OPTIMIZING THE USE OF RECLAIMED ASPHALT PAVEMENT (RAP) IN HOT MIX ASPHALT SURFACE MIXES

FABRIZIO MERONI

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Gerardo W. Flintsch, Chair
Alexander S. Brand
Filippo Giustozzi
Samer W. Katicha
Robert L. West

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ABSTRACT

The most common use of reclaimed asphalt pavement (RAP) is in the lower layers of a pavement structure, where it has been proven as a valid substitute for virgin materials. Instead, the use of RAP in surface mixes is more limited, with a major concern being that the high RAP mixes may not perform as well as traditional mixes. To reduce risks of compromised performance, the use of RAP has commonly been controlled by specifications that limit the allowed amount of recycled material in the mixes. However, significant cost and environmental savings can be achieved if more RAP is included in the surface layer. This dissertation develops an approach that can be followed to incorporate more RAP in the surface mix while maintaining good performance. The approach is based on the results from three studies that looked at how to optimize the design of the mix, in terms of rutting and fatigue resistance, when more RAP is used.

In the first study, a high RAP control mix and an optimized mix designed using different design compaction energy (65 and 50 gyrations respectively) were compared. The optimization process consisted in the definition of an alternative mix composition that supported the higher binder content allowed by the lower design compaction energy. Using Accelerated Pavement Testing and laboratory characterization it was possible to assess the potential of mix optimization with the objective of improving rutting resistance. The testing showed no indication that the optimized mixes would have rutting problems, supporting the implementation of the reduction of the design compaction energy level. The optimized mix exhibited a similar or superior rutting resistance in the full-scale setting, in the laboratory, and in the forensic investigation.

The second part focused on the production of highly recycled surface mixes capable of performing well. To produce the mixes, a balanced mix design (BMD) methodology was used and a comparison with traditional mixes, prepared in accordance with the requirements of the Virginia Department of Transportation (VDOT) volumetric mix design, was performed. Through the BMD procedure, which featured the indirect tensile cracking test for evaluating the cracking resistance and the Asphalt Pavement Analyzer for evaluating rutting resistance, it was possible to optimize the selection of the optimum asphalt content. Also, it was possible to obtain a highly recycled mix (45% RAP) capable of

achieving better overall performances than traditional mixes while carrying a large reduction in production cost.

The final part evaluated the laboratory performance of four different highly recycled surface mixes to support their possible implementation in the state of Virginia. The mixes featured either 30% or 45% RAP, different asphalt contents, the use of a WMA additive, and a rejuvenator. To analyze the mixes' performance in great depth, a three-level (base, intermediate, and advanced) testing framework was defined. Each level was characterized by an increasing degree of complexity and included tests to characterize both the cracking resistance and the rutting resistance. The study aimed at investigating the features of the various laboratory tests. Through the review of the theoretical background, the evaluation of the test procedures, and statistical analysis of the results, it was possible to identify the strengths and weaknesses of each test and to provide guidelines to develop appropriate quality assessment criteria and mix design methodology.

In summary, throughout this research, it was possible to observe that the respect of Superpave mix design requirements alone, with particular reference to gradation limits and volumetric properties, was not guarantee of satisfactory performance in terms of both cracking and rutting resistance. To increase the confidence in the RAP properties, increase the current recycling levels, and introduce more appropriate mix design specifications, BMD could be used (even with simple laboratory tests) to check performance-based criteria.

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GENERAL AUDIENCE ABSTRACT

Nowadays, transportation agencies are expected to perform a large number of pavement rehabilitation projects, while facing major limitations in budgetary funds. In order to have safe, efficient, and cost-effective roadways, the economic advantage of recycling is boosting an effort to increase the amount of RAP in asphalt mixtures. In addition, over the past decades, the environmental awareness of the transportation agencies and public increased significantly, pushing even more towards the use of new green technologies.

The use of RAP became noticeable in the 1970s and its popularity increased significantly since that time. However, there are still many open questions which prevent larger uses of recycled materials, mainly related to the design methodology and the field performances of recycled mixtures. Therefore, today there is a large untapped potential that would grow even more the magnitude of pavement recycling and of the associated benefits.

New design procedures, based on the support of laboratory tests to characterize the mixtures, and full-scale experiments are the tools that pavement engineers can use in order to enrich the knowledge of highly recycled road materials and grow the confidence of public agencies and contractors towards these new more sustainable solutions.

Throughout this dissertation it was possible to evaluate new innovative ways of incorporating more RAP in the asphalt mixtures through the analysis of current state of the art and the proposition of new procedures.

