

Figure 8-5 presents data from 150-mm mold. Similar to the results obtained from impact hammer, the gyratory locking point for the dense-graded asphalt mixtures increases as the maximum aggregate size increases. Again, the base mixtures obtained a higher average gyratory locking point of 80 gyrations than the surface mixtures, which achieved an average gyratory locking point of 47 gyrations. The Stone Mastic Asphalt mixtures had an average gyratory locking point of 94. It can be shown from the gyratory locking points results that when mixture has coarser aggregates then the gyratory locking point is higher. The difference between the locking points for SMA mixture is not as pronounced as for the dense-graded mixtures, where the difference is around one gyration for two blows. The interesting finding is that the gyratory locking point of the SMA is much higher than that of the dense-graded mixture. Also, for the SMA, the compaction effort required to achieve both locking points is similar, 94 gyrations for the gyratory locking point or 103 blows for the impact locking point.

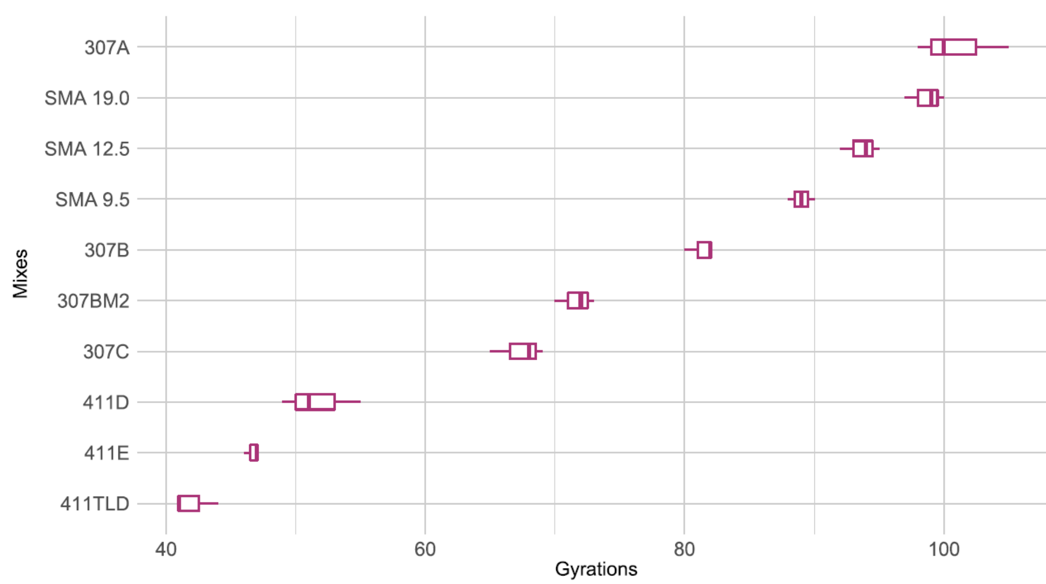


Figure 8-5. The Summary of the Gyratory Locking Points (Mold 150-mm)

Figure 8-6 presents the gyratory locking points obtained from the 100-mm diameter mold. The objective of utilizing different mold size was to analyze how mold size influences the locking points results. Previous research efforts focused on investigation whether differences in mold size affect compaction and properties of the asphalt mixture. Hall et al. 1996 investigated how the size of the sample affects compaction and volumetric properties of asphalt mixtures. The study utilized the same 150 mm mold for all the samples, and the weight of the samples varied from 2000 g to 6500 g. Hall concluded that compaction characteristics and volumetric properties change with the size of the sample. McGennis et al. 1996 presented the hypothesis that for the same mixture and the same number of gyrations, the resulting compaction should be the same for both molds. Two 12.7 mm (½") and five 19.1 mm (¾") nominal maximum size aggregate mixtures were tested. Specimens were prepared at the optimum asphalt content in the 150-mm and the 100-mm gyratory molds. Specimens specific gravities from the two mold sizes were compared. More than half of the results rejected this hypothesis. Jackson and Czor 2003 evaluated the use of the 100-mm mold to prepare test samples in the laboratory by collecting different mixes and comparing relative density obtained with the 100-mm mold and the 150-mm mold. This work found the significant statistical differences between results obtained from two sizes of molds. Nonetheless, the recorded differences were smaller than could be measured given the precision of the laboratory test methods, and the researchers concluded that there is no significant difference from an engineering point of view.

In the current study, the specimens compacted in 100-mm mold had the same aggregate and binder properties as samples compacted with the Marshall Hammer and 150 mm SGC molds. The gyratory locking point for the 150-mm specimens was significantly higher than that of the 100-mm specimens, which means more compaction efforts will be required to achieve skeleton structure in asphalt mixture if larger molds are used for specimen compaction. However, even with this significant difference in values of gyratory locking points, the results indicated that the rankings of mixtures for the two different mold sizes were identical. These rankings support the conclusion that the concept of locking point can discriminate the compaction properties of one mixture from another no matter which mold size is used.

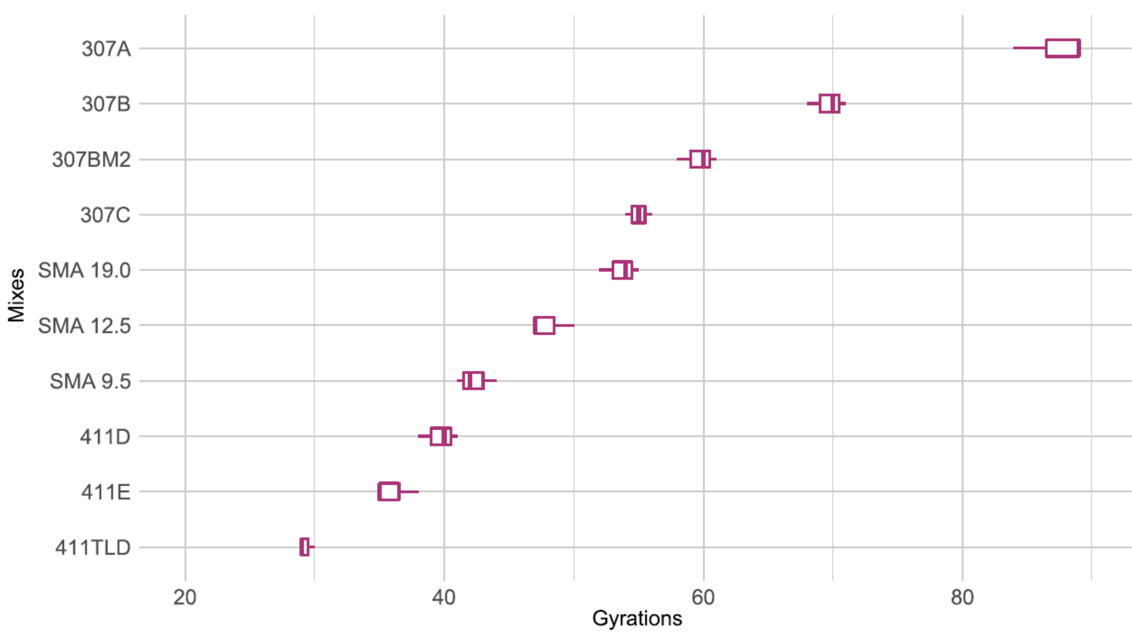


Figure 8-6. The Summary of the Gyratory Locking Points (Mold 100-mm)

Figure 8-7 presents a summary of all the locking points obtained during this research. It can be observed that the gyrations are more efficient in achieving the locking point than impact regardless of the mold used. However, the difference is higher for dense-graded mixtures. For all the mixtures, the ranking of locking points was maintained, but again for dense-graded mixtures, the difference was more significant than for SMA. The small difference in locking points for SMA may be attributed to the granulometric composition of these mixtures, where the finer sieves do not vary. These results may indicate that the finer particles of granulometric composition have a more significant influence on the locking point than courser aggregates.

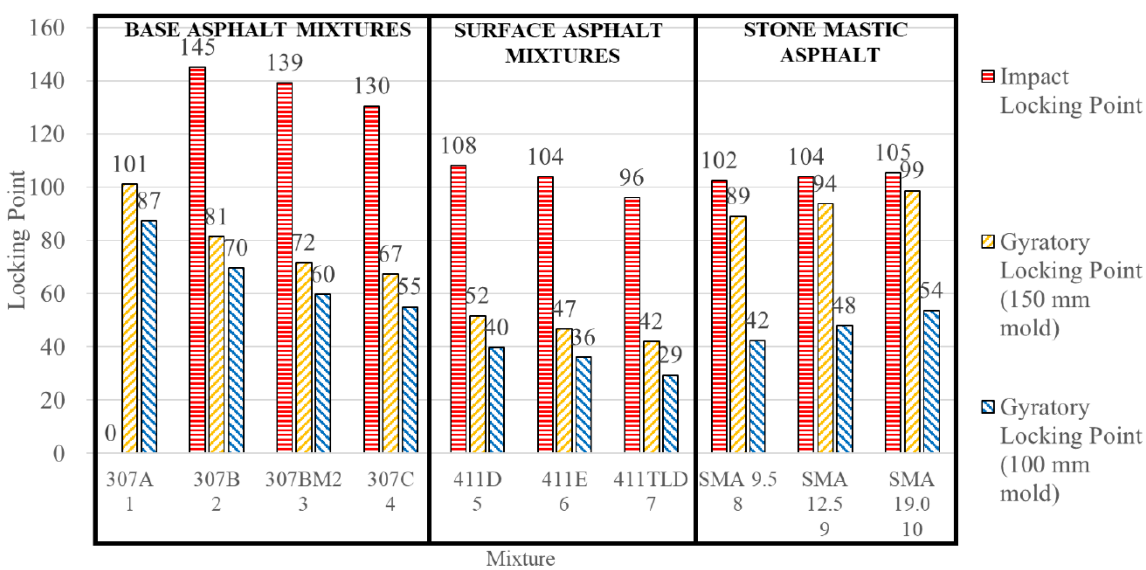


Figure 8-7. The Summary of the Locking Points

8.4 Conclusions

In this section, two compaction methods, the SGC and Marshall Compactor, were used to investigate the locking points of asphalt mixtures. Ten different asphalt mixtures were produced with the same aggregates pre-screened with sieves to eliminate the influence of aggregate gradation. The specimens were compacted utilizing a Marshall Hammer with an accelerometer attached to the falling mass and then with the SGC. The response of specimens obtained by the accelerometer and the change in specimen height in the SGC compaction were measured to determine impact and gyratory locking points for each mixture. The results indicate that the locking point is related to the full gradation of the blend consisting of coarse and fine aggregates. Dense-graded mixtures show a wider range in locking points than SMA mixtures. This difference may be attributed to granulometric variation. Dense-graded mixtures vary in both coarse and fine aggregates whereas SMA mixtures vary mostly in coarse aggregate. Dense-graded mixtures with smaller maximum aggregate size had a lower locking point. This finding may be attributed to the fact that these mixtures generally contain more asphalt binder, which serves as a lubricant and make compaction easier and faster. In this study one type of asphalt binder was used. The locking point of base mixtures was slightly under 150 blows, the common number of blows in the Marshall Mix Design. The SMA had the impact locking point just above 100 blows, again the common number in the mix design. In contrast, surface mixtures showed an impact locking point far below 150 blows. This finding indicates that surface mixtures may suffer overcompaction and aggregate breakage when 150 blows are used for mix design. The gyratory locking points for the 150-mm specimens were significantly higher than those for 100-mm specimens, but the ranking is maintained regardless mold size. This finding allows the comparison of the locking points for different mixtures if the same size of mold is utilized. In the current study, all comparisons were made with the samples of the same weight. Further study is needed to determine if the weight (height) of specimen will have influence on the locking point.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

In this research project, a wide range of TDOT typical asphalt mixtures were collected, designed and evaluated with the purpose of transferring the mix design method from Marshall to Superpave. The advantages and disadvantages of current TDOT mix design were compared to Superpave method in order to combine the benefits of both methods, which included the usage of the 150-mm mold to design larger stone mixtures and the 100-mm mold to design surface mixtures with the maximum aggregate size less than 1 in.. Additionally, the concept of the locking point was investigated and a new method of determining the impact locking point was developed. Based on the literature review and the results of the laboratory tests, the following conclusions can be summarized:

(1) Superpave can effectively eliminated the rutting distress, whereas at the same time it can introduce the issues such as increased permeability, being harder to compact, and the effect of variations in gradation. However, TDOT can benefit from the experience of other agencies to smoothly switch to Superpave.

(2) All the Tennessee neighbor states utilize the modified version of Superpave based on their experience and best practice with Marshall method. Most neighbors use a modified gradation that is a combination of Superpave control points and Marshall gradations. Most of the states use modified consensus aggregate

properties based on Marshall specifications, and only Arkansas uses the original AASHTO R35 criteria. Some states use just one value for all the mixtures, when others use different values for different traffic load, mix type, or asphalt binder grade.

(3) Most of the states made modifications to N_{design} included in AASHTO R35. Currently, only thirteen states use AASHTO R35 compaction effort and only two use values recommended by NCHRP 573.

(4) The plant mixtures resulted in a high variation of the results of the equivalent N_{design}. The range of equivalent N_{design} for D mixtures compacted in 150-mm mold was from 38 to 77 gyrations and from 32 to 86 gyrations for mixture compacted in 100-mm mold. For BM-2 mixtures the range of Ndesign was from 39 to 75 gyrations for 150-mm mold, and from 42 to 73 gyrations for 100-mm mold. The difference in asphalt content and the percent of particles that pass sieve No. 200 could affect the results of equivalent N_{design} obtained in this study. Therefore, it was necessary to repeat the process using laboratory mixtures.

(5) The gradation of TDOT's D and BM2 mixes are close to Superpave 12.5 mm and 25 mm. Based on the implementations of the Superpave mix design in other states, it can be suggested that TDOT can choose any of the three options for aggregate gradation: 1) keeping current grading table, 2) implementing

Superpave control points, or 3) making small modification to TDOT grading tables.

(6) Most of the states use a modified list of consensus aggregate properties, especially regarding to the fine aggregate angularity. The precision of fine angularity test is critical for bulk specific gravity (G_{sb}). Some agencies chose performance tests or simply reduce the limit of natural sand from 0% to 15% instead of the fine aggregate angularity.

(7) For laboratory mixtures, the N_{design} was defined based on the nine well-performing mixtures provided by TDOT. The results of back calculations showed that for BM2 mixes the range of equivalent N_{design} was from 71 to 75 gyrations (average 73 gyrations), while for D mixtures the range was from 64 to 72 gyrations (average 68 gyrations) by using 150-mm mold.

(8) For all the mixtures included in this study, a higher N_{design} yielded lower asphalt binder content and slightly lower moisture resistance (TSR). D-mixes have significantly higher moisture resistance (average 91%) than BM-2 mixes (average 81%). Mixtures 1160307 and 3160011 designed with N_{design} equal to 75 gyrations were outside the TDOT specifications for moisture damage set as 80% minimum.

(9) The asphalt mixture designed with different N_{design} followed the general trend of viscoelastic materials; namely, its dynamic modulus decreased with the increase in temperature and increased with the increase in loading frequency. The results

of flow number indicated that higher N_{design} will provide better rut resistance for the mixtures. The performance results at different N_{design} exhibited less variation than the results using different binders.

(10) The resilient modulus increases when increasing N_{design} for all the mixtures included in this study, which could be attributed to the higher asphalt content for mixes with lower N_{design}. Similarly, all the mixtures showed higher failure stress with higher N_{design}. The samples prepared with lower N_{design} represented a larger strain when failed, indicating that higher asphalt content increased the ductility of asphalt mixtures. Samples using lower N_{design} generally had higher $DCSE_f$, which indicates that lower N_{design} provided better resistance to failure as it required more energy to fracture asphalt mixture.

(11) All except one mixture presented higher rutting depth with lower N_{design}. Mixture 1160307, during the first 16,000 passes presented similar trend as others, but at the end the mix with N_{design}=75 gyrations obtained higher rut depth. It can be attributed to the moisture damage, as the stripping inflection point could be observed. Stripping inflection points appeared for all three BM2 mixtures designed with 75 gyrations.

(12) The accelerometer can determine different stages in the impact compaction process and to obtain the impact locking point. The impact locking point can be determined as the point where the acceleration and the response duration become

stable. The gyratory locking points for the 150-mm specimens were significantly higher than those for 100-mm specimens, but the ranking for different mixtures kept unchanged regardless of the mold size. This finding allows the comparison of the locking points for different mixtures if the same size of mold is utilized.

(13) Superpave mix design is generally utilizing the 150-mm (~6”) mold for mix design purpose, even that the 100-mm (~4”) mold can accommodate the aggregate smaller than 1”. In this study, following the Superpave specification, limited research was performed on utilizing the 100-mm mold for D mixes design, which resulted in the equivalent N_{design} of 49 gyrations. It is lower than the equivalent N_{design} of 68 gyrations by using the 150-mm mold.

On the basis of the conclusions obtained in this study, the following recommendations can be made:

(14) The utilization of the 100-mm (~4”) mold for Tennessee surface mixtures has the potential to decreases the aggregate and asphalt binder consumption required for mix design, which can reduce the cost of mix design as well as the exposure of technicians to heavy specimens. This potential should be investigated in the future.

(15) Based on the other state experiences, the field trial sections should be constructed, monitored and analyzed to detect possible issues during plant production,

placement, compaction and the early service life. The field trial section could also serve to develop construction specifications or/and modifications of existing specifications.

(16) It is recommended to continue the research to determine the required compaction effort for the other types of mixtures not included in this study, such as A, B, C, and SMA.

(17) TDOT is one of the few agencies that use the effective specific gravity to calculate VMA. From the experience of the other states, at the initial stage of Superpave implementation, it is important to have the precise values of VMA during mix design, plant production and compaction. It is recommended to conduct statewide workshops and an interlaboratory test program to determine the precision of each laboratories that will conduct Superpave mix designs and test G_{sb} to calculate VMA.

(18) The implementation of new mix design requires a statewide campaign to inform contractors and TDOT personal about advantages and challenges of the new method.

REFERENCE

American Association of State Highway Transportation Officials. (2017). AASHTO M323: Standard Specification for Superpave Volumetric Mix Design

American Association of State Highway Transportation Officials. (2017). AASHTO R35: Standard Practice for Superpave Volumetric Design for Asphalt Mixtures

American Association of State Highway Transportation Officials. (2015). AASHTO T166: Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens. American Association of State Highway and Transportation Officials

American Association of State Highway Transportation Officials. (2015). AASHTO T209: Theoretical Maximum Specific Gravity and Density of Hot Mix Asphalt. American Association of State Highway and Transportation Officials

American Association of State Highway Transportation Officials. (2015). AASHTO T245: Standard Method of Test for Resistance to Plastic Flow of Asphalt Mixtures Using Marshall Apparatus

American Association of State Highway Transportation Officials. (2014). AASHTO T283: Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage

American Association of State Highway Transportation Officials. (2019). AASHTO T324: Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures

American Society of Testing and Materials. (2017). ASTM C117-17: Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing

American Society of Testing and Materials. (2015). ASTM C127-15: Standard Test Method for Relative Density (Specific Gravity) and Absorption of Coarse Aggregate

American Society of Testing and Materials. (2015). ASTM C128-15: Standard Test Method for Relative Density (Specific Gravity) and Absorption of Fine Aggregate

American Society of Testing and Materials. (2019). ASTM C136-19: Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates

American Society of Testing and Materials. (2017). ASTM C1252-17: Standard Test Methods for

Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading)

American Society of Testing and Materials. (2014). ASTM D2419-14: Standard Test Method for Sand Equivalent Value of Soils and Fine Aggregate

American Society of Testing and Materials. (2019). ASTM D4791-19: Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate

American Society of Testing and Materials. (2013). ASTM D5821-13: Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate

Anderson, R. M., P. A. Turner, and R. L. Peterson. (2002). NCHRP Report 478: Relationship of Superpave Gyratory Compaction Properties to HMA Rutting Behavior. TRB, National Research Council, Washington, D.C.

Asphalt Institute. (2007). The Asphalt Handbook MS-4, 7th Edition. Asphalt Institute, Lexington, Kentucky.

Asphalt Institute. (2014). "MS-2 Asphalt Mix Design Methods." Asphalt Institute, Lexington, Kentucky.

Bahia, H., T. Friemel, P. Peterson, J. Russel and B. Poehnelt. (1998) Optimization of Constructability and Resistance to Traffic: A New Design Approach for HMA Using the Superpave Compactor. Asphalt Paving Technology, Journal of the Association of Asphalt Paving Technologists, Vol. 67, pp. 189-232.

Brown, E., Decker, D., Mallick, R., and Bukowski, J. (1999). "Superpave Construction Issues and Early Performance Evaluation." Annual Meeting of the Association of Asphalt Paving Technologists.

Brown, R., Kandhal, P., Roberts, F., Kim, R., Lee, D., and Kennedy, T. (2009). Hot Mix Materials, Mixture, Design, and Construction. NAPA Research and Education Foundation, Lanham, Maryland.

Cassidy, M., Spellerberg, P., and Savege, D. (1994). "A System for Calibration of the Marshall Compaction Hammer." National Institute of Standards and Technology, 5338.

Chadbourn, B., Skok, E., Newcomp, D., Crow, B. and Spindler, S. (1999) The Effect of Voids in Mineral Aggregate (VMA) on Hot-Mix Asphalt Pavements. Technical Raport Minnesota Department of Transportation

Choubane, B., Gale, P., and Musselman, J. (1998). "Investigation of Water Permeability of Coarse-Graded Superpave Mixes." AAPT, 67.

Clegg, B. An Impact Testing Device for in situ Base Course Evaluation. (1976). ARRB Proceedings, 8, 8, 1–6.

Clegg, B. (1978). “An Impact Soil Test for Low Cost Roads.” Proceedings from Second Conference Road Engineering Associates of Asia and Australia., 58–65.

Clegg, B. (1980). “An Impact Soil Test as Alternative to California Bearing Ratio.” Third ANZ Geomechanics Conference, 1–15.

Clegg, B. (1983). “Application of an Impact Test to Field Evaluation of Marginal Base Course Materials.” Transportation Research Record, 898, 174–181.

Cominsky, R., Huber, G., Kennedy, T., and Anderson, M. (1994). The Superpave Mix Design Manual for New Construction and Overlays. Strategic Highway Research Program. Washington, DC

Federal Highway Administration. Superpave Fundamentals Reference Manual. NHI Course #131053

Fishbaugh, W. (2016). “Road construction workers laying bitulithic pavement.” Florida Memory, <https://www.floridamemory.com/items/show/165432> (Mar. 13, 2016).

Georgia Department of Transportation. Standard Specifications Construction of Transportation Systems, 2013.

Hall, K., Dandu, S. and Gowda G. (1996). Effect of Specimen Size on Compaction and Volumetric Properties in Gyratory Compacted Hot-Mix Asphalt Concrete Specimens. In Transportation Research Record: Journal of the Transportation Research Board, No. 1545, Transportation Research Board of the National Academies, Washington, D.C., pp. 126–132

Harman, T., Bukowski, J. R., Moutier, F., Huber, G., and McGennis, R. (2002). History and Future Challenges of Gyratory Compaction: 1939 to 2001. Transportation Research Record, 1789(1), 200–207. https://doi.org/10.3141/1789-22

Huber, G. (2016). “History of asphalt mix design in North America, Part 1.” Asphalt magazine, <http://asphaltmagazine.com/history-of-asphalt-mix-design-in-north-america-part-1/> (Nov. 19, 2016).

Jackson, N. and Czor, L. (2003). 100-mm-Diameter Mold Used with Superpave Gyratory Compactor. ASCE Journal of Materials in Civil Engineering. Volume 15, Issue 1, pp. 60-66.

Leahy, R., and Briggs, R. (2001). “Washington State Department of Transportation Superpave Implementation.” Transportation Research Record, 1761, 32–40.

Lee, R. and Hoelscher, P. (2014). Experience with Superpave Mixtures. 2014 Construction Conference. Texas Department of Transportation

Mallick, R. (1999) Use of Superpave Gyratory Compactor to Characterize Hot-Mixture Asphalt. In Transportation Research Record: Journal of the Transportation Research Board, No. 1681, Transportation Research Board of the National Academies, Washington, D.C., pp. 86-96.

McGennis R., Anderson, R., Perdomo, D. and Turner, P. (1996) Issues Pertaining to Use of Superpave Gyratory Compactor. In Transportation Research Record: Journal of the Transportation Research Board, No. 1543, Transportation Research Board of the National Academies, Washington, D.C., pp. 139–144.

Mohammad, L. and K. Shamsi. (2007) A Look at the Bailey Method and Locking Point Concept in Superpave Mixture Design. Transportation Research Circular E-C124: Practical Approaches to Hot-Mix Asphalt Mix Design and Production Quality Control Testing. Transportation Research Board, pp. 12-32.

Musselman, J., Choubane, B., Page, G., and Upshaw, P. (1997). “EXPERIENCE WITH SUPERPAVE IMPLEMENTATION IN 1996.” Research Report FL/DOT/SMO/97-415.

N. A. P. A. (2016). “History of Asphalt .” History of Asphalt , <http://www.asphaltpavement.org /index.php?option=com_content&task=view&id=21&Itemid=41> (Nov. 14, 2016).

National Cooperative Highway Research Program. (2003). NCHRP 513: Simple Performance Tester for Superpave Mix Design: First-Article Development and Evaluation

National Cooperative Highway Research Program. (2004). NCHRP 530: Evaluation of Indirect Tensile Test (IDT) Procedures for Low-Temperature Performance of Hot Mix Asphalt.

National Cooperative Highway Research Program. (2007). NCHRP 573: Superpave Mix Design: Verifying Gyration Levels in the N_{design} Table

Polaczyk P., B. Han, B. Huang, X. Jia and X. Shu. (1998) Evaluation of the hot mix asphalt compactability utilizing the impact compaction method, *Construction and Building Materials*. 187: 131-137, DOI: 10.1016/j.conbuildmat.2018.07.117

Polaczyk, P., X. Shu, H. Gong, and B. Huang. (2019a) Influence of aggregates angularity on the locking point of asphalt mixtures, *Road Materials and Pavement Design*, 20:sup1, pp. S183-S195, DOI: 10.1080/14680629.2019.1588151

Polaczyk, P., B. Huang, X. Shu, and H. Gong. (2019b). Investigation into Locking Point of Asphalt Mixtures Utilizing Superpave and Marshall Compactors, *Journal of Materials in Civil Engineering*,

2019. 31, 9. DOI: 10.1061/(ASCE)MT.1943-5533.0002839

Siddiqui, Z., Trethewey, M., and Anderson, D. (1988). "Variables Affecting Marshall Test Results." Transportation Research Record, 1171, 139–148.

Tennessee Department of Transportation. (2015). Standard Specifications for Road and Bridge Construction

Transportation Research Board (2002). "Method Summarized for Gradation Selection in Hot-Mix Asphalt Design." Transportation Research Circular Number E-C044.

Transportation Research Board (2005). SUPERPAVE Performance by Design. FINAL REPORT OF THE TRB SUPERPAVE COMMITTEE

Vavrik, W. and S. Carpenter. (1998) Calculating Air Voids at Specified Number of Gyrations in Superpave Gyratory Compactor. In Transportation Research Record: Journal of the Transportation Research Board, No. 1630, Transportation Research Board of the National Academies, Washington, D.C., pp. 117–125.

Willoughby, K., Weston, J., Pierce, L., and Mahoney, J. (2004). "Comparison of Performance and Cost Between Superpave and Conventional Hot-Mix Asphalt in Washington State." Journal of the Transportation Research Board, 1869, 32–40.