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I also want to thank my other committee members: Dr. Alexander Brand, Dr. Samer Katicha, and Dr. Robert West. Their amazing knowledge and mastery of the field were matched by their helpfulness and kindness. Each of the committee members brought their unique perspectives which were indispensable for making this dissertation possible.

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Finally, this section would not be complete without acknowledging my family: my parents, who always motivated me to chase my dreams, no matter how hard they seemed to be, and my brother which, no matter how far, has always been a reference and a guide for me.

DEDICATION

This dissertation is dedicated to my caring and supportive parents: my mother Luisella Grugni and my father Giuseppe Meroni who taught me to always believe in myself and that no challenge is too big, no matter the circumstances.

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AASHTO: American Association of State Highway and Transportation Officials
ALDOT: Alabama Department of Transportation
ALF: Accelerated Loading Facility
AMPT: Asphalt Mixture Performance Tester
APA: Asphalt Pavement Analyzer
APAS: Accelerated Pavement Aging System
APT: Accelerated Pavement Testing
ARRA: Asphalt Recycling and Reclaiming Association
ASTM: American Society for Testing and Materials
BMD: Balanced Mix Design
CALTRANS: California Department of Transportation
CFN: Confined Flow Number
CMOD: Crack Mouth Opening Displacement
DCT: Disk-Shaped Compact Tension
DSR: Dynamic Shear Rheometer
DT: Direct Tension
ESAL: Equivalent Single Axle Load
FHWA: Federal Highway Administration
FI: Flexibility Index
FDOT: Florida Department of Transportation
FN: Flow Number
GHG: Greenhouse Gas
GLWT: Georgia Loaded Wheel Tester
GWP: Global Warming Potential
HMA: Hot Mix Asphalt
HVS: Heavy Vehicle Simulator
HWTT: Hamburg Wheel Tracking Test
IDT: Indirect Tensile
ITTD: Indirect Tensile Test Device
LaDOTD: Louisiana Department of Transportation and Development
LLD: Load Line Displacement
LTRC: Louisiana Transportation Research Center
LVDT: Linear Variable Differential Transformer
LWT: Loaded Wheel Tester
MDD: Multi-Depth Deflectometer
MTS: Material Test System
NAPA: National Asphalt Pavement Association
NCHRP: National Cooperative Highway Research Program
NJDOT: New Jersey Department of Transportation
NMAS: Nominal Maximum Aggregate Size
OT: Overlay Test
PG: Performance Grade
RAP: Reclaimed Asphalt Pavement
RAPBR: RAP Binder Ratio
RAS: Reclaimed Asphalt Shingles
RSCH: Repeated Shear Test at Constant Height

RTFO: Rolling Thin Film Oven
SAPA: State Asphalt Pavement Associations
SBR: Styrene-Butadiene-Rubber
SBS: Styrene-Butadiene-Styrene
SCB: Semi Circular Bend
SHRP: Strategic Highway Research Program
SM: Surface Mix
SMA: Stone Matrix Asphalt
SPT: Simple Performance Test
SSR: Stress Sweep Rutting
SST: Superpave Shear Tester
S-VECD: Simplified Viscoelastic Continuum Damage
TSR: Tensile Strength Ratio
TSRST: Thermal Stress Restrained Specimen Test
TxDOT: Texas Department of Transportation
UNEP: United Nations Environment Program
UTSST: Uniaxial Thermal Stress and Strain Test
VDOT: Virginia Department of Transportation
VTRC: Virginia Transportation Research Council
VTTI: Virginia Tech Transportation Institute
VFA: Voids Filled with Asphalt
VMA: Voids in Mineral Aggregate
WMA: Warm Mix Asphalt
WRC: Wheel Reflection Cracking
WTI: West Texas Intermediate