APPENDIX A: Dynamic Modulus and Phase Angle Results

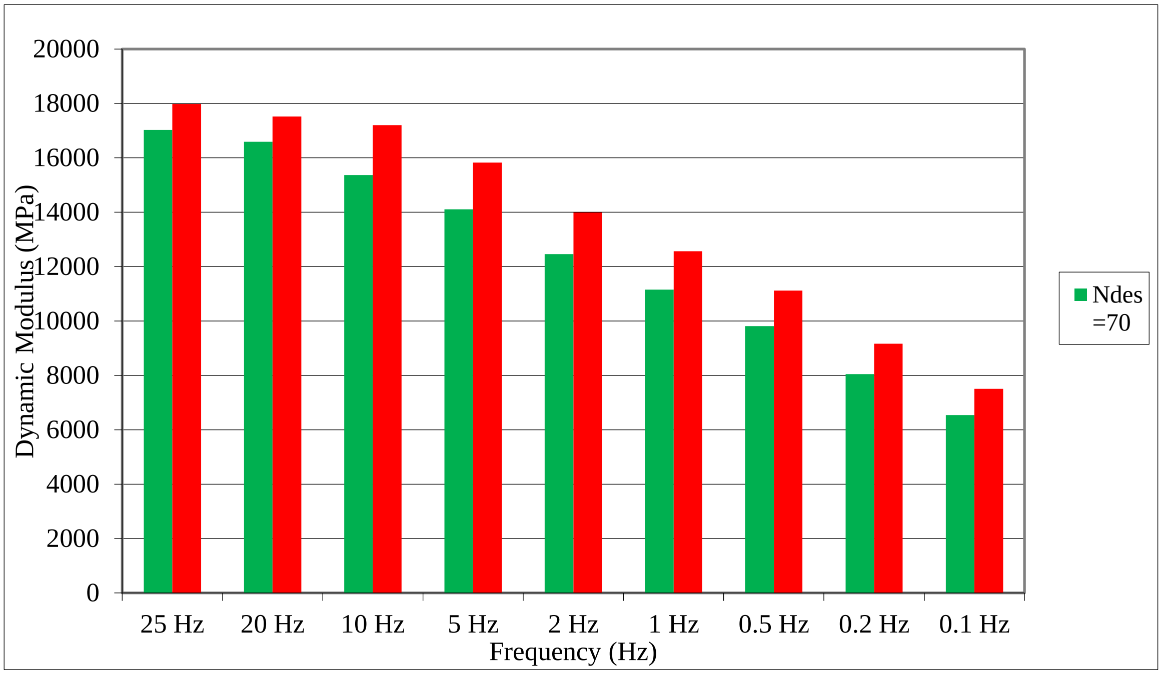


Figure A-1 Dynamic Modulus of Mixture 1 (1160307) at 4°C

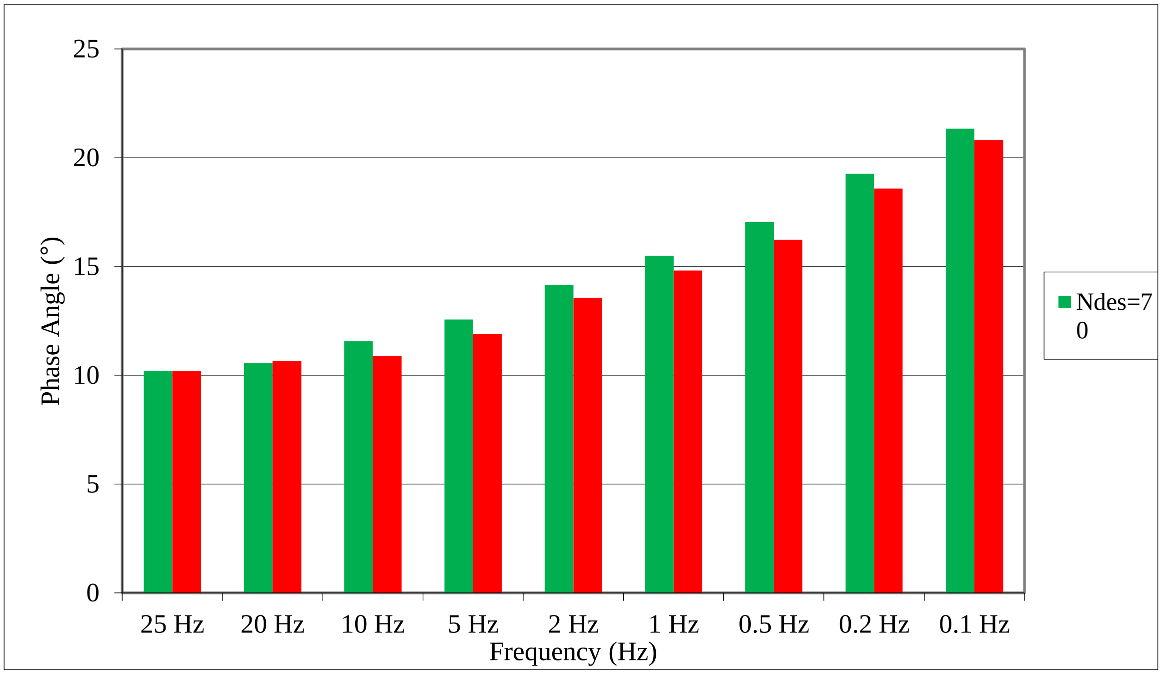


Figure A-2 Phase Angle of Mixture 1 (1160307) at 4°C

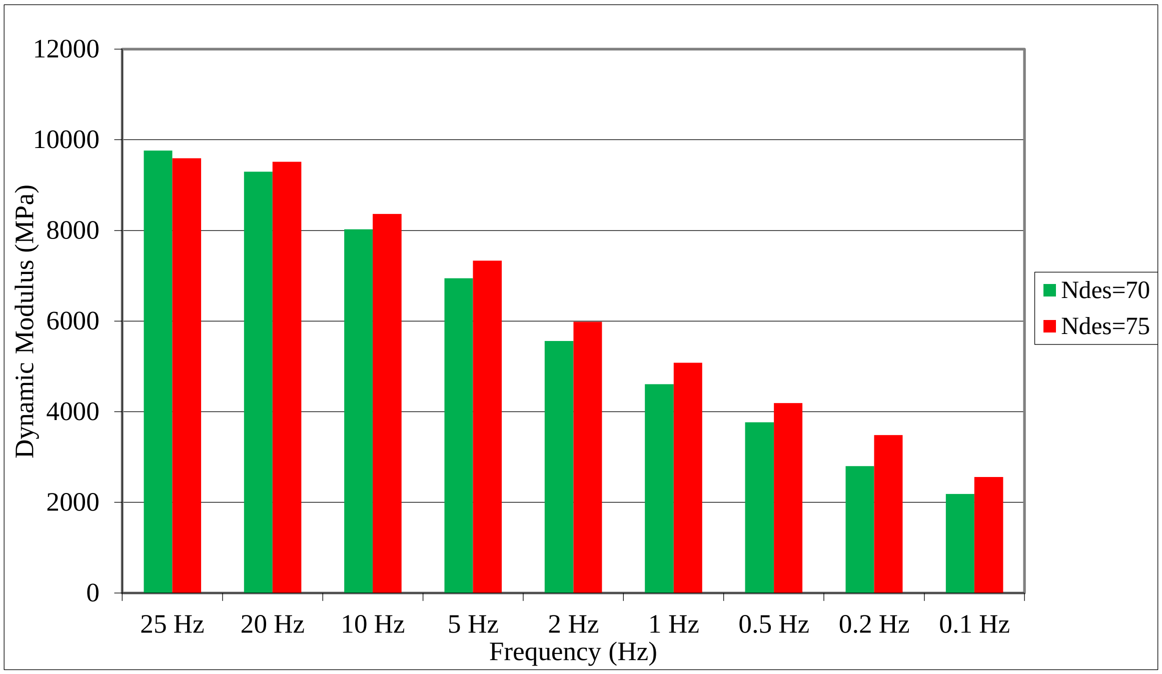


Figure A-3 Dynamic Modulus of Mixture 1 (1160307) at 20°C

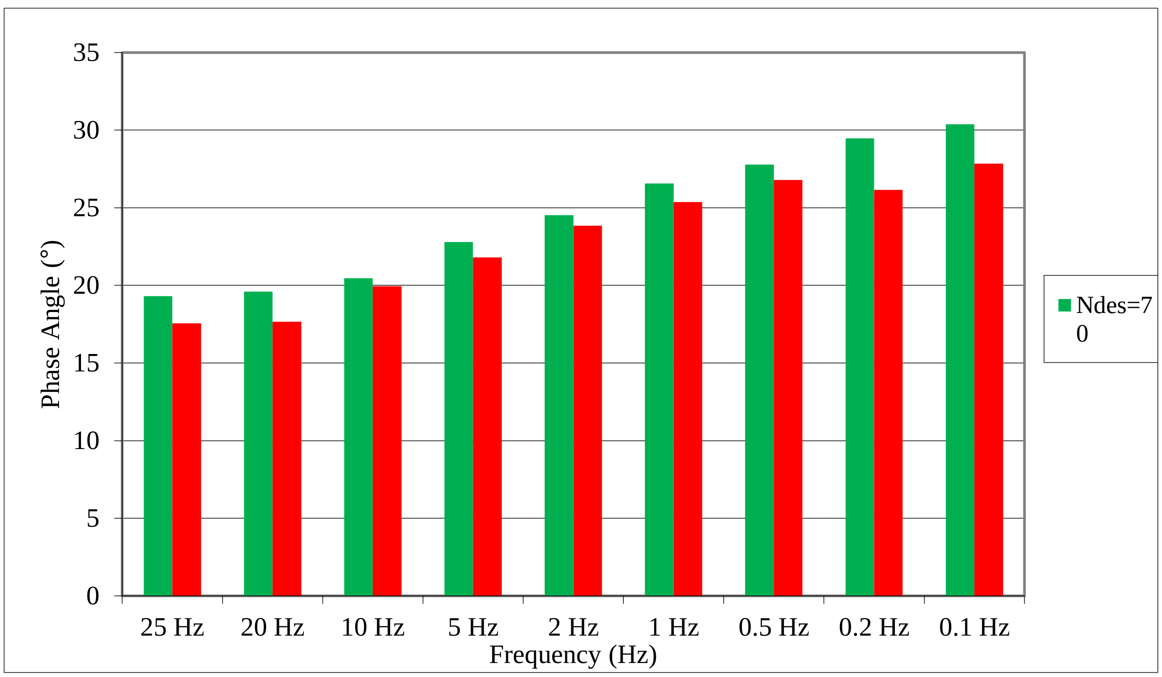


Figure A-4 Phase Angle of Mixture 1 (1160307) at 20°C

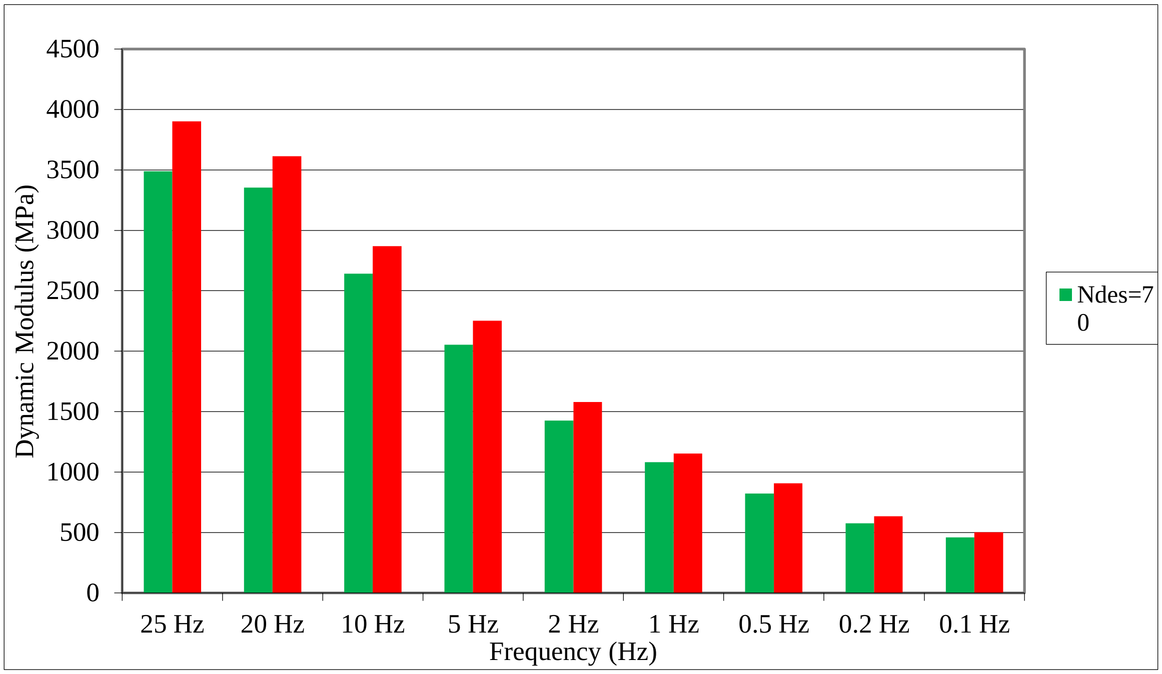


Figure A-5 Dynamic Modulus of Mixture 1 (1160307) at 40°C

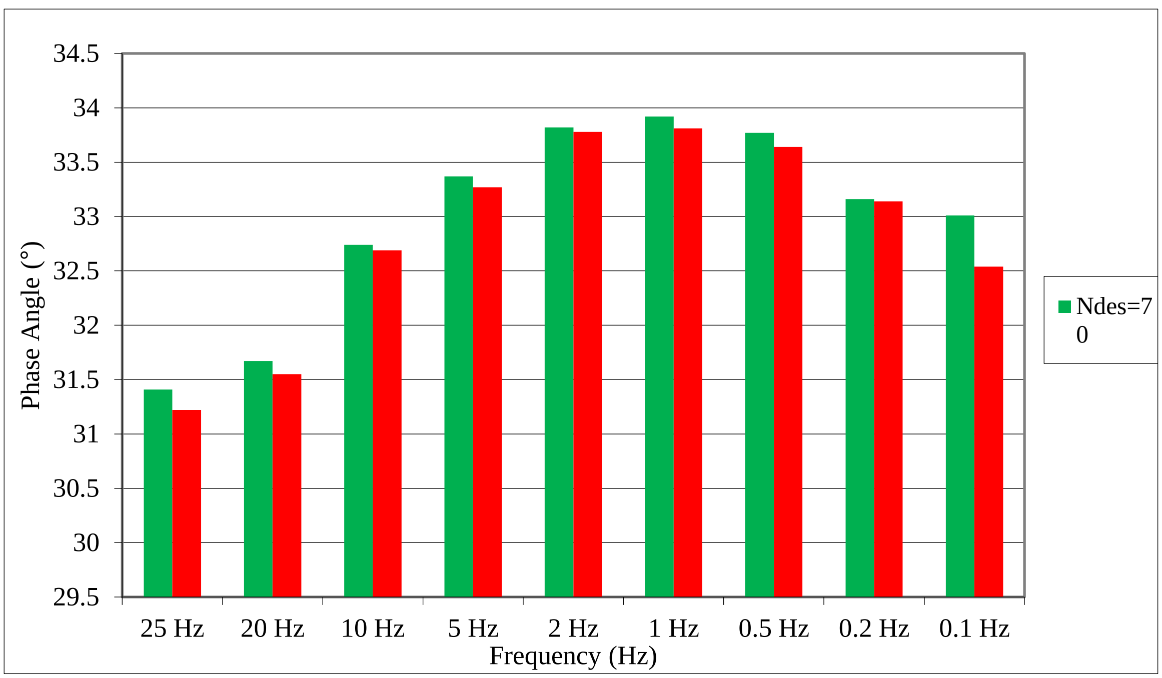


Figure A-6 Phase Angle of Mixture 1 (1160307) at 40°C

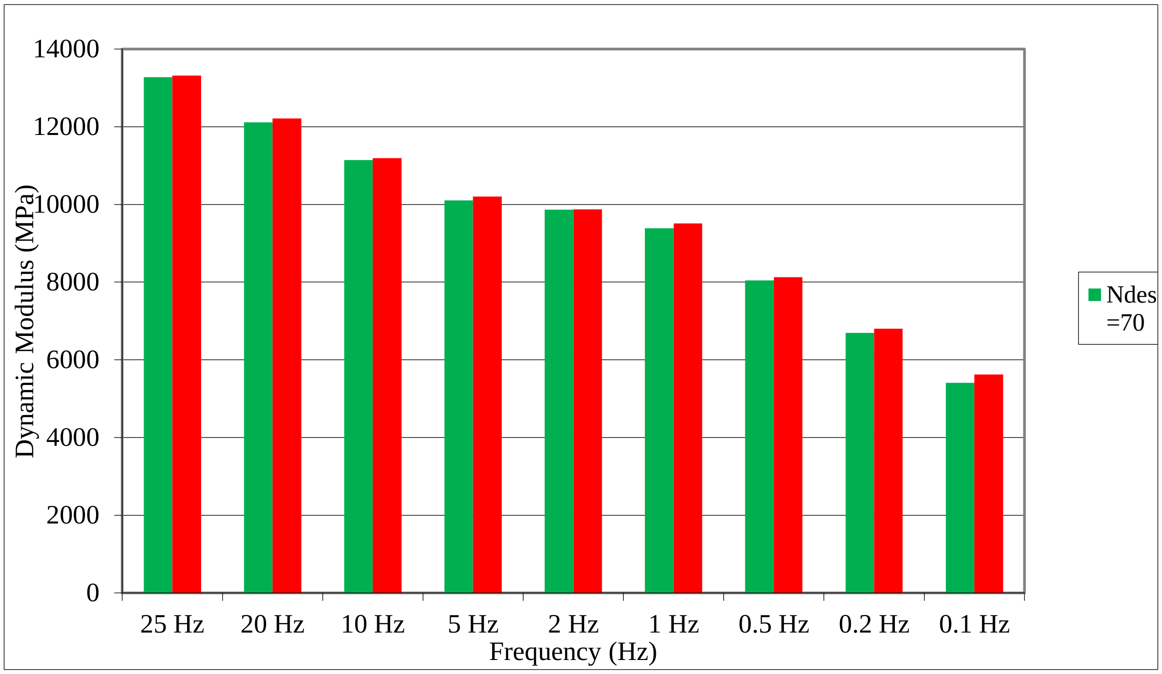


Figure A-7 Dynamic Modulus of Mixture 2 (3160011) at 4°C

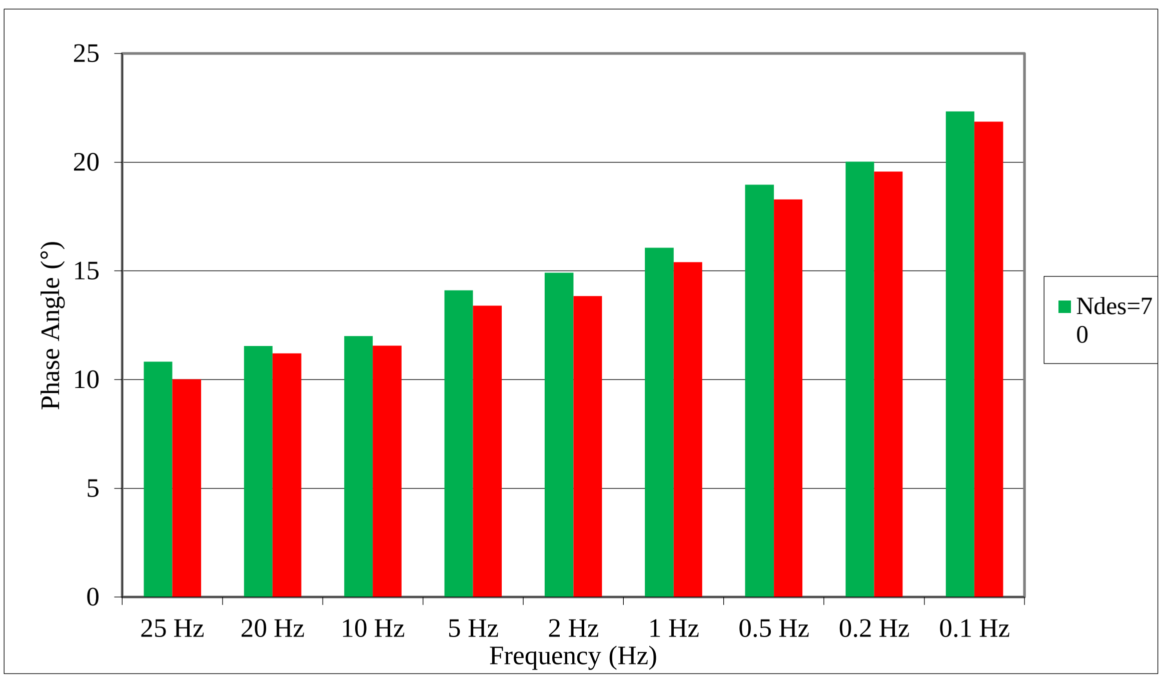


Figure A-8 Phase Angle of Mixture 2 (3160011) at 4°C

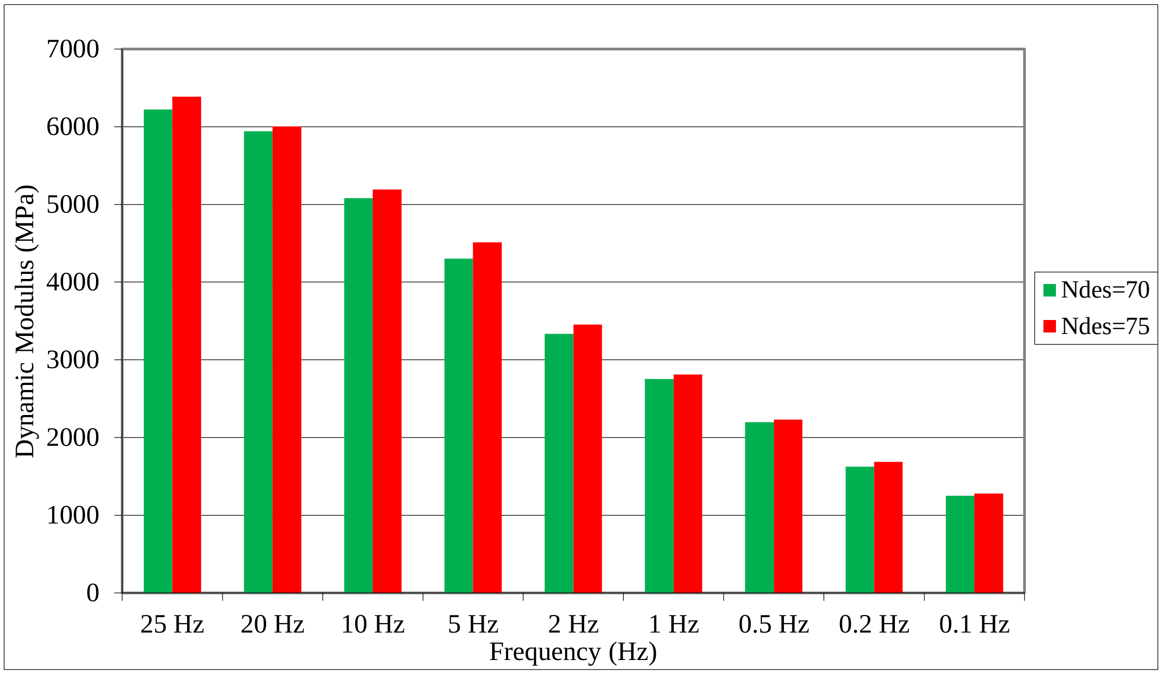


Figure A-9 Dynamic Modulus of Mixture 2 (3160011) at 20°C

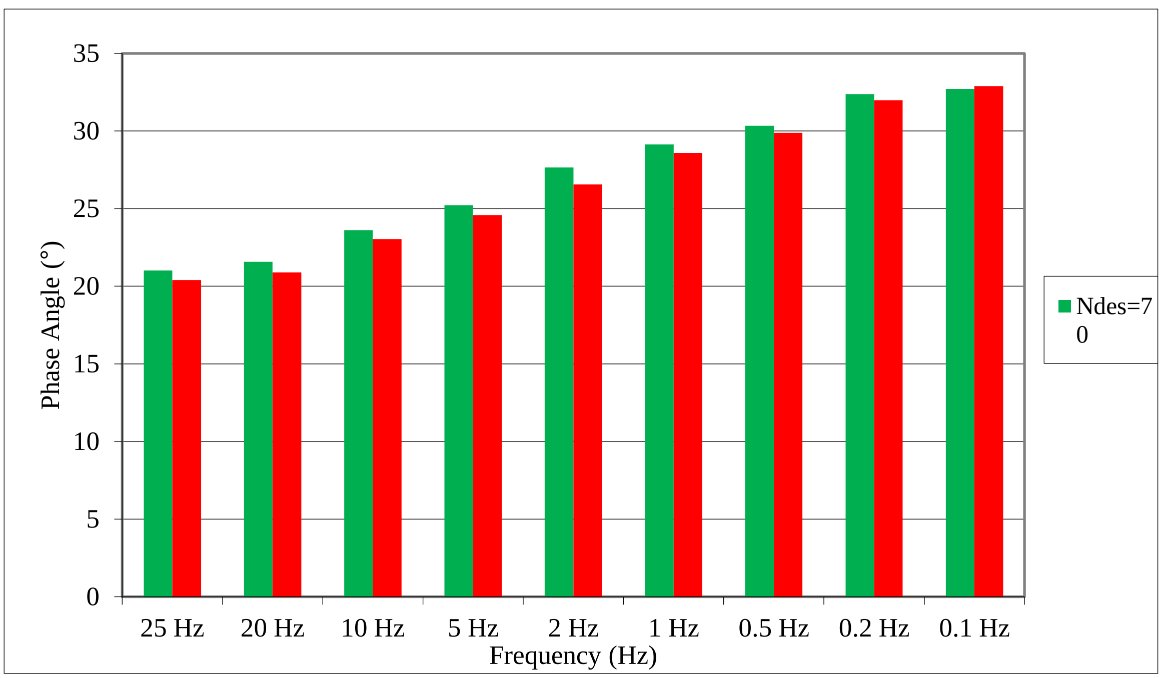


Figure A-10 Phase Angle of Mixture 2 (3160011) at 20°C

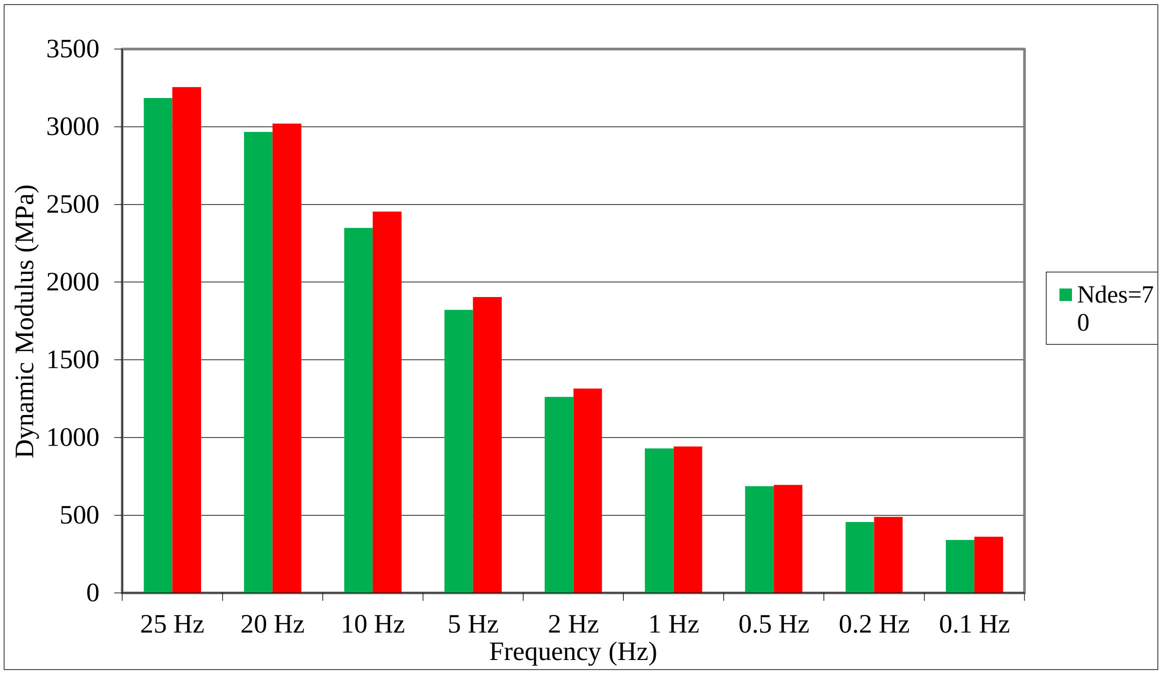


Figure A-11 Dynamic Modulus of Mixture 2 (3160011) at 40°C

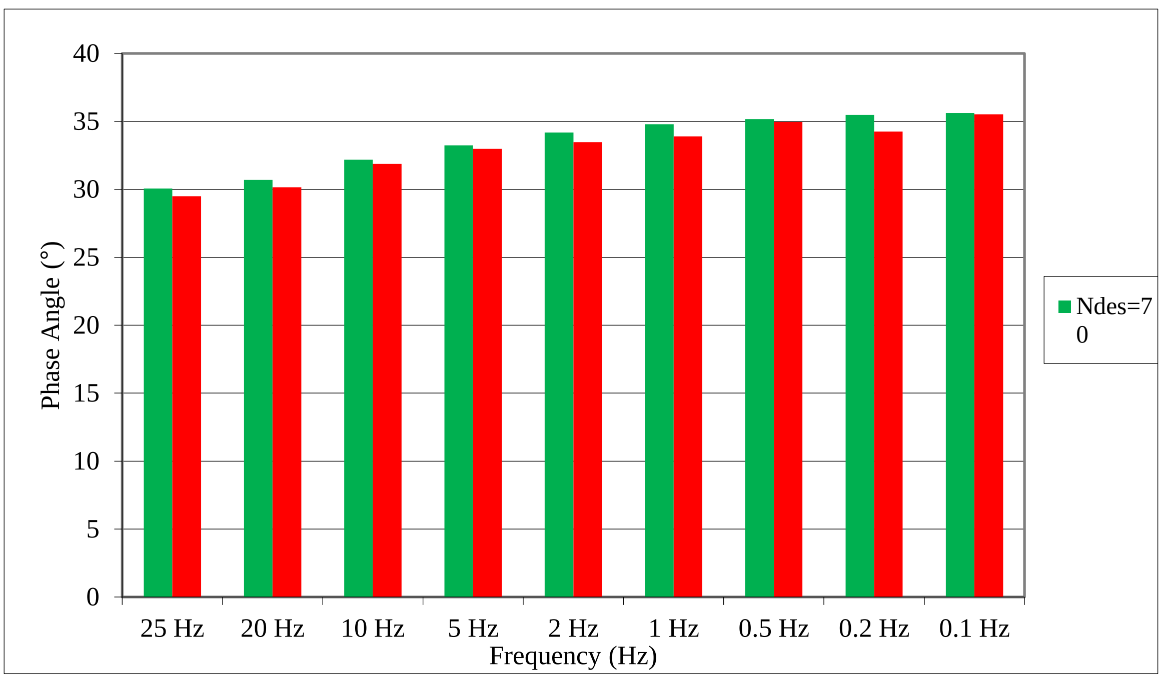


Figure A-12 Phase Angle of Mixture 2 (3160011) at 40°C

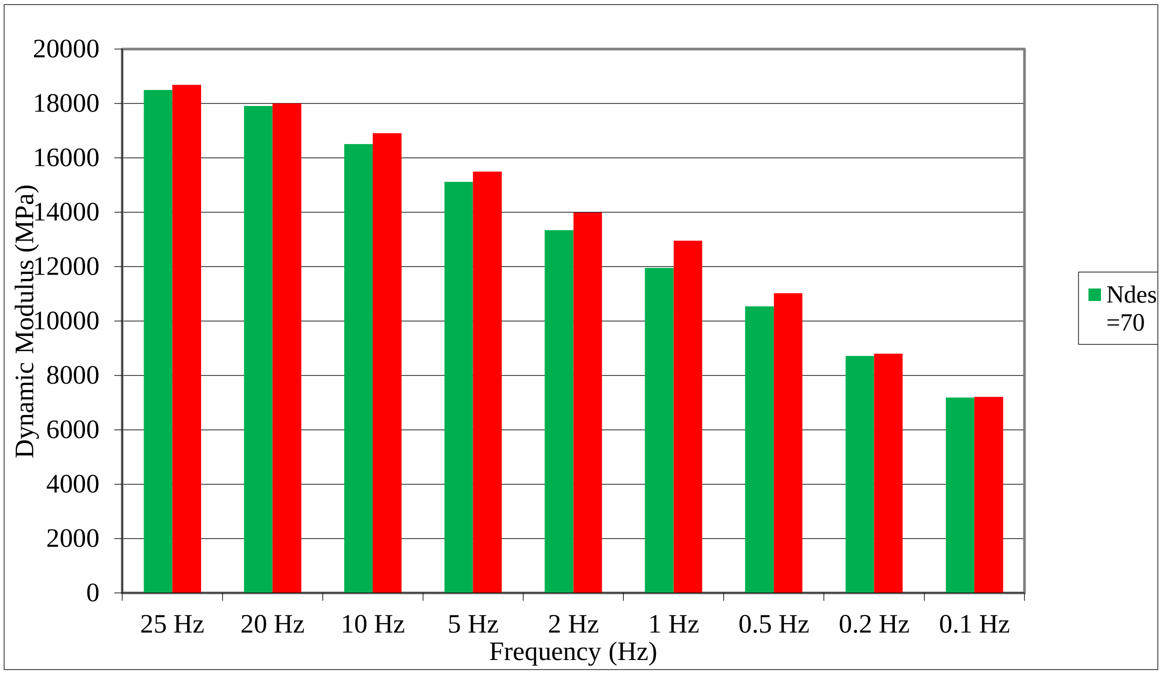


Figure A-13 Dynamic Modulus of Mixture 3 (4160056) at 4°C

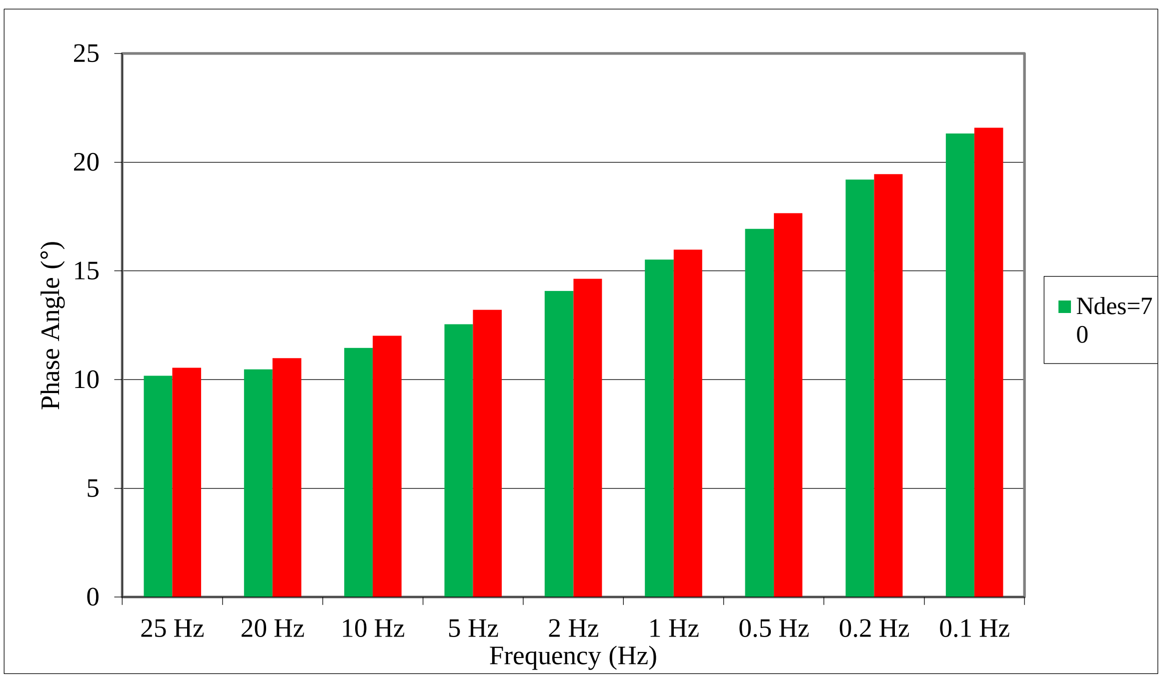


Figure A-14 Phase Angle of Mixture 3 (4160056) at 4°C

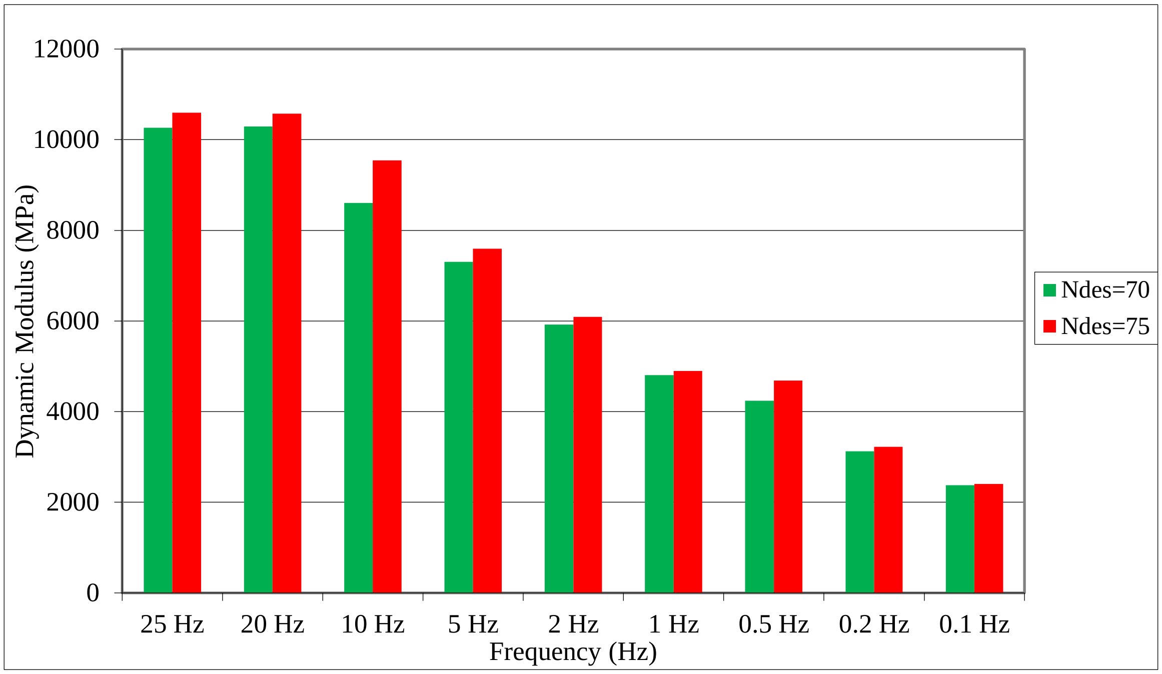


Figure A-15 Dynamic Modulus of Mixture 3 (4160056) at 20°C

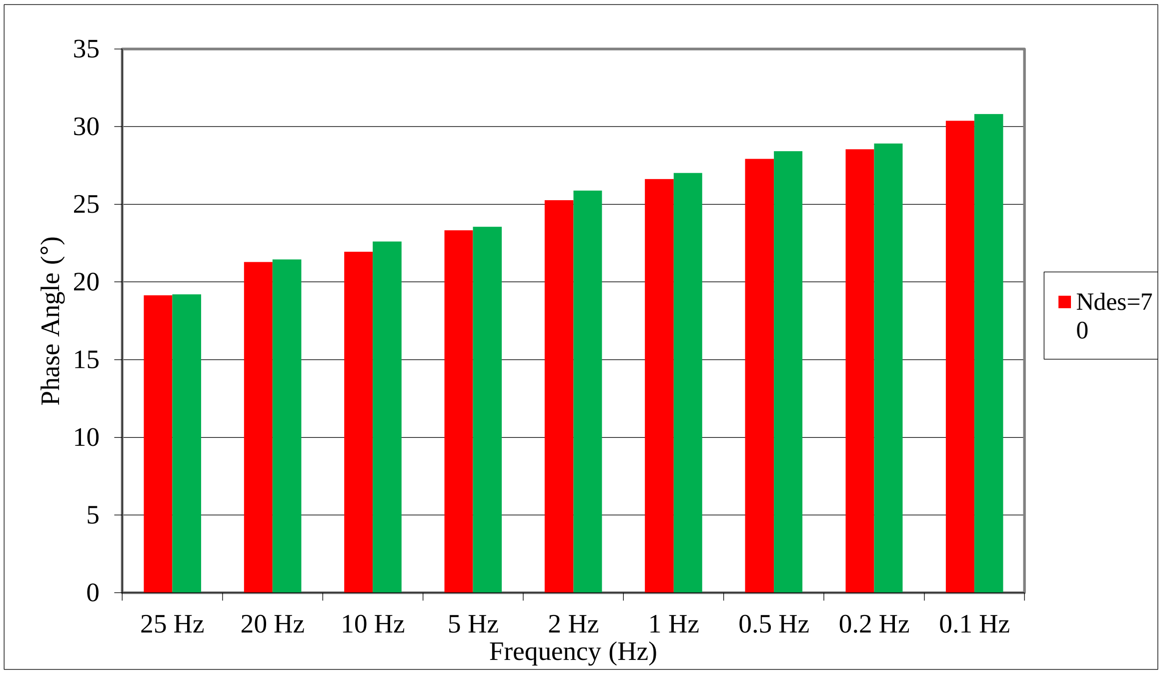


Figure A-16 Phase Angle of Mixture 3 (4160056) at 20°C

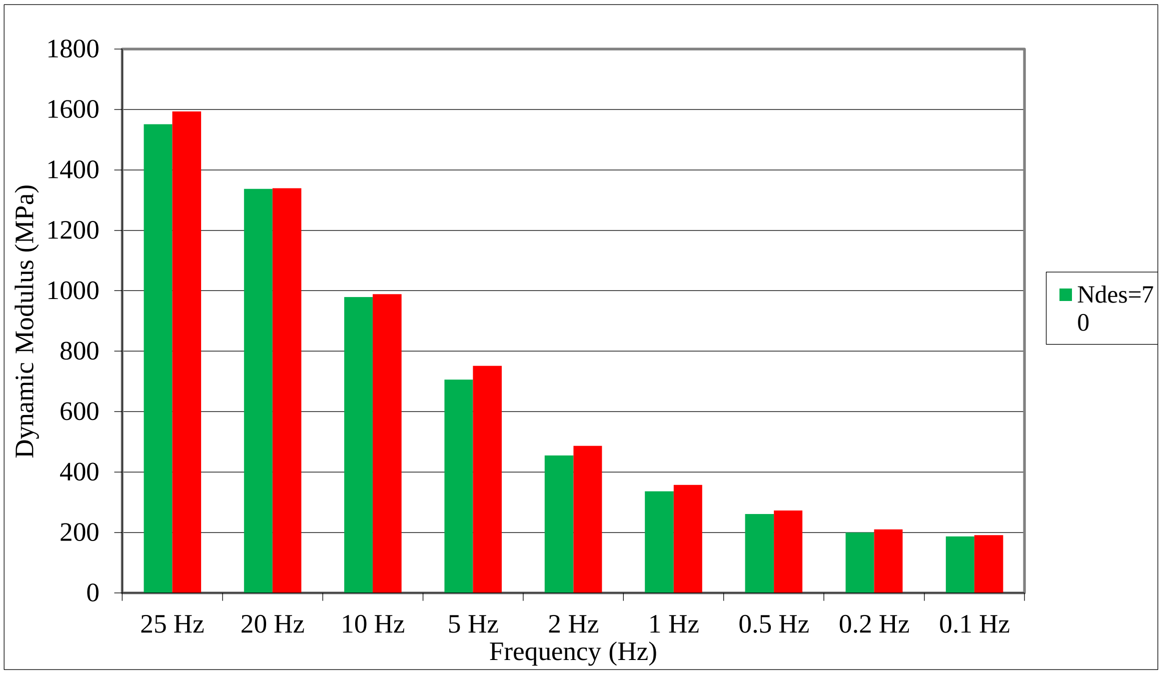


Figure A-17 Dynamic Modulus of Mixture 3 (4160056) at 40°C

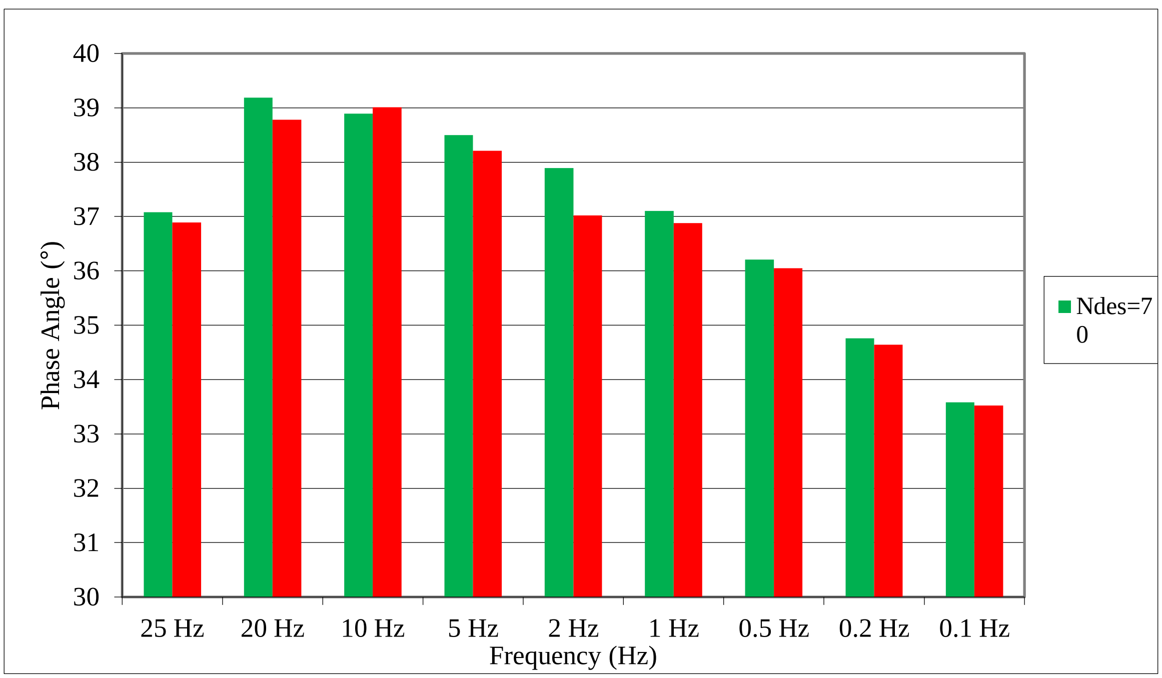


Figure A-18 Phase Angle of Mixture 3 (4160056) at 40°C

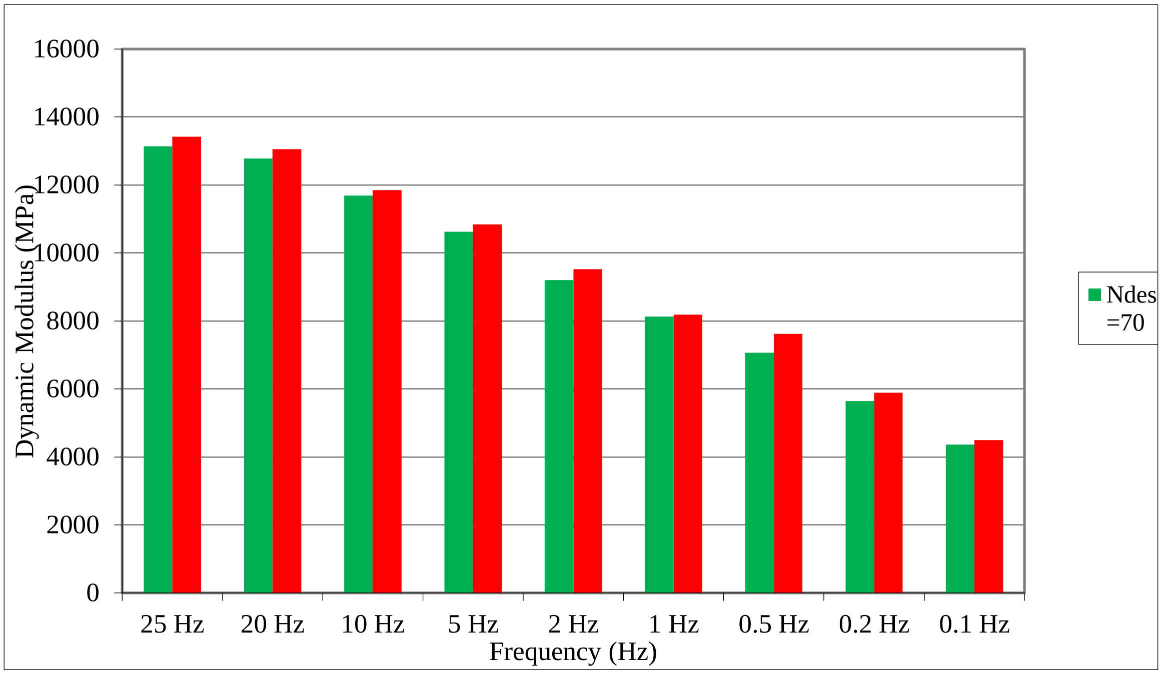


Figure A-19 Dynamic Modulus of Mixture 4 (1160315) at 4°C

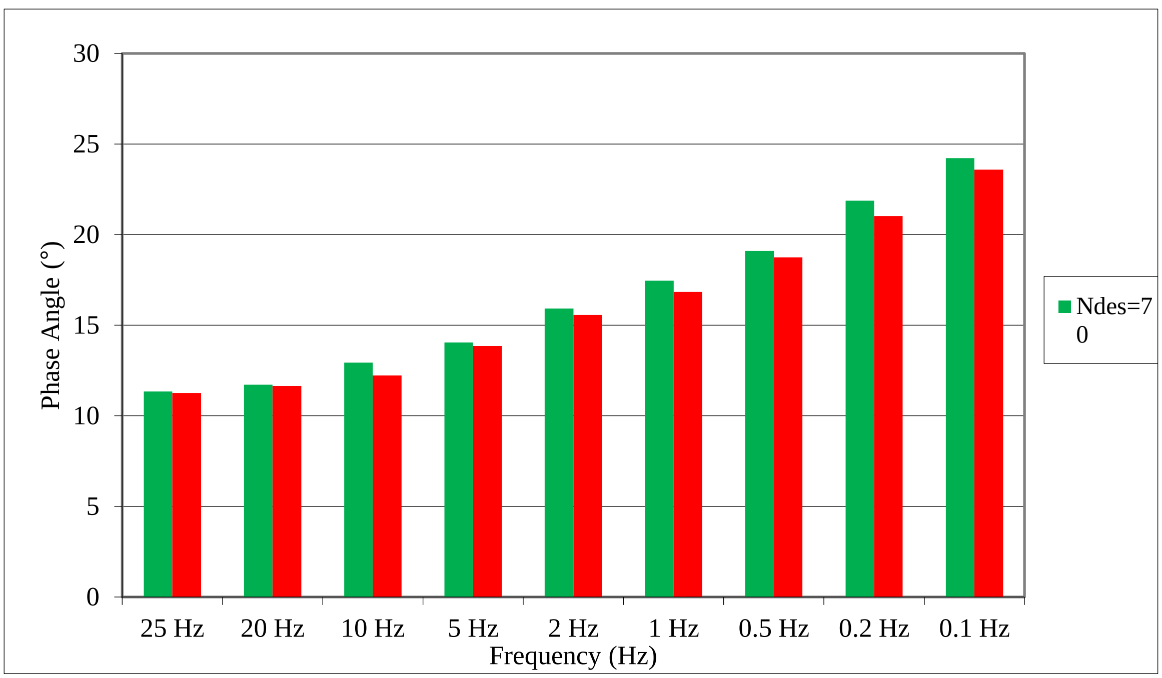


Figure A-20 Phase Angle of Mixture 4 (1160315) at 4°C

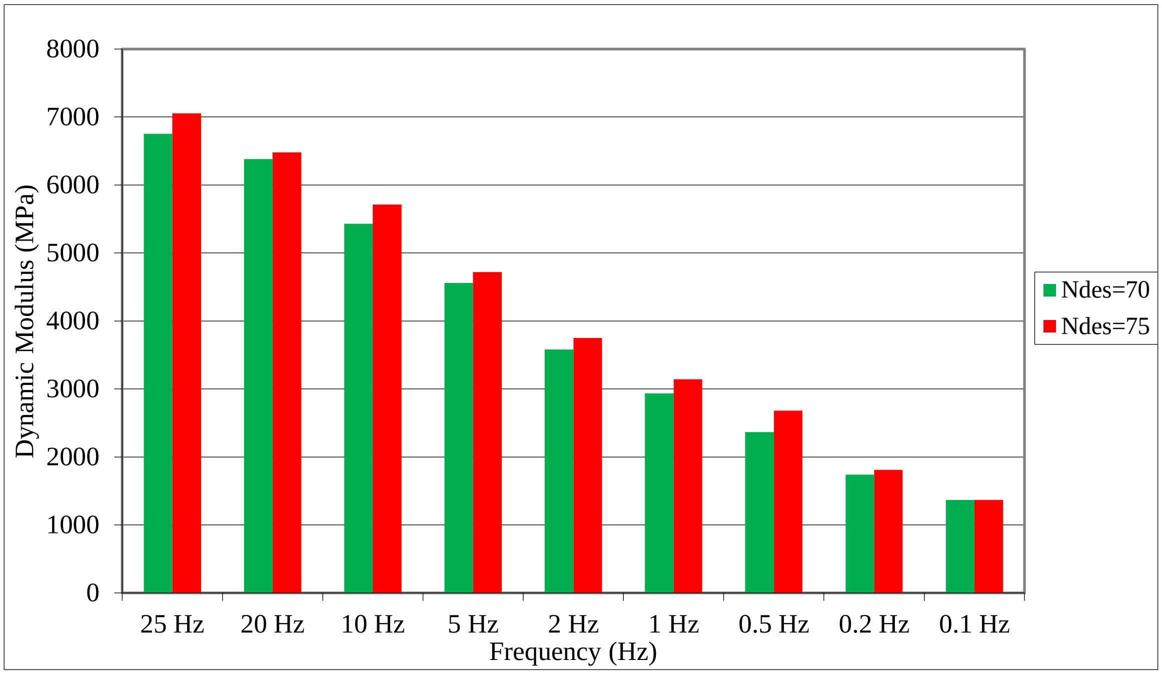


Figure A-21 Dynamic Modulus of Mixture 4 (1160315) at 20°C

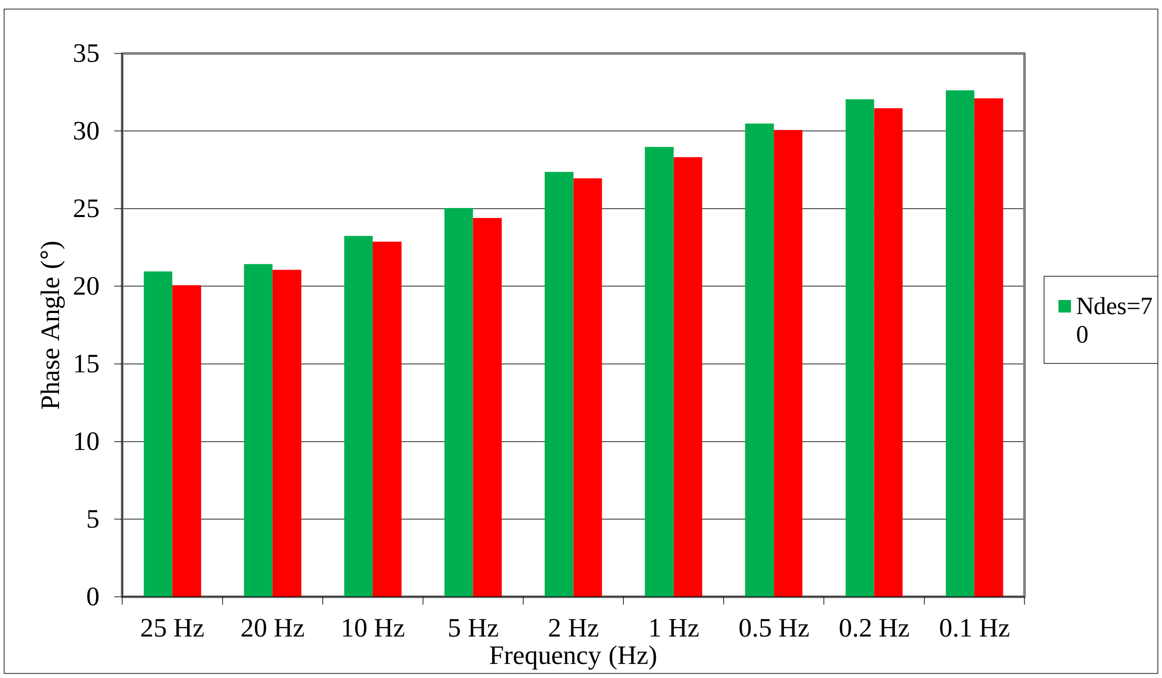


Figure A-22 Phase Angle of Mixture 4 (1160315) at 20°C

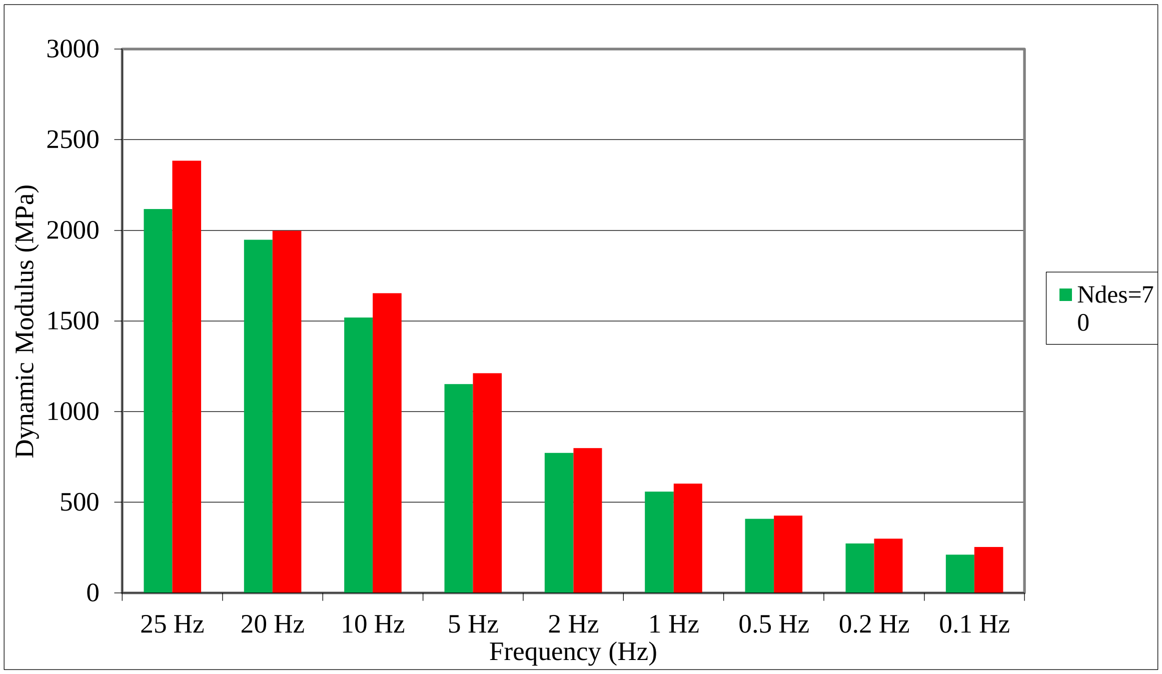


Figure A-23 Dynamic Modulus of Mixture 4 (1160315) at 40°C

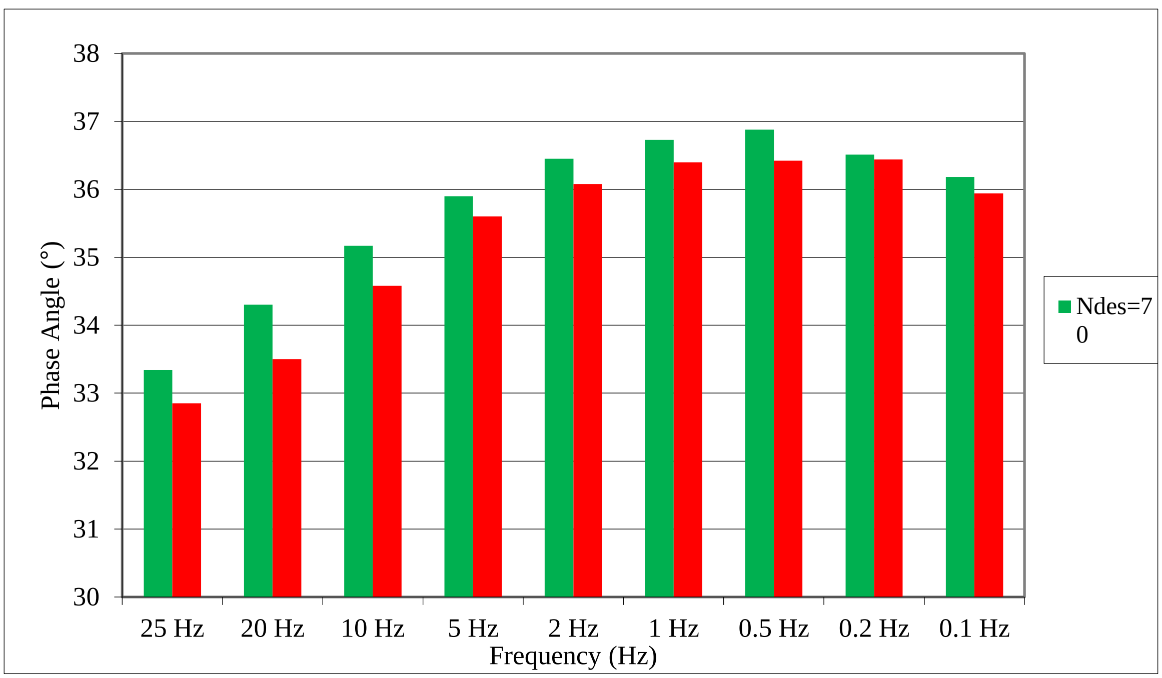


Figure A-24 Phase Angle of Mixture 4 (1160315) at 40°C

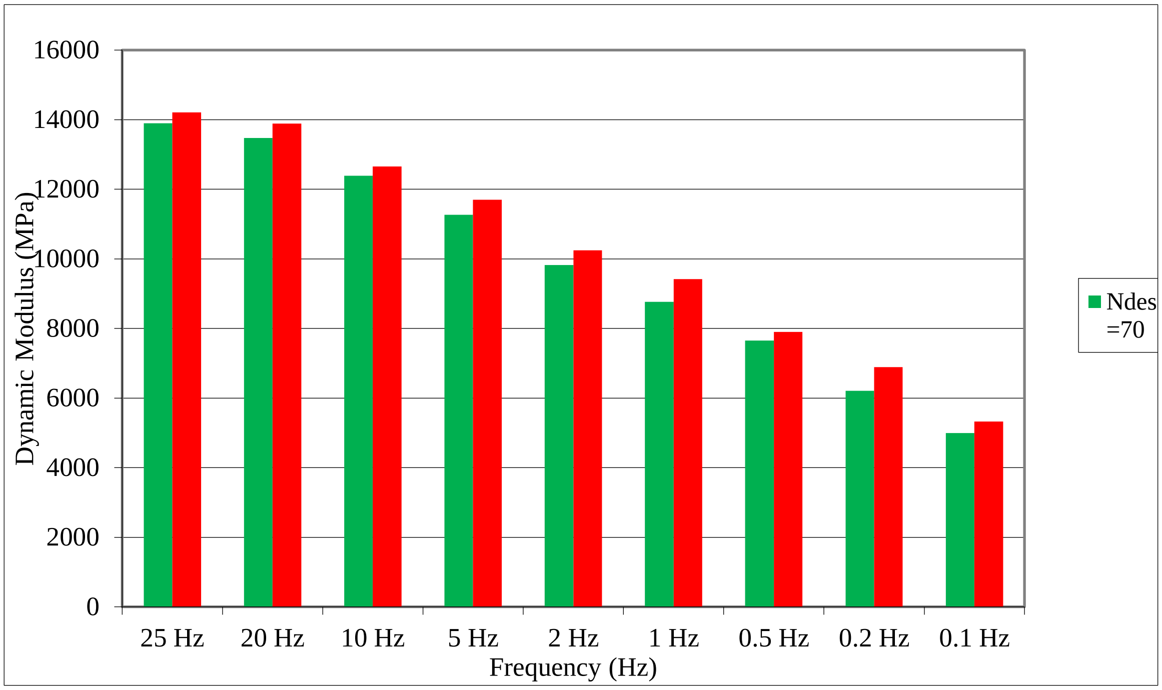


Figure A-25 Dynamic Modulus of Mixture 5 (1160371) at 4°C

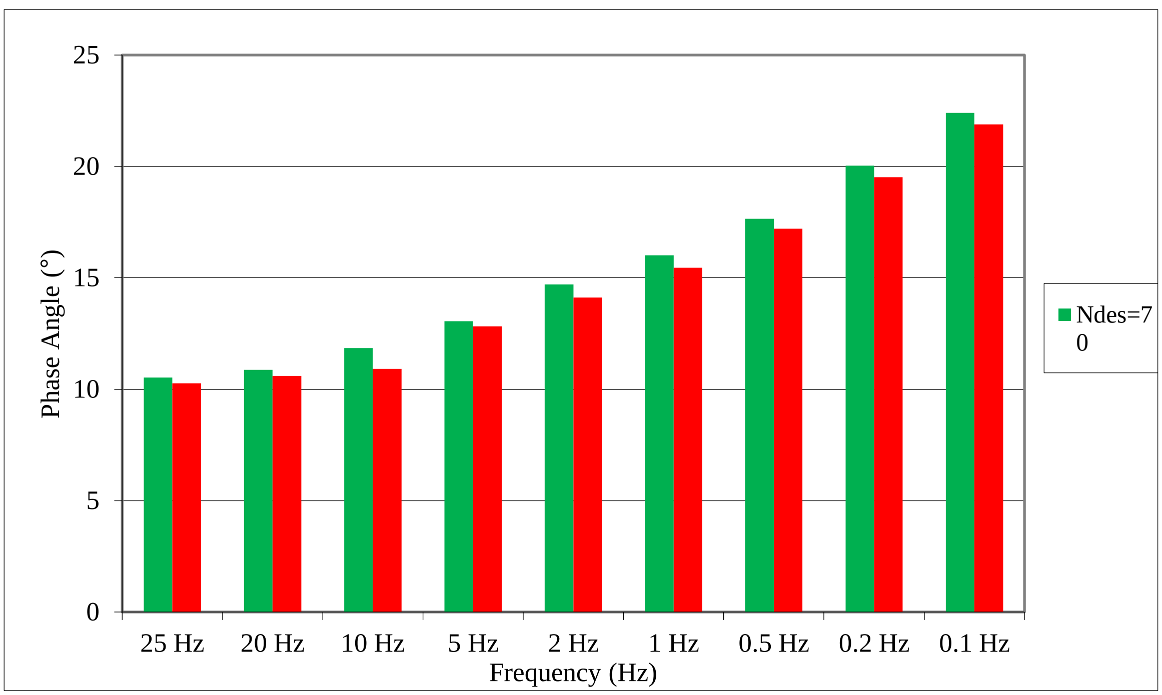


Figure A-26 Phase Angle of Mixture 5 (1160371) at 4°C

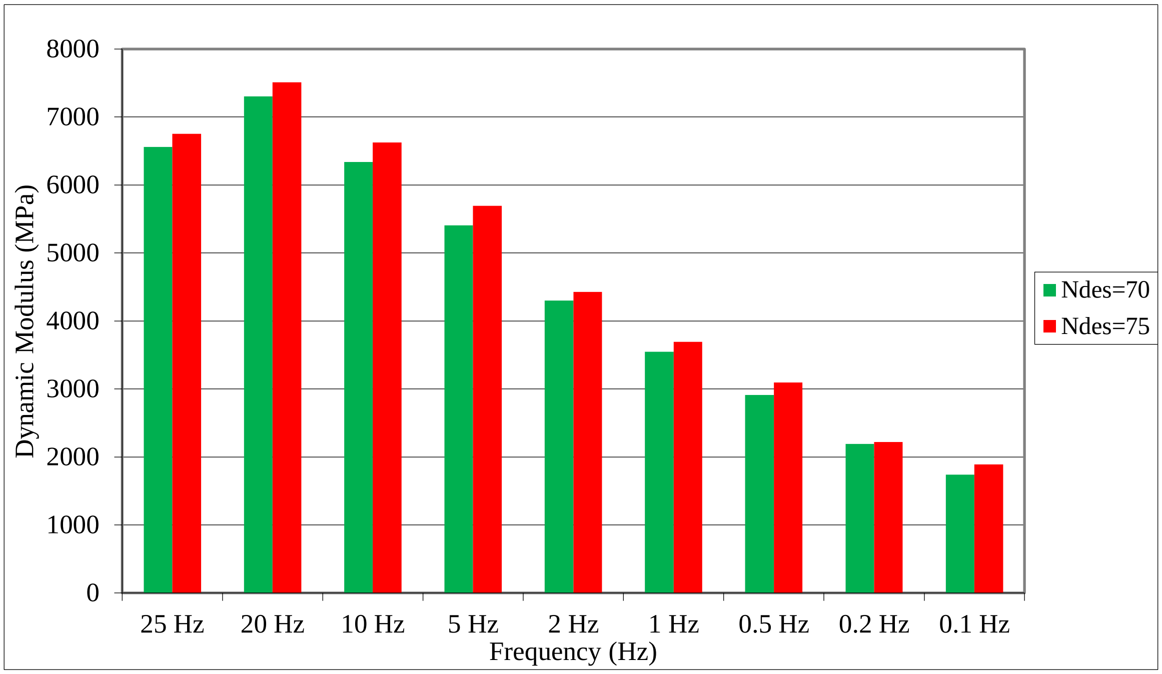


Figure A-27 Dynamic Modulus of Mixture 5 (1160371) at 20°C

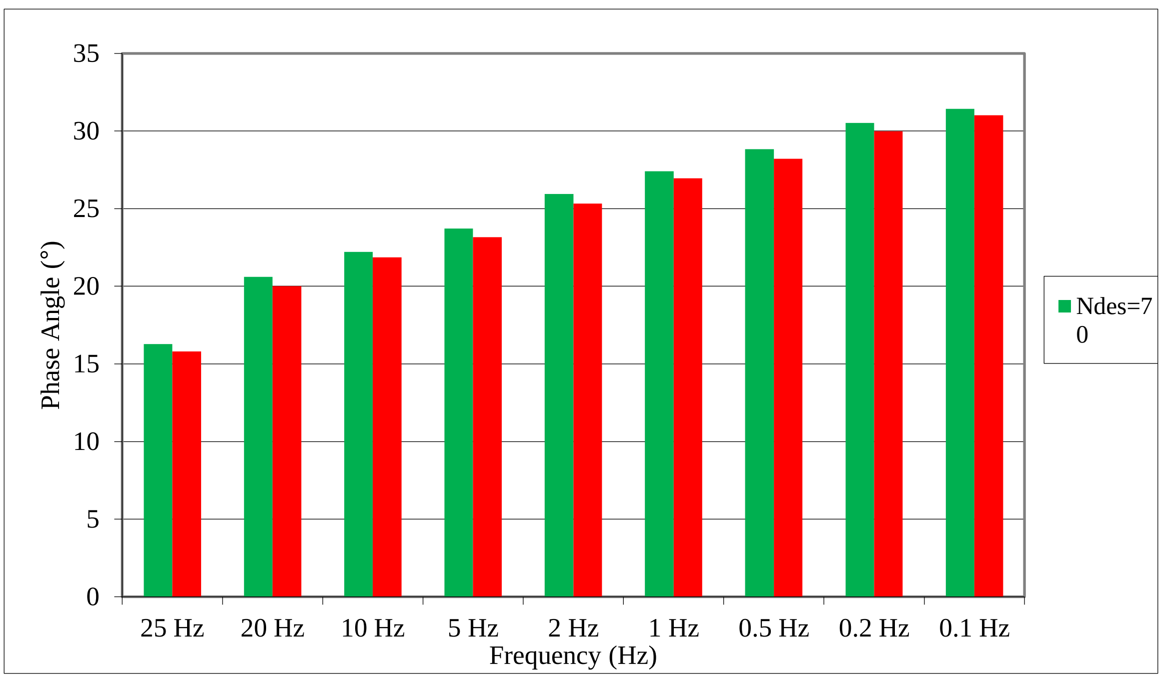


Figure A-28 Phase Angle of Mixture 5 (1160371) at 20°C

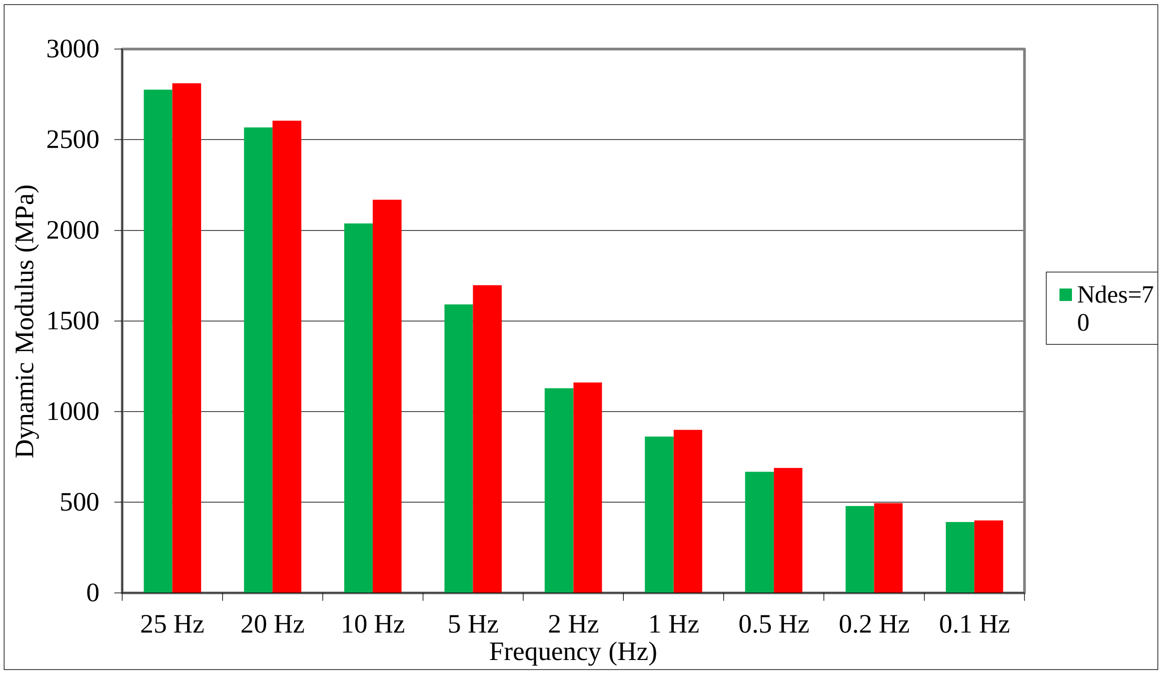


Figure A-29 Dynamic Modulus of Mixture 5 (1160371) at 40°C