CHAPTER 1 – INTRODUCTION

1.1 OVERVIEW

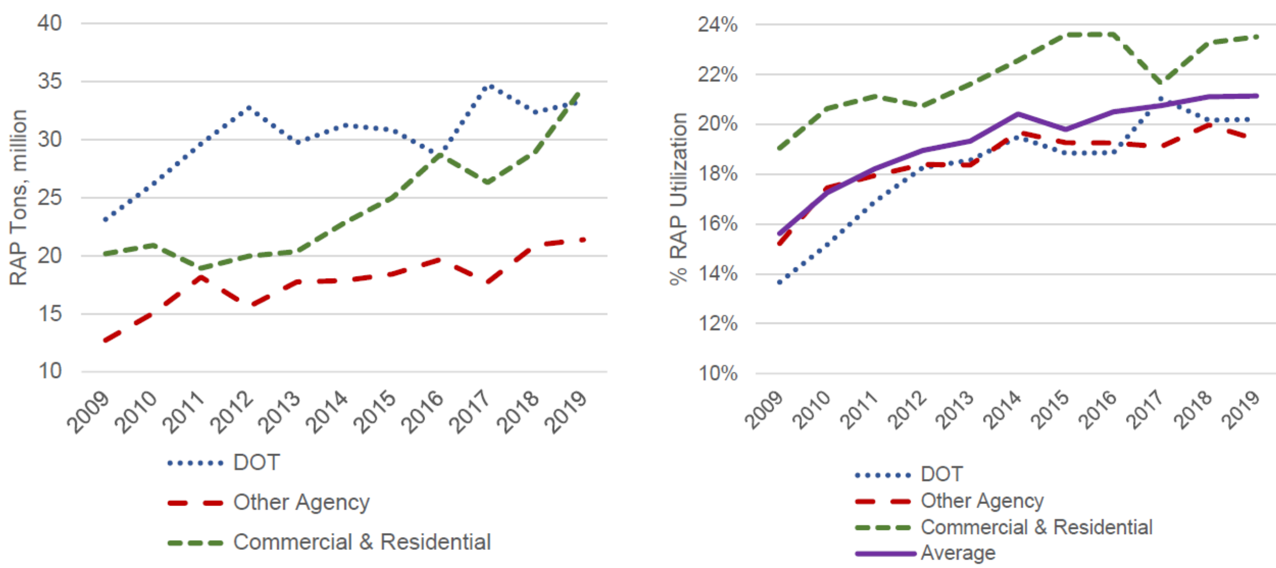
The interest of the asphalt industry towards recycling and the use of reclaimed asphalt pavement (RAP) was originally sparked in the 1970s when the oil crisis caused an increase in the price of bitumen. However, pavement recycling turned out to be a very valuable tool for many other reasons, which still today contribute to building a sustainable transportation infrastructure. Besides reducing the initial cost of asphalt mixtures, it has been demonstrated that the use of RAP helps saving natural resources, reducing landfill, and lowering emissions. Because of these substantial environmental benefits, many countries have implemented laws that require a minimum percentage of RAP be used in the production of asphalt mixtures (ARRA, 2001). The use of RAP contributes to the Green Economy, which aims at "improving human well-being and social equity, while significantly reducing environmental risks and ecological scarcities" as defined by the United Nations Environment Program (UNEP, 2010). However, in order for that contribution to be a positive one, the initial economic savings, have to be accompanied by adequate field performance. If the RAP inclusion results in shorter pavement lifespans or inappropriate performances, the initial benefits would become irrelevant because of the necessity of further construction works and maintenance operations.

Today, a 15 to 20 percent RAP content is becoming a standard practice of asphalt mixture production (Tarsi et al., 2020). In the US, the use of RAP is well established (89.2 million tons of RAP were used in 2019) and the average percentage of RAP used in asphalt mixtures (the estimated production of asphalt mixtures in 2019 was 421.9 million tons) has increased from 15.6 percent in 2009 to 21.1 percent in 2019 (Williams et al., 2020).. However, as shown in Figure 1, most of this increase occurred between 2009 and 2014 with the percentage of RAP use in asphalt mixtures remaining mostly the same after 2014.

While US advancement toward the use of more RAP in the mixtures has practically stalled, other countries are already using more than twice the amount of RAP used in the US. For instance, in the Netherlands it is common practice to include 50% RAP in most asphalt mixtures (Mohajeri, 2015) and in Japan an average of 47 percent RAP content has been reported (West & Copeland, 2015). In both cases, a different approach to mix design and the implementation of RAP and asphalt mixture testing procedures were instrumental.

In 2018, a NAPA's survey of the state asphalt pavement associations (SAPAs) identified mixture performance testing at the design stage as the main tool that can lead to increased RAP use. One of the major impediments that have limited the use of RAP are the strict volumetric specifications used in mixture design. RAP materials contain a large percentage of fines which causes the mixture to fail volumetric restrictions if too much RAP is used. RAP fractionation can help reducing the amount of fines and hence has been shown to be beneficial in allowing more RAP use. Still, as of 2011 only five agencies allowed 30% or more RAP in their mixtures (Copeland, 2011) and, in 2017, SAPAs indicated that specifications limits are still the main limitation to a greater use of RAP (Williams et al., 2018).

This dissertation uses laboratory and full-scale accelerated pavement testing of asphalt mixtures to show that it is possible to modify the mixture specification requirements so that more RAP can be used without sacrificing mixture performance; in some cases, mixture performance can actually be improved compared to mixtures designed with current specifications.