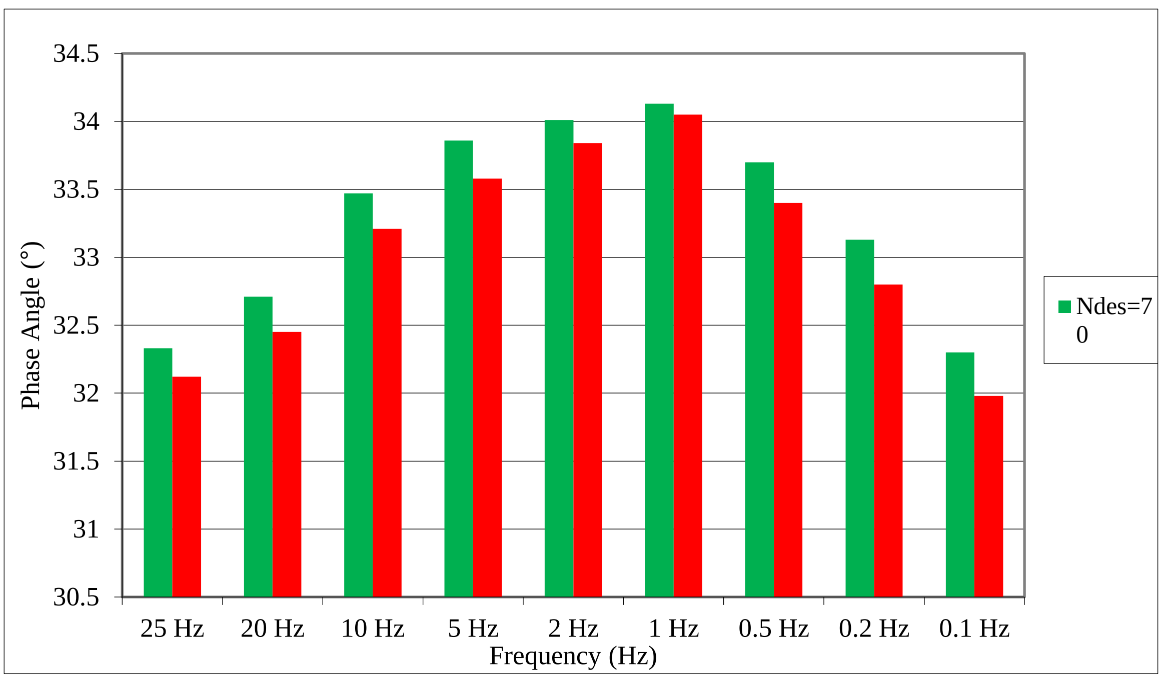


Figure A-30 Phase Angle of Mixture 5 (1160371) at 40°C

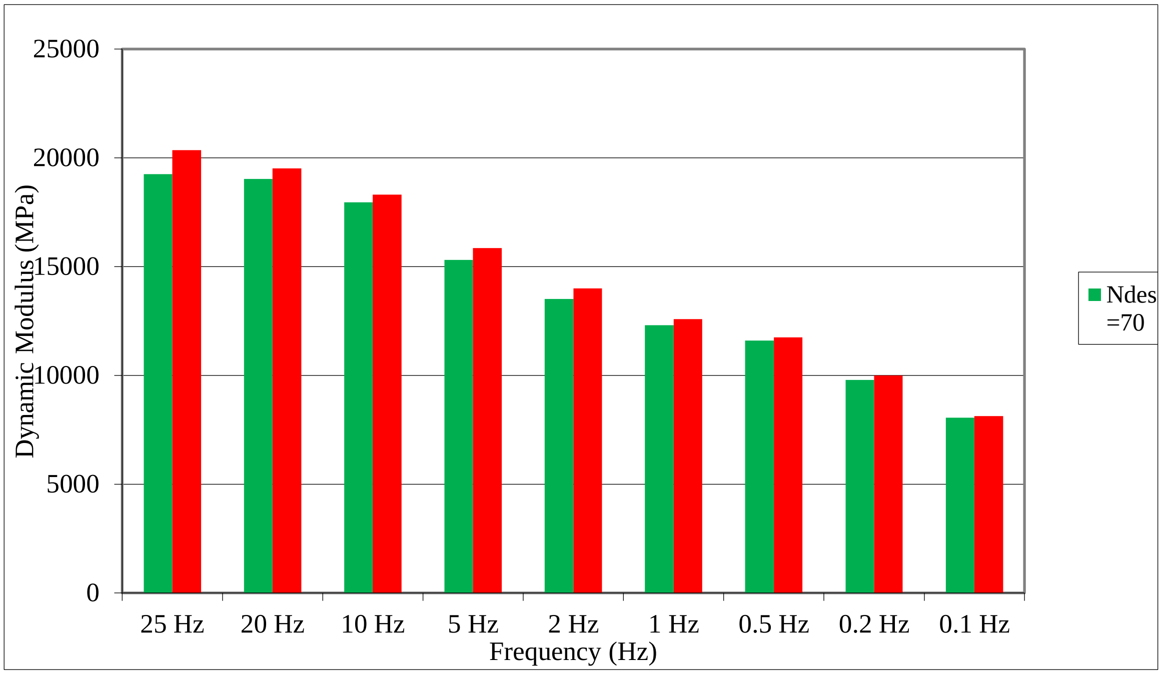


Figure A-31 Dynamic Modulus of Mixture 6 (1160463) at 4°C

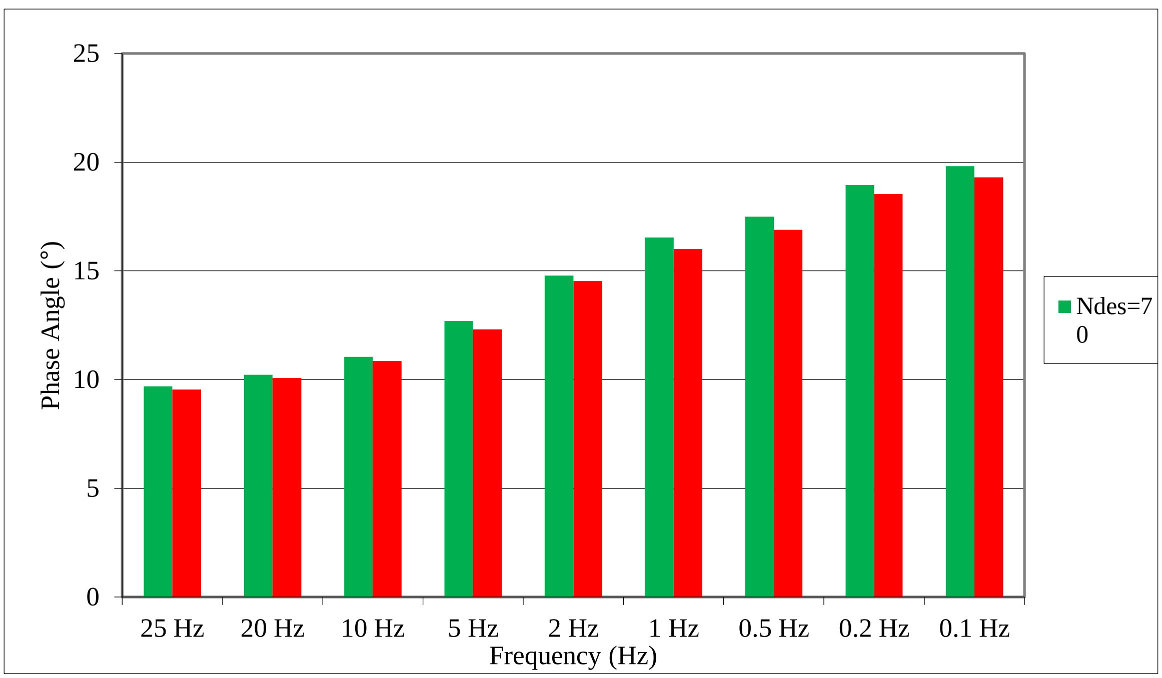


Figure A-32 Phase Angle of Mixture 6 (1160463) at 4°C

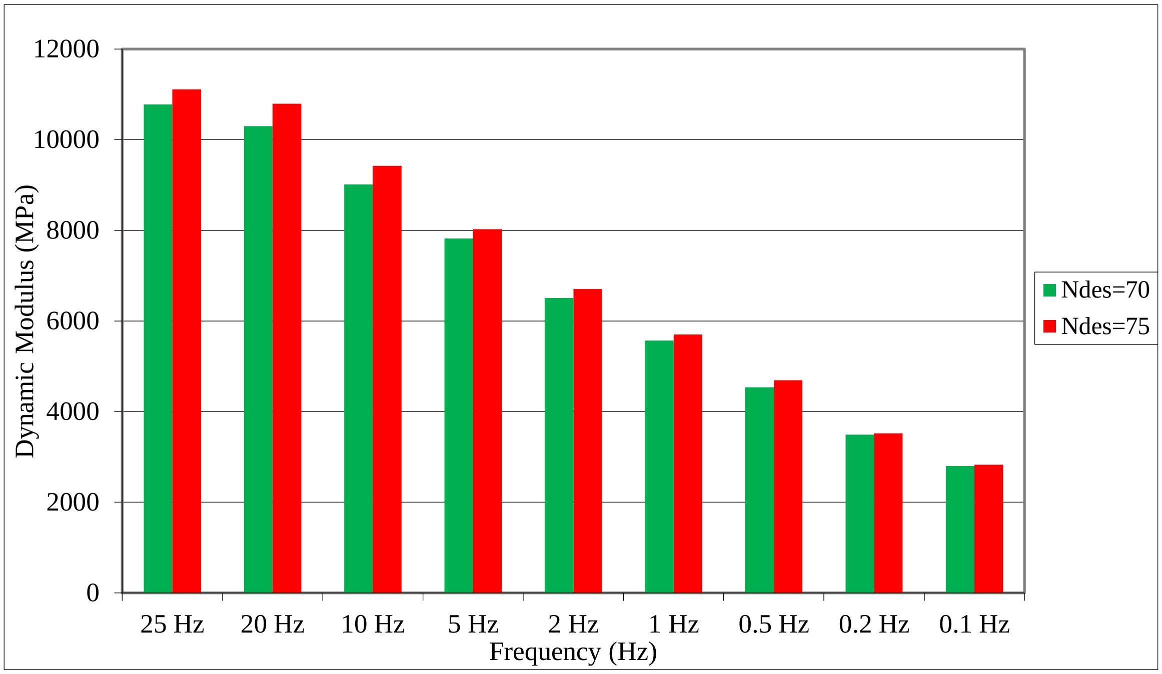


Figure A-33 Dynamic Modulus of Mixture 6 (1160463) at 20°C

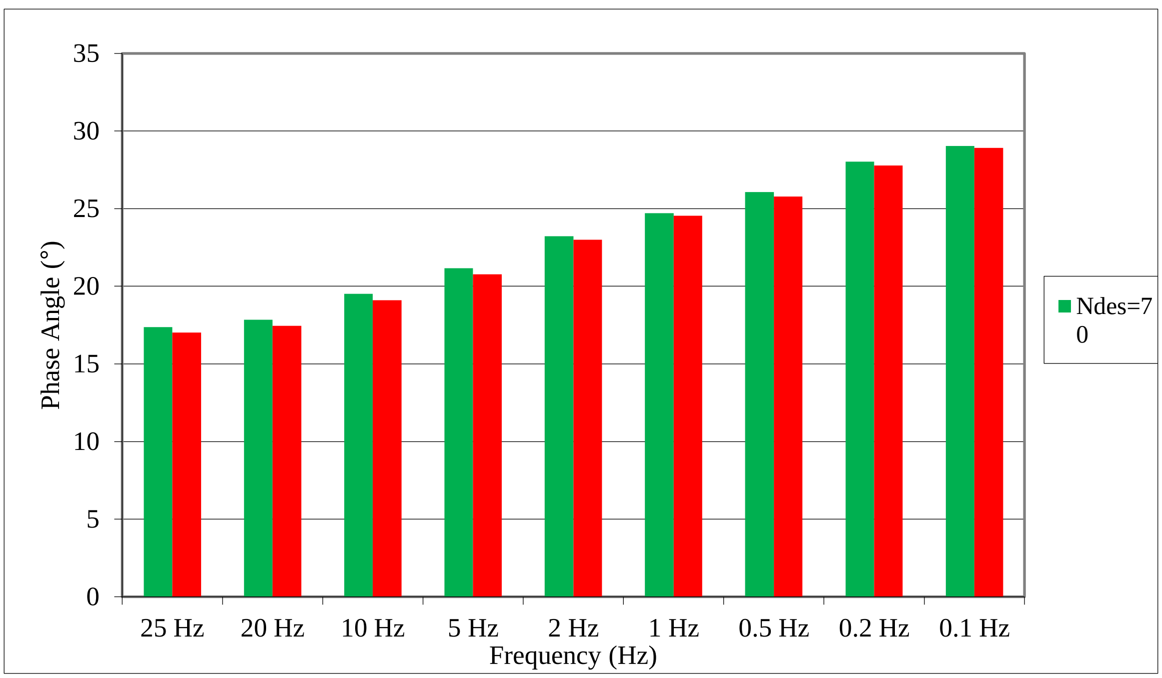


Figure A-34 Phase Angle of Mixture 6 (1160463) at 20°C

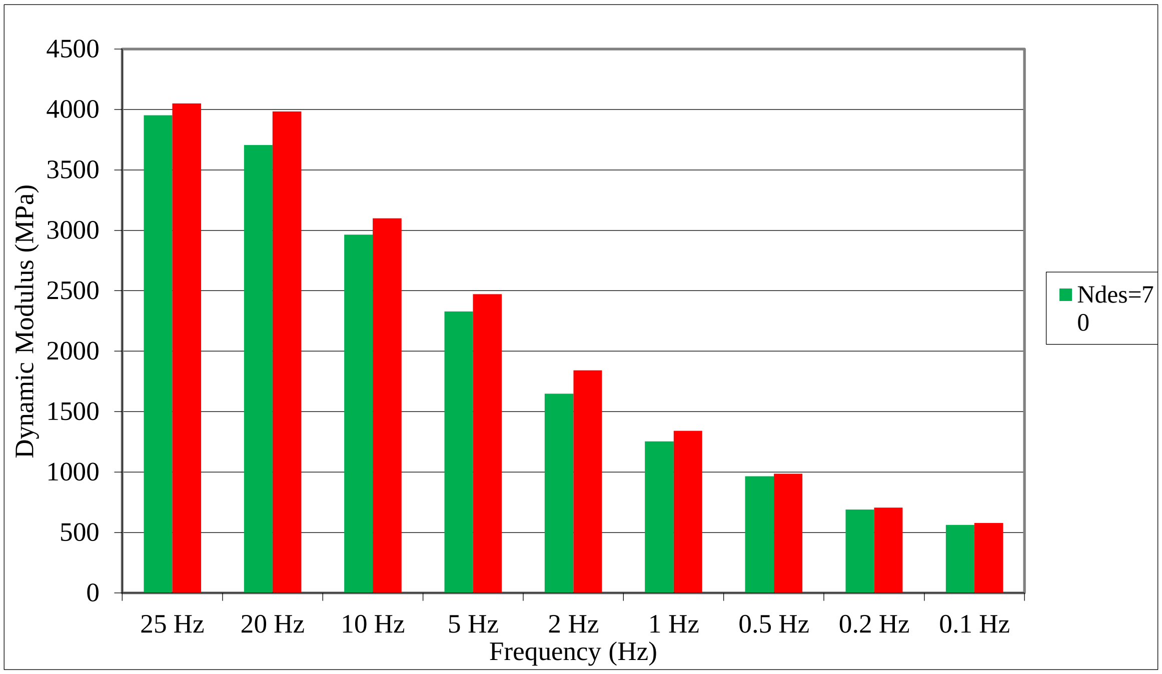


Figure A-35 Dynamic Modulus of Mixture 6 (1160463) at 40°C

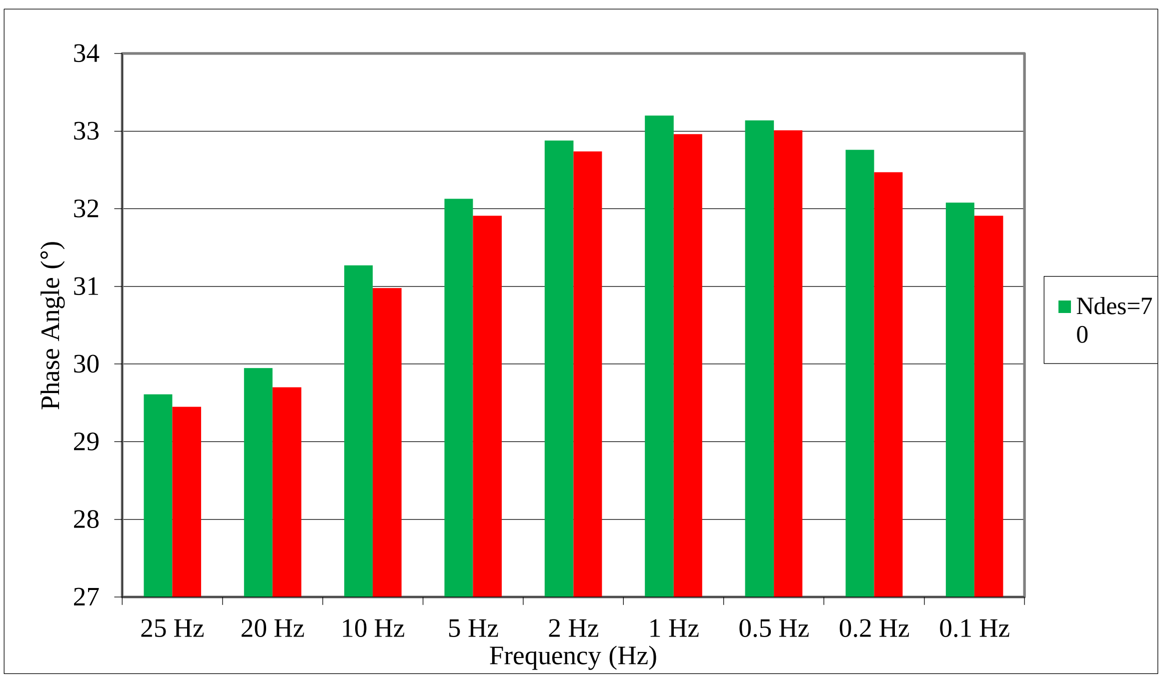


Figure A-36 Phase Angle of Mixture 6 (1160463) at 40°C

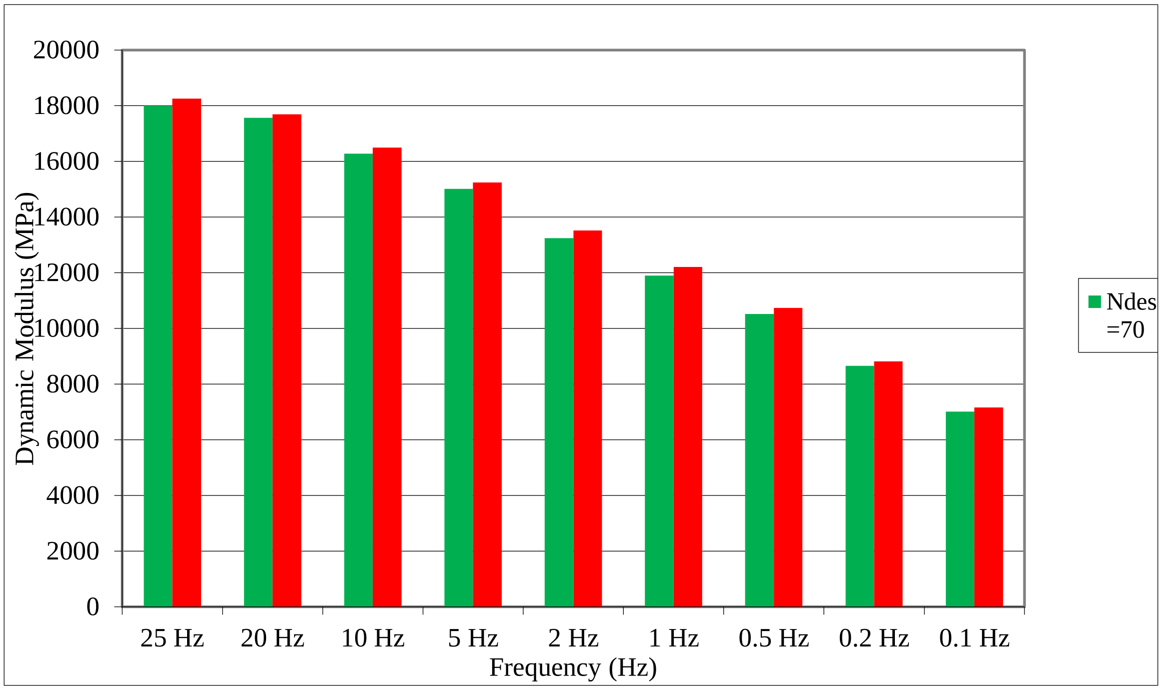


Figure A-37 Dynamic Modulus of Mixture 7 (4160010) at 4°C

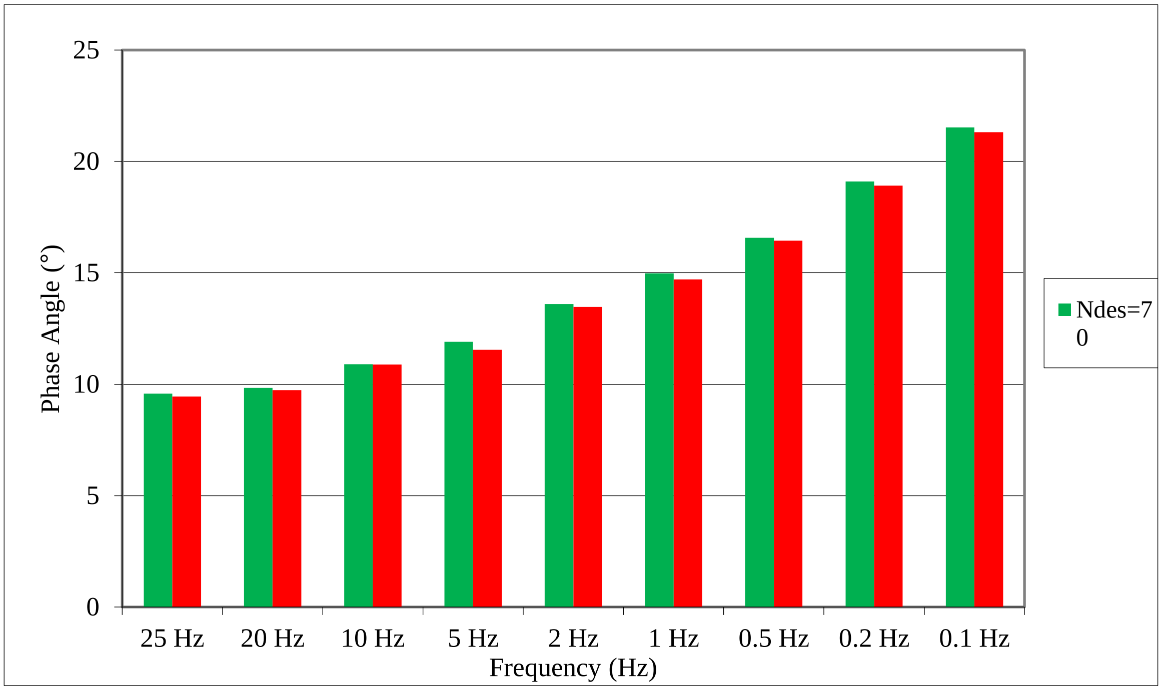


Figure A-38 Phase Angle of Mixture 7 (4160010) at 4°C

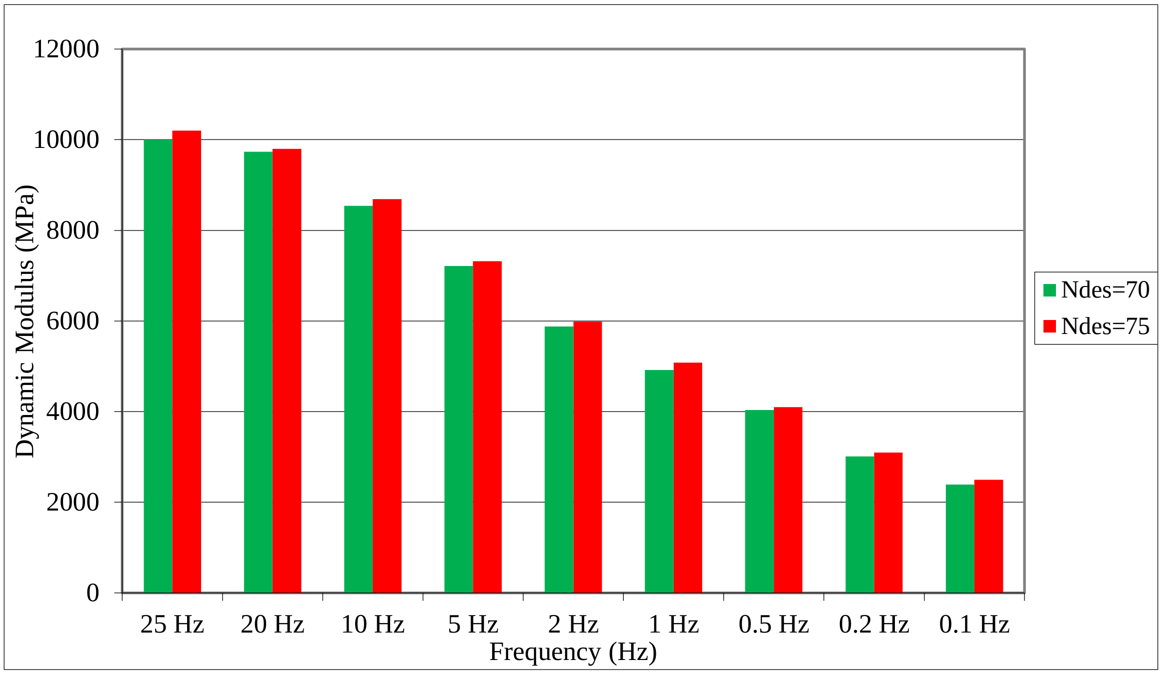


Figure A-39 Dynamic Modulus of Mixture 7 (4160010) at 20°C

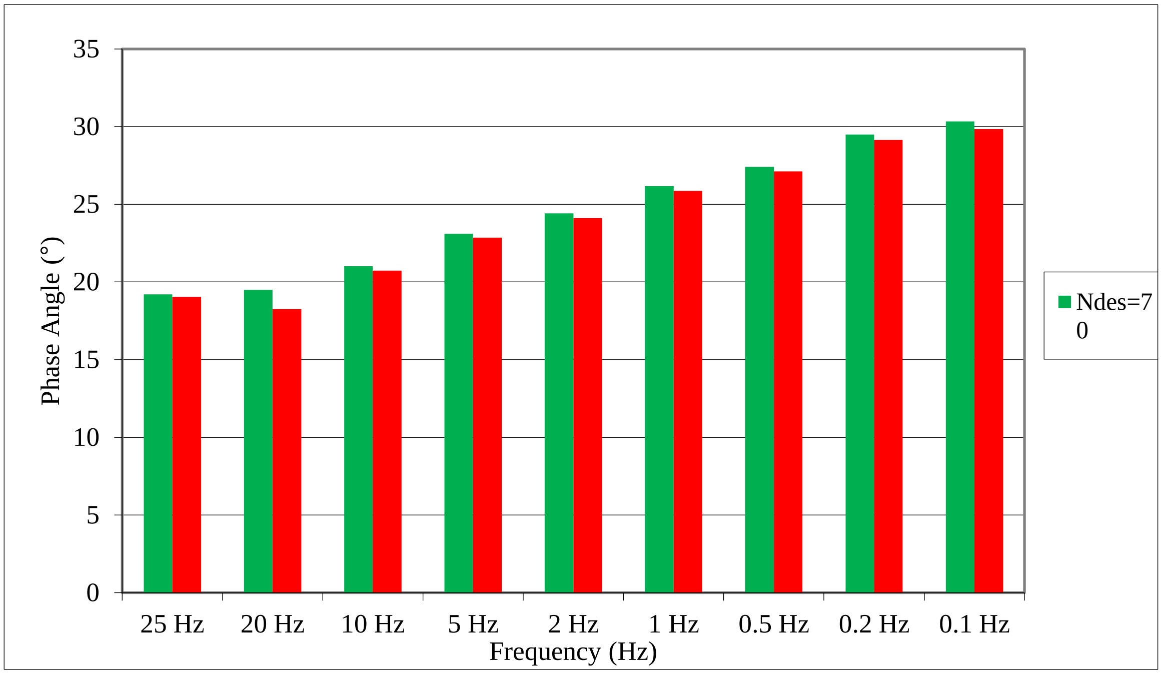


Figure A-40 Phase Angle of Mixture 7 (4160010) at 20°C

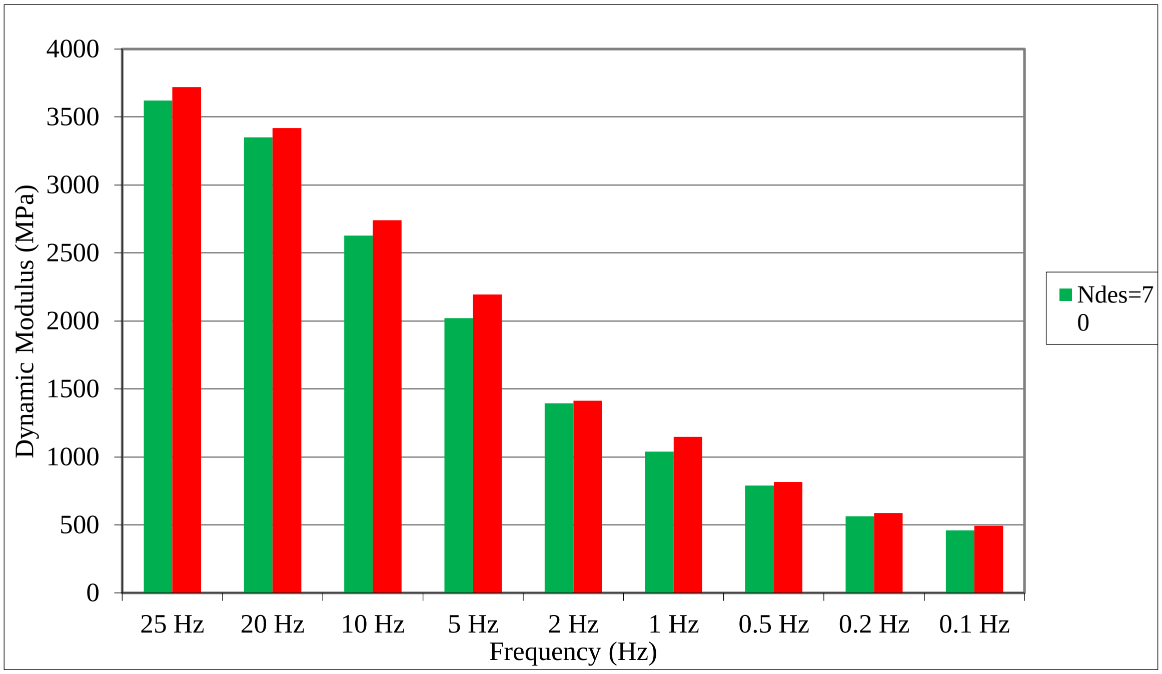


Figure A-41 Dynamic Modulus of Mixture 7 (4160010) at 40°C

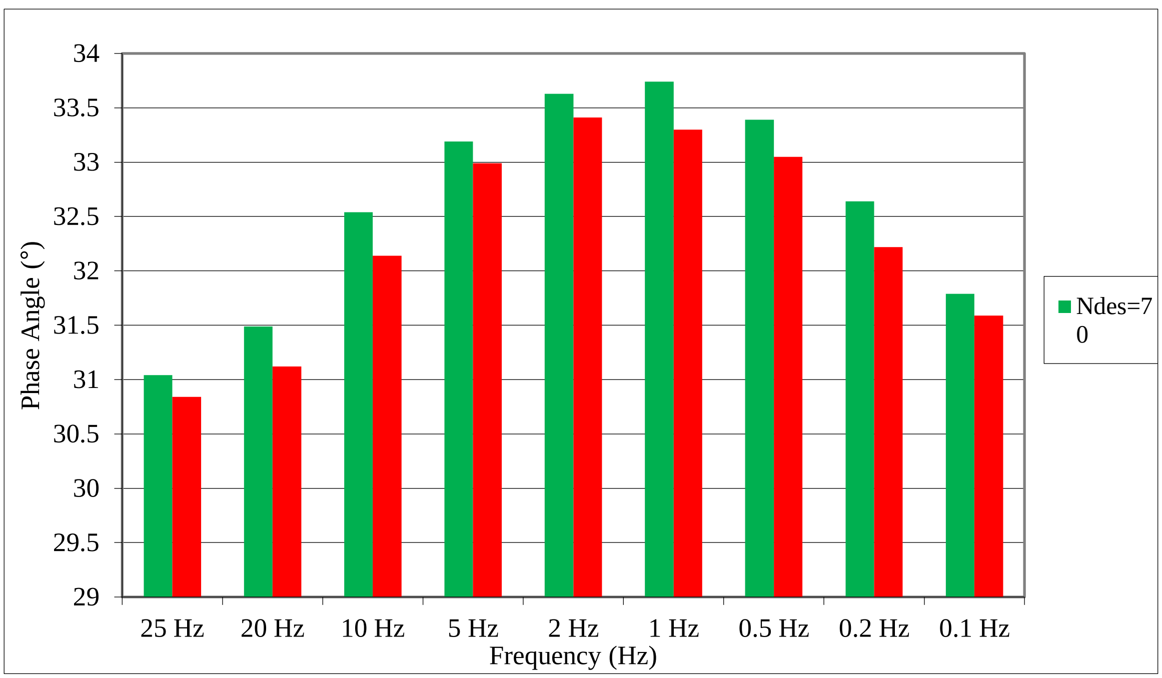


Figure A-42 Phase Angle of Mixture 7 (4160010) at 40°C

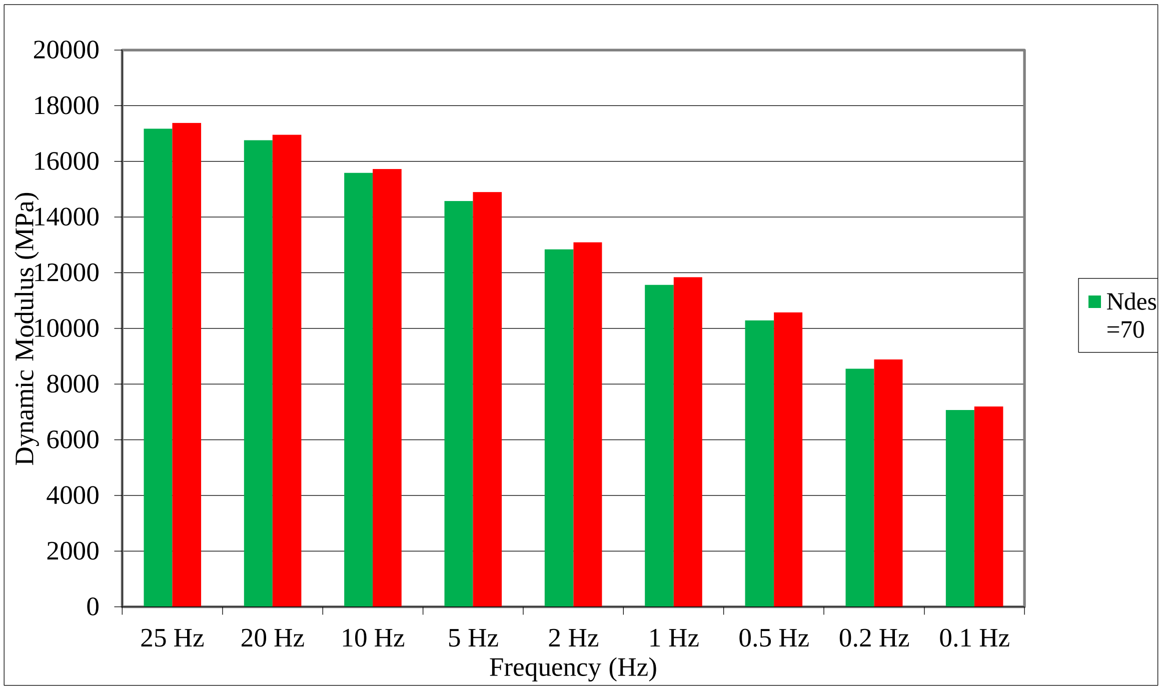


Figure A-43 Dynamic Modulus of Mixture 8 (4160049) at 4°C

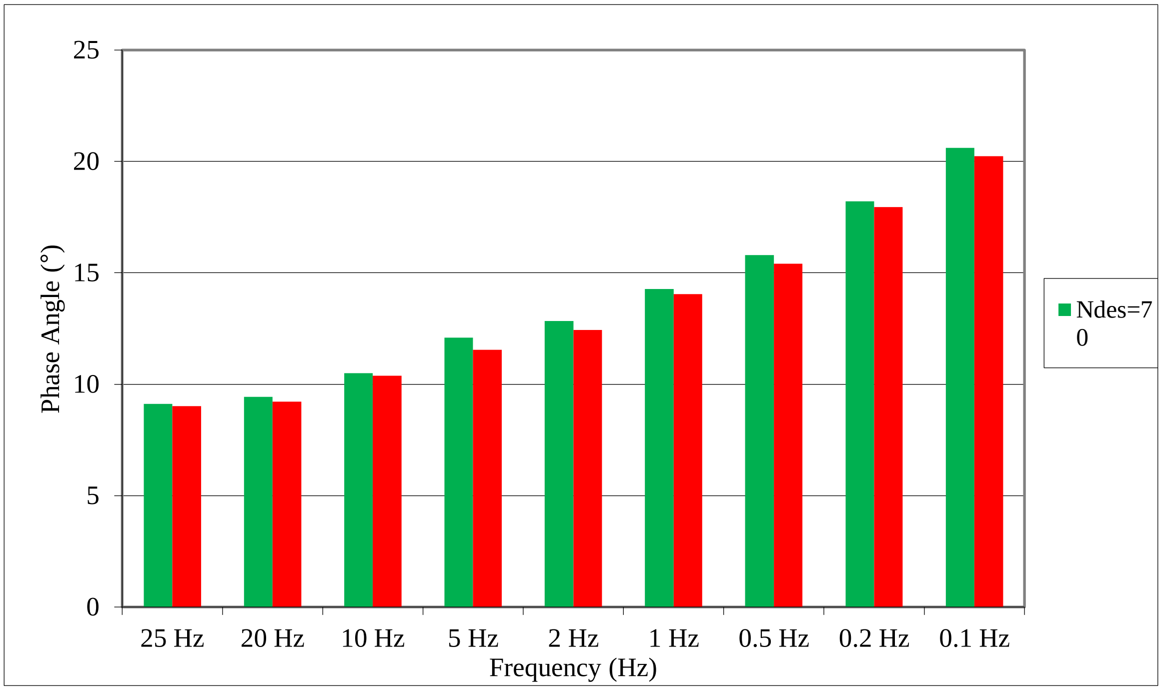


Figure A-44 Phase Angle of Mixture 8 (4160049) at 4°C

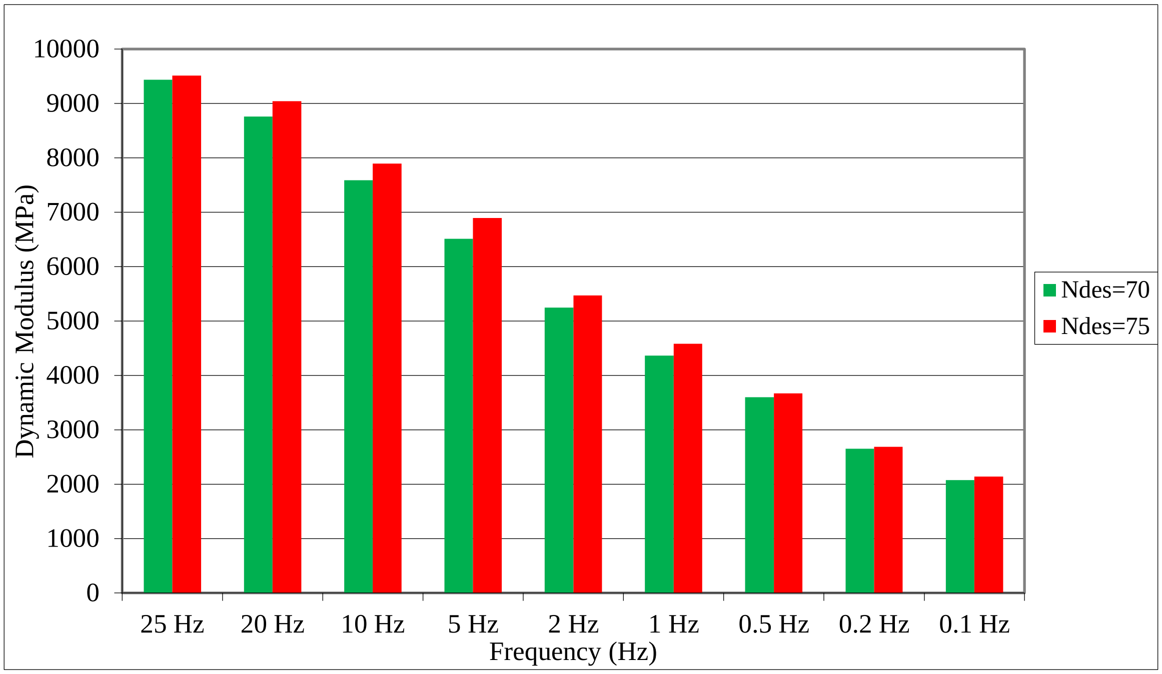


Figure A-45 Dynamic Modulus of Mixture 8 (4160049) at 20°C

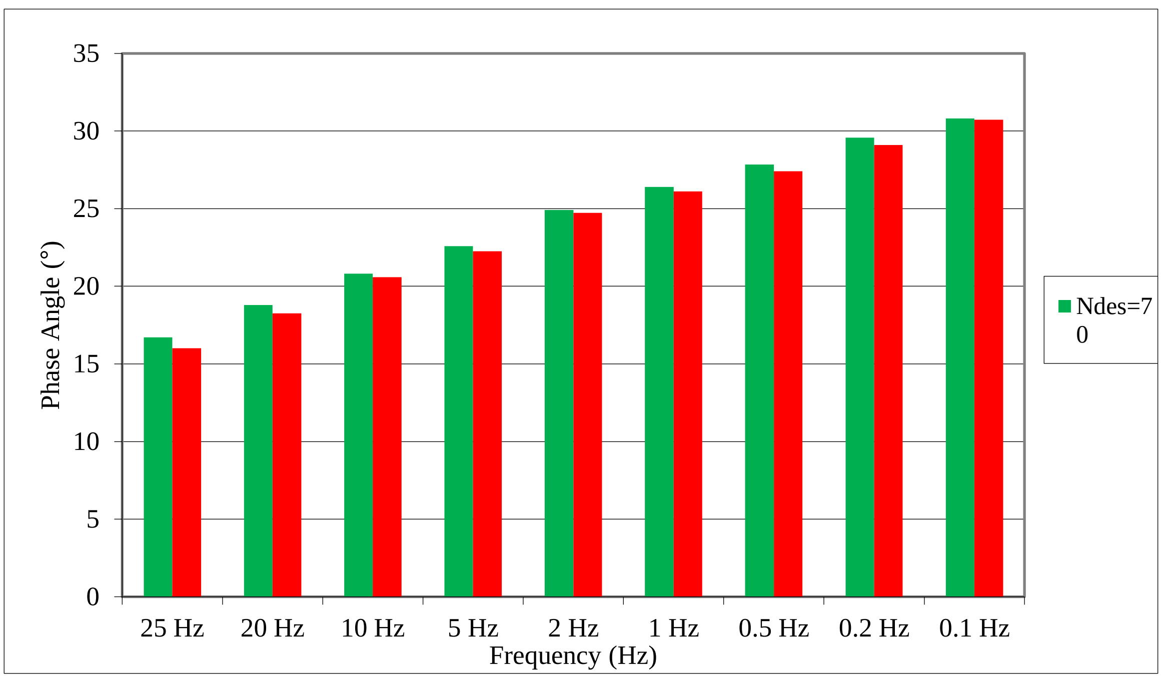


Figure A-46 Phase Angle of Mixture 8 (4160049) at 20°C

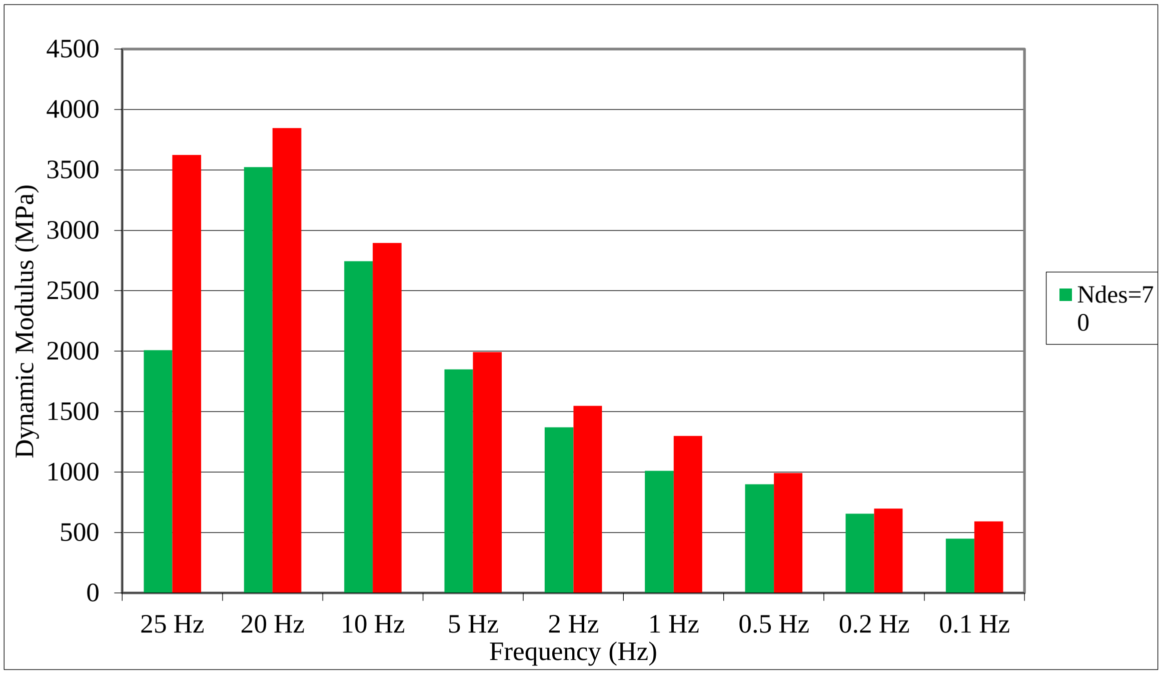


Figure A-47 Dynamic Modulus of Mixture 8 (4160049) at 40°C

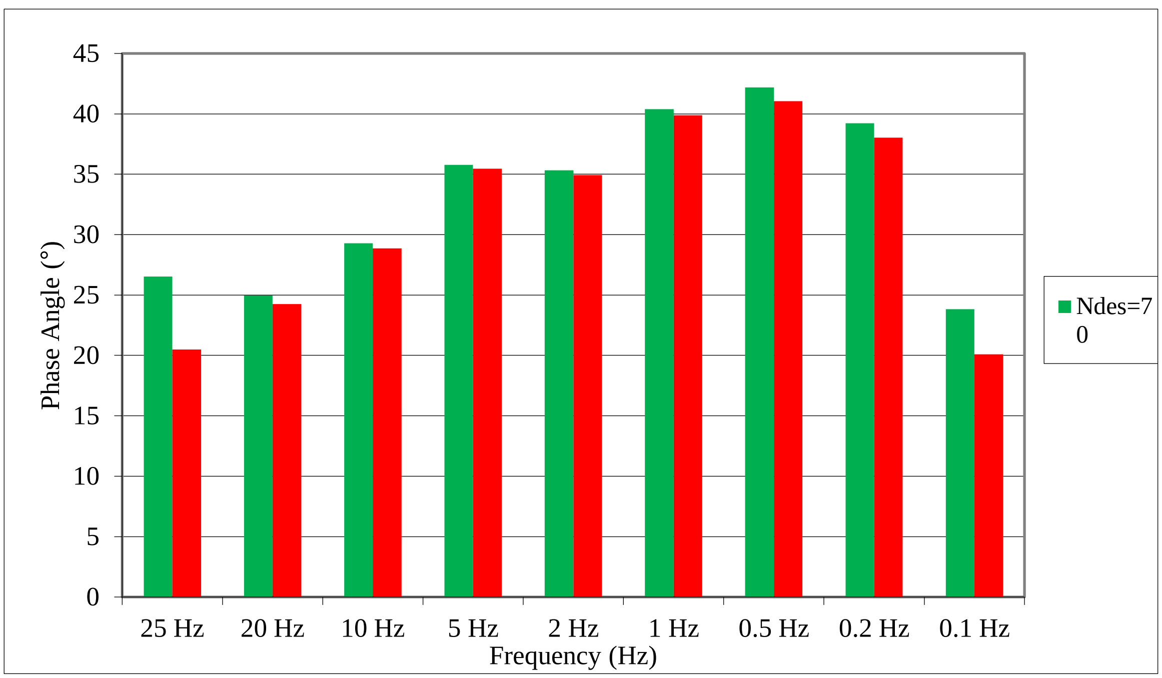


Figure A-48 Phase Angle of Mixture 8 (4160049) at 40°C

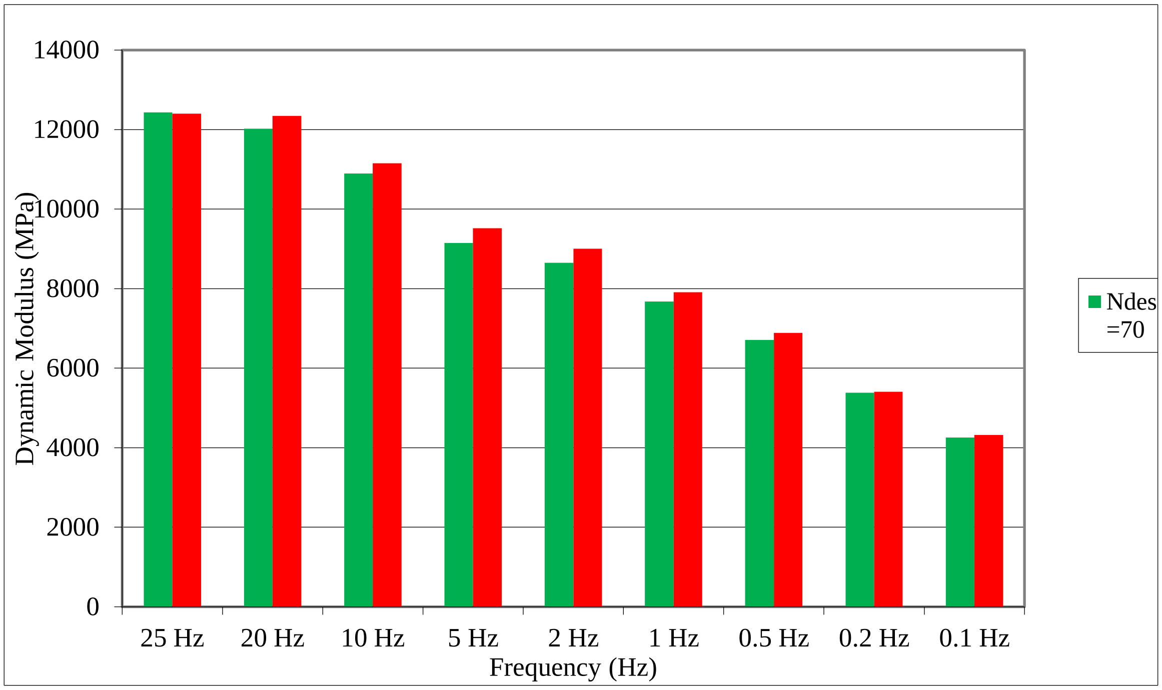


Figure A-49 Dynamic Modulus of Mixture 9 (4160125) at 4°C

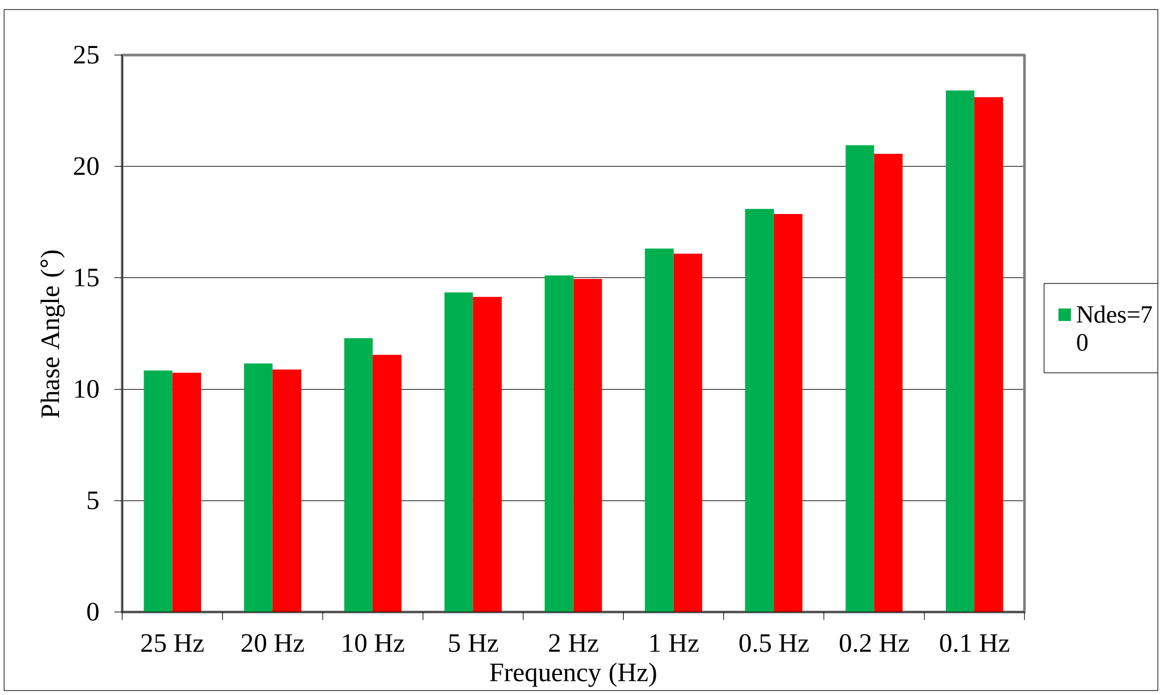


Figure A-50 Phase Angle of Mixture 9 (4160125) at 4°C

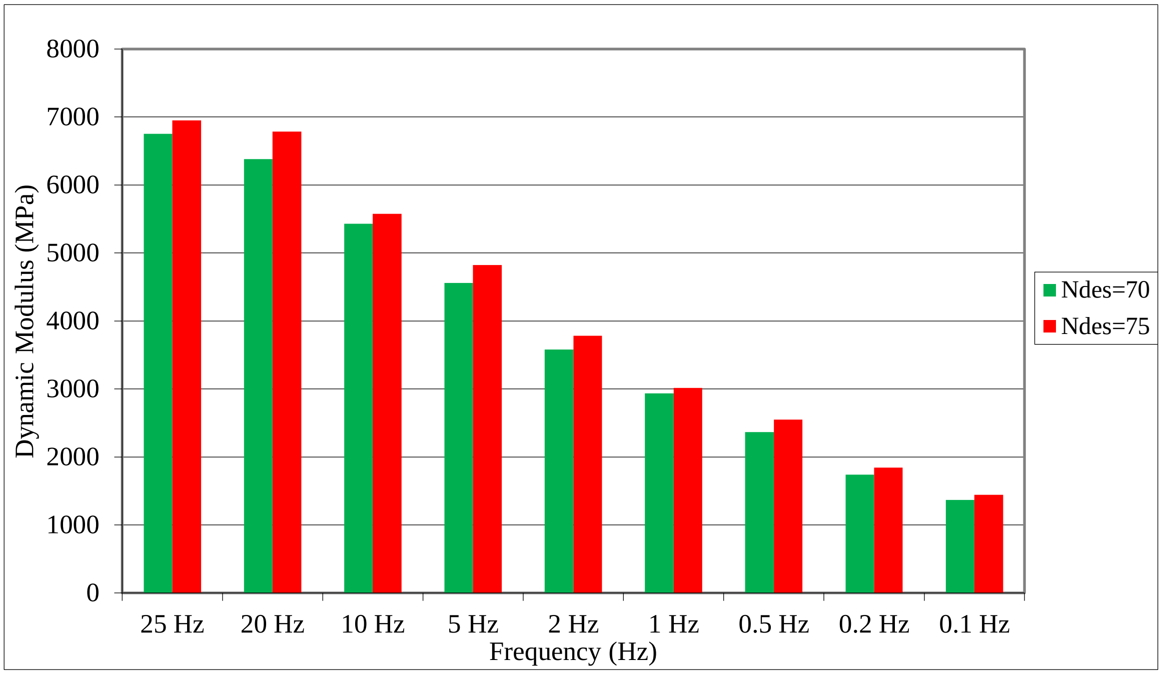


Figure A-51 Dynamic Modulus of Mixture 9 (4160125) at 20°C

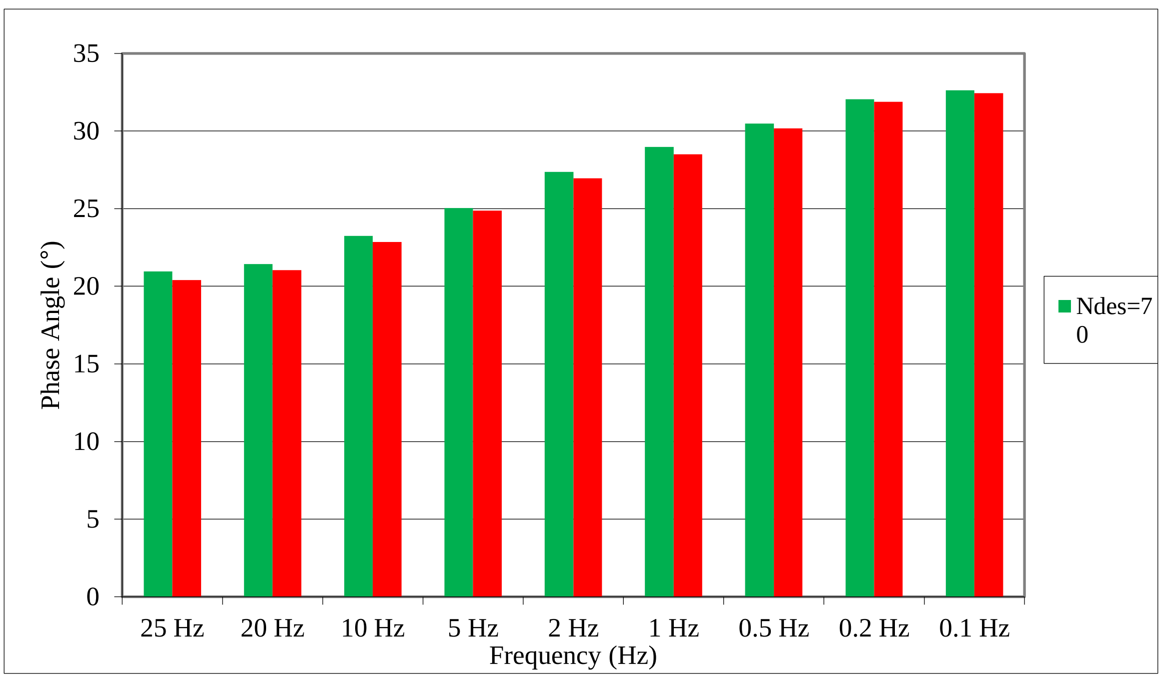


Figure A-52 Phase Angle of Mixture 9 (4160125) at 20°C

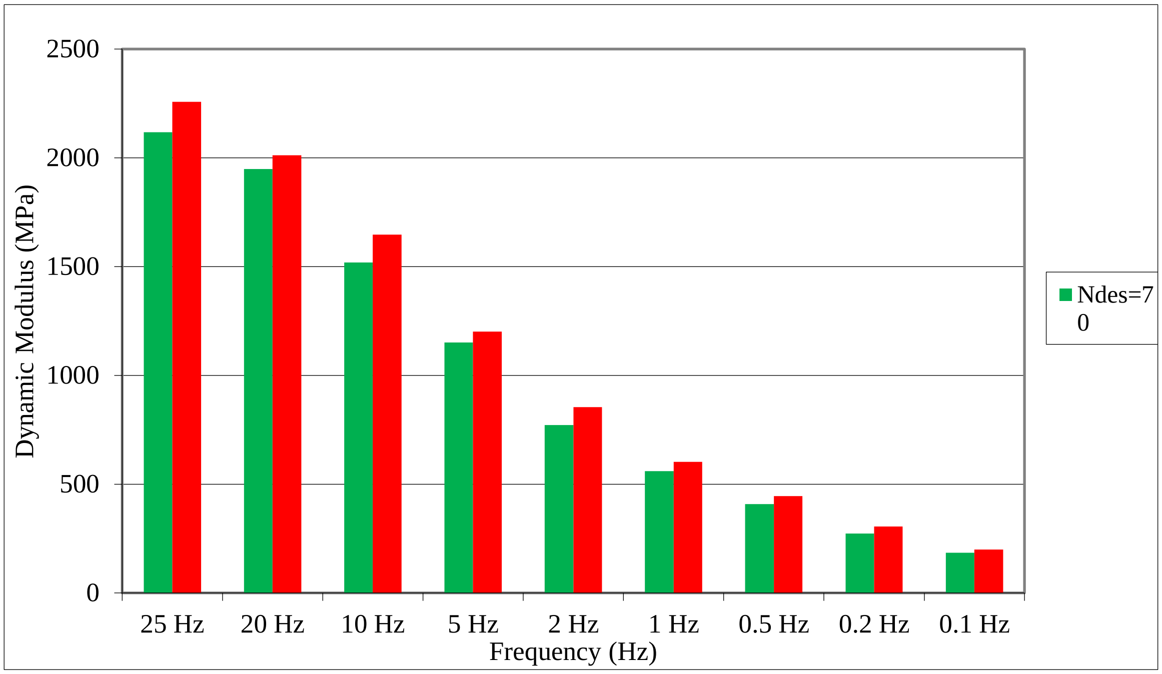


Figure A-53 Dynamic Modulus of Mixture 9 (4160125) at 40°C

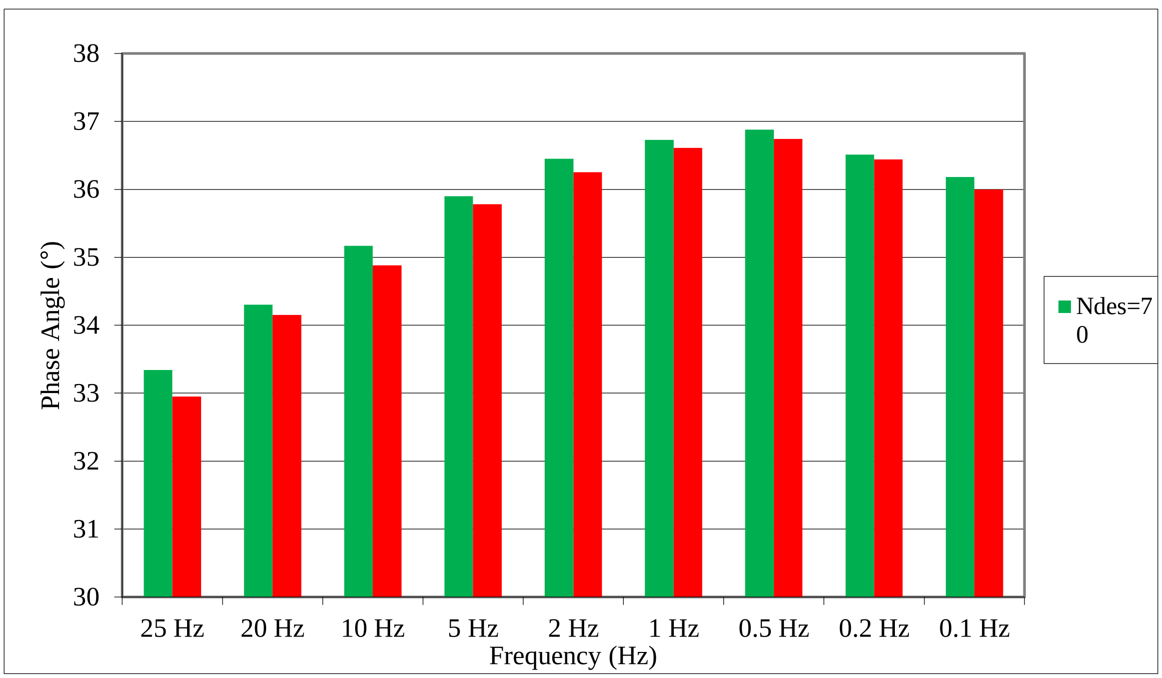


Figure A-54 Phase Angle of Mixture 9 (4160125) at 40°C

APPENDIX B: Job Mix Formulas

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 V4.10

Date 03/09/2016 **Roadway Surface** No

Region 1

Hot-mix Producer Duracap Asphalt Paving - Heiskell Asphalt Mix

Type BPMB-HM **Mix** 307-BM2 PG 76-22 **Item**

Serial No.:	Design No.: 1160307

Material	Size or Grade	Producer and Location	Percent Used
Soft Limestone (aka Non-Sur	#57	Aggregate USA - Heiskell I-75	33.425
Soft Limestone (aka Non-Sur	#7	Aggregate USA - Heiskell I-75	19.100
Soft Limestone (aka Non-Sur	#10	Aggregate USA - Heiskell I-75	23.875
RAP	RAP Processed -1/2	RAP - Duracap Asphalt Paving - Knoxville Asphalt Mix	20.212
Asphalt Cement	PG 76-22	MARATHON PETROLEUM CO., KNOXVILLE	3.388

Percent AC in RAP1:		Optimum AC Content:	4.50	Total	100.000
Percent AC in RAP2:	5.5		Anti-Strip Supplier:	Tri-State Sand LLC	
Anti-Strip Additive:		Arr Maz	Dosage:	0.5%	
AC Contribution:	Virgin AC 3.39	RAP AC 1.11	Percent Virgin AC:	75.3	
Asphalt Sp. Gravity:	1.028		Dust to Asphalt Ratio:	1.27	

% Fracture Face on CA:	n/a	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:		Eff. Gravity of Agg:	2.795
Theo.Gravity of RAP2:	2.546		
Theo. Gravity of Mix: 2.594	T.S.R.: 96.3	Lbs/Ft^3:	161.9
L.O.I.:		Ignition Oven Corr. Factor:	0.41
		Warm Mix?	No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	305	Mixing Temp Range(°F):	290ºF ≤ T ≤ 330ºF
Lab Compaction Temp (± 5 °F):	290	Delivery Temperature(°F):	290ºF ≤ T ≤ 330ºF

	Percents Used								
Sieve Size	#57	#7	#10				RAP Processed -1/2	% Req.	Design Range
	35.0	20.0	25.0				20.0	100	
2"									
1.5"									
1.25"	100	100	100				100	100	100
1"									
3/4"	80	100	100				100	93	81-93
5/8"									
1/2"									
3/8"	27	59	100				95	65	57-73
No.4	5	8	91				78	42	40-56
No.8	3	2	58				63	29	28-43
No.16									
No.30	2	1	23				31	13	13-25
No.50	1	1	15				22	9	9-19
No.100	0.8	0.6	12.7				16.5	6.9	6-10
No.200	0.6	0.4	11.4				12.8	5.7	2.5-6.5

Requested: Matt Nelson - 2674
Contractor Personnel and Lab Tech Cert No.

Approved: Billy [signature] 1537
Supervisor

Date last lab inspection 12/1/2015

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 V4.06

Date	03-17-2016	Roadway Surface	Yes
Region	1		
Hot-mix Producer	Harrison Construction - Jefferson City Asphalt Mix		

Type	ACS-HM	Mix	411-D PG 64-22	Item	411-01.10

Serial No.:		Design No.:	1160371

Material	Size or Grade	Producer and Location	Percent Used
Granite	D Rock	Harrison Construction - Waynesville NC Aggregate	47.150
Soft Limestone (aka Non-Sur	#10	Aggregate USA - Jefferson City	22.632
Natural Sand	Natural Sand	Vulcan Materials - Greeneville Greystone Rd Sand	23.575
Baghouse Fines	Baghouse Fines	RAP - Harrison Construction - Morristown Asphalt Mix	0.943
Asphalt Cement	PG 64-22	PHILLIPS 66, KNOXVILLE	5.700

Percent AC in RAP1:		Optimum AC Content:	5.70	Total	100.000	
Percent AC in RAP2:		Anti-Strip Supplier:	Tri-State Sand LLC			
Anti-Strip Additive:	Arr-Maz	Dosage:	0.5%			
AC Contribution:	Virgin AC	5.70	RAP AC		Percent Virgin AC:	
Asphalt Sp. Gravity:	1.03	Dust to Asphalt Ratio:	0.83			

% Fracture Face on CA:	n/a	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:		Eff. Gravity of Agg:	2.786
Theo.Gravity of RAP2:			

Theo. Gravity of Mix:	2.539	T.S.R.:	98.7	Lbs/Ft^3:	158.4
L.O.I.:	12.0	Ignition Oven Corr. Factor:	0.45		
		Warm Mix?	No		

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	290	Mixing Temp Range(°F):	270ºF ≤ T ≤ 310ºF
Lab Compaction Temp (± 5 °F):	270	Delivery Temperature(°F):	270ºF ≤ T ≤ 310ºF

Sieve Size	Percents Used								
	D Rock	#10	Natural Sand	Baghouse Fines				% Req.	Design Range
	50.0	24.0	25.0	1.0				100	
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"	100	100	100	100				100	100
1/2"	100	100	100	100				100	95-100
3/8"	81	100	100	100				91	80-93
No.4	25	91	99	100				60	54-76
No.8	5	58	86	100				39	35-57
No.16									
No.30	4	25	51	100				22	17-29
No.50	3	18	26	100				13	10-18
No.100	2.0	13.5	9.6	100.0				7.6	3-10
No.200	1.0	10.0	3.3	100.0				4.7	0-6.5

Requested: Bobby R Everett LT-018
Contractor Personnel and Lab Tech Cert No.

Approved:
Regional Materials and Tests Supervisor

Date last lab inspection 14-12-15

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 V4.10

Date 04/12/2016 Roadway Surface Yes

Region 1

Hot-mix Producer Rogers Group - Knoxville Candora Ave Asphalt Mix

Type ACS-HM Mix 411-D PG 76-22 Item

Serial No.:	Design No.: 1160463

Material	Size or Grade	Producer and Location	Percent Used
Hard Limestone (Type II)	#7	Rogers Group - Englewood Aggregate	33.005
Slag	Coarse Slag	Tube City IMS - Knoxville Aggregate	9.430
Hard Limestone (Type II)	#10 - Washed	Rogers Group - Englewood Aggregate	18.860
Soft Limestone (aka Non-Sur	#10	Rogers Group - Oak Ridge Aggregate	9.430
Natural Sand	Natural Sand	Newport Sand & Gravel	14.145
RAP	RAP Processed -1/2	RAP - Rogers Group - Knoxville Candora Ave Asphalt Mix	9.979
Asphalt Cement	PG 76-22	MARATHON PETROLEUM CO., KNOXVILLE	5.151

Percent AC in RAP1:		Optimum AC Content:		5.70	Total	100.000
Percent AC in RAP2:	5.5		Anti-Strip Supplier:		Tri-State Sand LLC	
Anti-Strip Additive:		ARR-MAZZ		Dosage:		0.5%
AC Contribution:	Virgin AC	5.15	RAP AC	0.55	Percent Virgin AC:	90.4
Asphalt Sp. Gravity:		1.025		Dust to Asphalt Ratio:		0.89

% Fracture Face on CA:	n/a	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:		Eff. Gravity of Agg:	2.795
Theo.Gravity of RAP2:			
Theo. Gravity of Mix: 2.544	T.S.R.: 85.8	Lbs/Ft³:	158.8
L.O.I.:		Ignition Oven Corr. Factor:	0.51
		Warm Mix?	No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	310	Mixing Temp Range(°F):	290ºF ≤ T ≤ 330ºF
Lab Compaction Temp (± 5 °F):	275	Delivery Temperature(°F):	290ºF ≤ T ≤ 330ºF

	Percents Used								
Sieve	#7	Coarse Slag	#10 - Washed	#10	Natural Sand		RAP Processed - 1/2	% Req.	Design
Size	35.0	10.0	20.0	10.0	15.0		10.0	100	Range
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"	100	100	100	100	100		100	100	100
1/2"	98	100	100	100	100		100	99	95-100
3/8"	75	82	100	100	100		97	89	80-93
No.4	25	46	97	78	99		76	63	54-76
No.8	8	28	67	48	88		58	43	35-57
No.16									
No.30	3	12	32	24	47		34	21	17-29
No.50	2	7	19	17	22		24	13	10-18
No.100	2.0	5.0	9.5	14.5	8.9		19.0	7.8	3-10
No.200	1.0	3.5	6.0	11.9	4.0		13.8	5.1	0-6.5

Requested: Jeremy S Dickson, 2094
Contractor Personnel and Lab Tech Cert No.

Approved: Billy [illegible] 1537

Date last lab inspection 11/19/2014

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 V4.10

Date	01-11-2016	Roadway Surface	No
Region	3		
Hot-mix Producer	LoJac - Hermitage		

Type BPMB-HM Mix 307-BM2 PG 64-22 Item 307-01.08

Serial No.:	16M0017	Design No.:	3160011

Material	Size or Grade	Producer and Location	Percent Used
Soft Limestone (aka Non-Sur	BM-2 Rock	Vulcan Materials - Hermitage	28.740
Soft Limestone (aka Non-Sur	#7	Vulcan Materials - Hermitage	23.950
Natural Sand	Natural Sand	Pine Bluff Sand & Gravel - Nashville	19.160
RAP	Recycled Shingles (RAS) - 3/8	RAS - Ground Up Recycling, Mt Juliet	3.342
RAP	RAP Processed -3/4	RAP - LoJac - Hermitage	22.069
Asphalt Cement	PG 64-22	MARATHON PETROLEUM CO., NASHVILLE TERMINAL	2.739

Percent AC in RAP1:	14.0	Optimum AC Content:		4.20	Total	100.000
Percent AC in RAP2:	4.5		Anti-Strip Supplier:		Westvaco Polychemical Dept	
Anti-Strip Additive:		Evotherm M1 @ Terminal		Dosage:		0.5%
AC Contribution:	Virgin AC	2.74	RAP AC	1.46	Percent Virgin AC:	65.2
Asphalt Sp. Gravity:		1.03		Dust to Asphalt Ratio:		1.11

% Fracture Face on CA:	n/a	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:	2.65	Eff. Gravity of Agg:	2.667
Theo.Gravity of RAP2:	2.68		
Theo. Gravity of Mix: 2.500	T.S.R.: 87.1	Lbs/Ft^3:	156.0
L.O.I.:		Ignition Oven Corr. Factor:	
		Warm Mix?	No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	300	Mixing Temp Range(°F):	270ºF ≤ T ≤ 310ºF
Lab Compaction Temp (± 5 °F):	270	Delivery Temperature(°F):	270ºF ≤ T ≤ 310ºF

Sieve Size	Percents Used							% Req.	Design Range
		BM-2 Rock	#7	Natural Sand		Recycled Shingles (RAS) - 3/8	RAP Processed - 3/4		
		30.0	25.0	20.0		3.0	22.0	100	
2"									
1.5"									
1.25"		100	100	100		100	100	100	100
1"									
3/4"		78	100	100		100	100	93	81-93
5/8"									
1/2"									
3/8"		33	70	100		100	92	71	57-73
No.4		7	14	98		98	74	44	40-56
No.8		6	6	92		95	56	37	28-43
No.16									
No.30		5	5	61		55	30	23	13-25
No.50		4	4	12		45	22	11	9-19
No.100		2.5	2.5	3.0		31.0	18.0	6.9	6-10
No.200		1.5	1.5	1.0		19.0	14.0	4.7	2.5-6.5

Requested: LoJac Ent.Casin Swann PT1826
Contractor Personnel and Lab Tech Cert No.

Approved: ____________
Regional Materials and Tests Supervisor

Date last lab inspection 05-03-15

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

Date	05/16/2016	Roadway Surface	Yes
Region	4		
Hot-mix Producer	Ford Construction - Dresden Asphalt Mix		

Type	ACS-HM	Mix	411-D PG 64-22	Item	411-01.10

Serial No.:		Design No.:	4160125

Material	Size or Grade	Producer and Location	Percent Used
Gravel	D Rock	Ford Construction - Troy Aggregate	47.050
Soft Limestone (aka Non-Sur	#10	Vulcan Materials - Grand Rivers KY	23.525
Natural Sand	Natural Sand	Ford Construction - Troy Aggregate	23.525
Asphalt Cement	PG 64-22	ERGON ASPHALT & EMULSIONS, MEMPHIS, TN	5.900

Percent AC in RAP1:		Optimum AC Content:	5.90	Total	100.000
Percent AC in RAP2:			Anti-Strip Supplier:	Arr-Mazz Products	
Anti-Strip Additive:		Ad-here LA-2		Dosage:	0.5%
AC Contribution:	Virgin AC	5.90	RAP AC		Percent Virgin AC:
Asphalt Sp. Gravity:		1.0218		Dust to Asphalt Ratio:	0.93

% Fracture Face on CA:	89.6	% Glassy Particles on CA:	n/a
Theo. Gravity of RAP1:		Eff. Gravity of Agg:	2.529
Theo. Gravity of RAP2:			

Theo. Gravity of Mix:	2.327	T.S.R.:	94.0	Lbs/Ft^3:	145.2
L.O.I.:	8.5	Ignition Oven Corr. Factor:			0.54
		Warm Mix?			No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	310	Mixing Temp Range(°F):	270°F ≤ T ≤ 310°F
Lab Compaction Temp (± 5 °F):	290	Delivery Temperature(°F):	270°F ≤ T ≤ 310°F

	Percents Used								
Sieve Size	D Rock	#10	Natural Sand					% Req.	Design Range
	50.0	25.0	25.0					100	
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"	100	100	100					100	100
1/2"	98	100	100					99	95-100
3/8"	85	100	100					93	80-93
No.4	40	97	98					69	54-76
No.8	18	72	84					48	35-57
No.16									
No.30	9	38	60					29	17-29
No.50	7	30	18					16	10-18
No.100	4.0	20.0	2.0					7.5	3-10
No.200	3.0	15.0	1.0					5.5	0-6.5

Requested: Ken Summers LT-3401
Contractor Personnel and Lab Tech Cert No

Approved: 5-16-16
Regional Materials and Tests Supervisor
FOR MB

Date last lab inspection 2/3/2015

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 | V4.08

Date 03-14-2016 | Roadway Surface Yes

Region 1

Hot-mix Producer Blalock Asphalt - Sevierville Asphalt Mix

Type ACS-HM | Mix 411-D PG 70-22 | Item 411-02.10

Serial No.:	Design No.:	1160315

Material	Size or Grade	Producer and Location	Percent Used
Gravel	D Rock	Vulcan Materials - Greeneville Greystone Rd Sand	47.100
Soft Limestone (aka Non-Sur	#10	Vulcan Materials - Sevierville	23.550
Natural Sand	Natural Sand	Vulcan Materials - Greeneville Greystone Rd Sand	23.550
Asphalt Cement	PG 70-22	PHILLIPS 66, KNOXVILLE	5.800

Percent AC in RAP1:		Optimum AC Content:	5.80	Total	100.000
Percent AC in RAP2:		Anti-Strip Supplier:		Tri-State Sand LLC	
Anti-Strip Additive:		Pavegrip	Dosage:	0.5%	
AC Contribution:	Virgin AC 5.80	RAP AC		Percent Virgin AC:	
Asphalt Sp. Gravity:	1.033	Dust to Asphalt Ratio:		0.92	

% Fracture Face on CA:	91	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:		Eff. Gravity of Agg:	2.644
Theo.Gravity of RAP2:			
Theo. Gravity of Mix:	2.425	T.S.R.: 89.7	Lbs/Ft^3: 151.3
L.O.I.:	10.8	Ignition Oven Corr. Factor:	0.45
		Warm Mix?	No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	310	Mixing Temp Range(°F):	290ºF ≤ T ≤ 330ºF
Lab Compaction Temp (± 5 °F):	300	Delivery Temperature(°F):	290ºF ≤ T ≤ 330ºF

	Percents Used								
Sieve	D Rock	#10	Natural Sand					% Req.	Design
Size	50.0	25.0	25.0					100	Range
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"	100	100	100					100	100
1/2"	90	100	100					95	95-100
3/8"	66	100	100					83	80-93
No.4	22	93	98					59	54-76
No.8	14	60	84					43	35-57
No.16									
No.30	11	25	51					25	17-29
No.50	8	17	25					15	10-18
No.100	5.2	13.3	8.9					8.2	3-10
No.200	3.1	11.1	4.0					5.3	0-6.5

Requested: Josh Lawrence LT-728 | Approved:

Contractor Personnel and Lab Tech Cert No. | Regional Materials and Tests Supervisor

Date last lab inspection 07-12-15

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

V4 10

Date: 01/20/2016 — Roadway Surface: Yes
Region: 4
Hot-mix Producer: Lehman Roberts - Memphis Carrier Asphalt Mix

Type: ACS-HM — Mix: 411-D PG 76-22 — Item:

Serial No.:	Design No.: 4160010

Material	Size or Grade	Producer and Location	Percent Used
Gravel	D Rock	Memphis Stone & Gravel - Arlington Aggregate	47.050
Natural Sand	Natural Sand	Memphis Stone & Gravel - Arlington Aggregate	16.938
Soft Limestone (aka Non-Su	#10	Vulcan Materials - Memphis	15.997
RAP	RAP Processed -1/2	RAP - Lehman Roberts - Memphis Carrier Asphalt Mix	4.979
RAP	RAP Processed - 5/16	RAP - Lehman Roberts - Memphis Carrier Asphalt Mix	9.958
Asphalt Cement	PG 76-22	MARATHON PETROLEUM CO., MEMPHIS, TN	5.078

Percent AC in RAP1:	5.5	Optimum AC Content:	5.90	Total	100.000
Percent AC in RAP2:	5.5	Anti-Strip Supplier:			Tri-State Sand LLC
Anti-Strip Additive:		Ad-Here 7700		Dosage:	0.5%
AC Contribution:	Virgin AC 5.08	RAP AC 0.82		Percent Virgin AC:	86.1
Asphalt Sp. Gravity:	1.03	Dust to Asphalt Ratio:			0.94

% Fracture Face on CA:	92	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:	2.616	Eff. Gravity of Agg:	2.605
Theo.Gravity of RAP2:	2.668		
Theo. Gravity of Mix:	2.389	T.S.R.: 91.8	Lbs/Ft^3: 149.1
L.O.I.:	9.1	Ignition Oven Corr. Factor:	0.22
		Warm Mix?	No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	320	Mixing Temp Range(°F):	290ºF ≤ T ≤ 330ºF
Lab Compaction Temp (± 5 °F):	320	Delivery Temperature(°F):	290ºF ≤ T ≤ 330ºF

Sieve Size	D Rock	Natural Sand	#10			RAP Processed - 1/2	RAP Processed - 5/16	% Req.	Design Range
	50.0	18.0	17.0			5.0	10.0	100	
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"	100	100	100			100	100	100	100
1/2"	91	100	100			97	100	95	95-100
3/8"	76	100	100			92	99	87	80-93
No.4	42	97	96			63	82	66	54-76
No.8	24	81	73			50	63	48	35-57
No.16									
No.30	10	49	39			28	40	26	17-29
No.50	6	14	28			15	23	13	10-18
No.100	4.0	1.1	21.0			7.2	15.0	7.6	3-10
No.200	3.0	0.5	15.0			6.2	11.0	5.6	0-6.5

Requested: Tony Crutcher 1863 Cert#1863
Contractor Personnel and Lab Tech Cert No.

Approved: Larry Doles 3-28-16 FOR MB
Regional Materials and Tests Supervisor

Date last lab inspection 2/4/2014

UPDATED 7/11/16

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 V4.10

Date	01/19/2016	Roadway Surface	Yes
Region	4		
Hot-mix Producer	Standard Construction - Cordova Asphalt Mix		
Type	ACS-HM	Mix	411-D PG 76-22
Item			

Serial No.:		Design No.:	4160049

Material	Size or Grade	Producer and Location	Percent Used
Gravel	D Rock	Standard Construction - Millington Aggregate	45.216
Soft Limestone (aka Non-Sur	#10	Fullen Dock and Warehouse	16.014
Natural Sand	Natural Sand	Standard Construction - Byhalia Mississippi	18.840
RAP	RAP Processed - 5/16	RAP - Standard Construction - Cordova Asphalt Mix	10.064
RAP	RAP Processed -1/2	RAP - Standard Construction - Cordova Asphalt Mix	4.898
Asphalt Cement	PG 76-22	ERGON ASPHALT & EMULSIONS, MEMPHIS, TN	4.968

Percent AC in RAP1:	6.4	Optimum AC Content:		5.80	Total	100.000
Percent AC in RAP2:	3.8		Anti-Strip Supplier:			Arr-Mazz Products
Anti-Strip Additive:		Adhere LA-2/Aqua Foaming		Dosage:		0.5%
AC Contribution:	Virgin AC	4.97	RAP AC	0.83	Percent Virgin AC:	85.7
Asphalt Sp. Gravity:		1.025	Dust to Asphalt Ratio:			1.03

% Fracture Face on CA:	93.4	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:	2.525	Eff. Gravity of Agg:	2.591
Theo.Gravity of RAP2:	2.497		

Theo. Gravity of Mix:	2.380	T.S.R.:	89.5	Lbs/Ft^3:	148.5
L.O.I.: (1)	9.0	Ignition Oven Corr. Factor:			0.32
		Warm Mix?			No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	310	Mixing Temp Range(°F):	290ºF ≤ T ≤ 330ºF
Lab Compaction Temp (± 5 °F):	300	Delivery Temperature(°F):	290ºF ≤ T ≤ 330ºF

	Percents Used								
Sieve Size	D Rock 48.0	#10 17.0	Natural Sand 20.0			RAP Processed - 5/16 10.0	RAP Processed - 1/2 5.0	% Req. 100	Design Range
2"									
1.5"			(1) UPDATED LOI TARGET FROM 10.9						
1.25"			TO 9.0 WS 7/11/16						
1"									
3/4"									
5/8"	100	100	100			100	100	100	100
1/2"	95	100	100			100	100	98	95-100
3/8"	77	100	100			100	85	88	80-93
No.4	49	91	97			90	40	69	54-76
No.8	30	45	87			67	33	48	35-57
No.16									
No.30	13	19	64			41	15	27	17-29
No.50	8	15	11			24	10	12	10-18
No.100	6.4	13.7	0.8			17.2	7.5	7.7	3-10
No.200	4.8	11.3	0.5			13.4	5.8	6.0	0-6.5

Requested: Blake Carroll LT2115
Contractor Personnel and Lab Tech Cert No.

Approved: [signature] 7/11/16
Regional Materials and Tests Supervisor

Date last lab inspection 3/11/2014

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2015 V4.0 V4.10

Date	01/07/2016	Roadway Surface	Yes
Region	4		
Hot-mix Producer	Standard Construction - Cordova Asphalt Mix		

Type BPMB-HM Mix 307-BM2 PG 76-22 Item

Serial No.:	Design No.: 4160056

Material	Size or Grade	Producer and Location	Percent Used
Gravel	BM-2 Rock	Standard Construction - Millington Aggregate	19.040
Soft Limestone (aka Non-Sur	#57	Fullen Dock and Warehouse	19.040
Natural Sand	Natural Sand	Standard Construction - Millington Aggregate	13.328
Soft Limestone (aka Non-Sur	#10	Fullen Dock and Warehouse	10.472
RAP	RAP Processed -1/2	RAP - Standard Construction - Cordova Asphalt Mix	16.754
RAP	RAP Processed - 5/16	RAP - Standard Construction - Cordova Asphalt Mix	18.210
Asphalt Cement	PG 76-22	ERGON ASPHALT & EMULSIONS, MEMPHIS, TN	3.156

Percent AC in RAP1:	3.4	Optimum AC Content:	4.80	Total	100.000
Percent AC in RAP2:	5.9		Anti-Strip Supplier:	Arr-Mazz Products	
Anti-Strip Additive:	Adhere LA-2/Aqua Foaming		Dosage:	0.5%	
AC Contribution:	Virgin AC 3.16	RAP AC 1.64	Percent Virgin AC:	65.7	
Asphalt Sp. Gravity:	1.029		Dust to Asphalt Ratio:	1.24	

% Fracture Face on CA:	94.8	% Glassy Particles on CA:	n/a
Theo.Gravity of RAP1:	2.525	Eff. Gravity of Agg:	2.630
Theo.Gravity of RAP2:	2.497		
Theo. Gravity of Mix: 2.447	T.S.R.: 89.2	Lbs/Ft^3:	152.7
L.O.I.: ~~10.6~~		Ignition Oven Corr. Factor:	0.35
		Warm Mix?	No

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	310	Mixing Temp Range(°F):	290ºF ≤ T ≤ 330ºF
Lab Compaction Temp (± 5 °F):	300	Delivery Temperature(°F):	290ºF ≤ T ≤ 330ºF

Sieve Size	BM-2 Rock 20.0	#57 20.0	Natural Sand 14.0	#10 11.0		RAP Processed - 1/2 17.0	RAP Processed - 5/16 18.0	% Req. ✓ 100	Design Range
2"									
1.5"									
1.25"	100	100	100	100		100	100	100	100
1"									
3/4"	85	79	100	100		100	100	93	81-93
5/8"									
1/2"									
3/8"	55	19	100	100		85	100	72	57-73
No.4	30	4	97	91		40	90	53	40-56
No.8	17	3	87	45		33	67	39	28-43
No.16									
No.30	8	3	64	19		15	41	23	13-25
No.50	7	2	11	15		10	24	11	9-19
No.100	4.4	2.2	1.4	13.7		7.5	17.2	7.4	6-10
No.200	3.9	1.9	1.0	11.3		5.8	13.4	6.0	2.5-6.5

Requested: Blake Carroll LT2115
Contractor Personnel and Lab Tech Cert No.

Approved: [signature]
Regional Materials and Tests Supervisor
2/26/16

Date last lab inspection 3/11/2014