(a) (b)

Figure 1 (a) RAP use by sector (million tons), (b) average percent RAP used by sector (Williams et al., 2020)

1.2 PROBLEM STATEMENT

RAP is a valuable, high-quality material that can be used as an aggregate and virgin asphalt binder substitute in asphalt mixtures, or other applications, such as bases, subbases, and embankments. The most expensive and economically valuable material in asphalt mixtures is the asphalt binder. Therefore, within a pavement structure, the most advantageous use of RAP is in the intermediate and surface layers of flexible pavements where the less expensive binder from RAP can replace a portion of the more expensive virgin binder (Copeland, 2011). It is clear from the previous overview section that RAP is not used at its full potential in the US. Many laboratory studies and field projects confirmed that RAP content in surface mixtures can be greatly increased from current levels. To make that happen, changes, based on mixture performance evaluation, should be made in the mix design process.

Three main factors, which currently limit the use of RAP in the surface mixture are:

- Current mix design procedures are still strongly dependent on the Superpave volumetric mix design system and prevent a more widespread use of innovative design procedures.
- The use of RAP is commonly controlled by strict design specifications that limit the allowed amount of recycled material in the mixes.
- Because there is no definitive consensus on the field performance of mixtures that incorporate high levels of RAP, transportation agencies generally regard the use of high RAP in mixtures as risky and a possible detriment to overall performance.

An appropriate performance-based mix design system can help address these factors. In fact, it has been shown that volumetric mix design restrictions still allow a wide range of mixture performance as measured in the laboratory, especially when these mixtures include new materials such as rejuvenators, polymers, recycled asphalt shingles (RAS), and recycled tire rubber (West et al., 2013). This suggests that adequately selected performance tests can be used to design good performing mixtures that incorporate higher percentages of RAP.

1.3 OBJECTIVE

The objective of the research presented in this dissertation was to propose changes in the mixture design methodologies currently used by the Commonwealth of Virginia so that more RAP can be used during mixture production. These changes are based on mixture performance evaluation and ensure that the produced mixtures perform at or above the performance levels of currently produce mixtures that incorporate lower percentages of RAP. To achieve the objective, the two following goals were set:

1. Produce and characterize optimized high-RAP content mixtures capable of at least perform as well as the traditional mixtures.
2. Show that performance-based mix design procedures can support the use of high amounts of RAP and provide insight on what performance tests would be most appropriate.

To achieve the first goal, the research aimed at evaluating highly recycled asphalt mixtures that presented innovative design solutions and comparing them to traditional mixes in terms of performance. With respect to the second goal, the study focused on the evaluation of multiple laboratory tests and the analysis of the challenges that could arise when using a performance-based mix design.

1.4 RESEARCH APPROACH

The dissertation objective was achieved by completing the following tasks:

- Literature Review. An extensive review of scientific papers, technical reports, standard methods for testing, and books was performed. This review was fundamental to guide the development of mix design procedures for recycled mixtures and evaluate the impacts of recycling on performance. Four main areas were covered: history of asphalt mix design, pavement recycling, evolution of laboratory performance testing, and full-scale testing.
- Evaluation of the feasibility of modifying the Superpave requirements currently enforced in the Commonwealth of Virginia. The goal was to verify how a mixture designed with a lower target gyrations number (50) would perform compared to a traditional one (designed with 65 gyrations). In particular, the analysis focused on the impact on the rutting performance of the higher binder content allowed into the 50-gyrations mixture. This task was completed with both laboratory tests and the use of a full-scale Heavy Vehicle Simulator.
- Implementation of a performance-based design method. A balanced mix design procedure was introduced with the objective of designing high-RAP content mixtures while checking their performance with respect to cracking and rutting.
- Laboratory performance evaluation of high-RAP mixtures. This task focus was twofold: (1) evaluate the performances, with respect to both cracking and rutting, of different highly recycled mixes through laboratory testing to assess their possible use in the field; (2) compare the ability of each test to differentiate the mixes with respect to the property investigated and provide guidance for their use in the mix design and quality assessment phases.

1.5 ORGANIZATION OF THE DISSERTATION

This dissertation is composed of six chapters and follows a manuscript format which includes a collection of papers. Each manuscript is included as an individual chapter of the dissertation.

Chapter 1 – Introduction. This chapter is an introduction to the research. The chapter provides an overview of the dissertation, establishes the problem statement and the research objectives, and the introduces research approach.

Chapter 2 – Literature Review. This chapter presents the necessary background for this dissertation. It includes a literature review that discusses several subjects fundamental to pavement recycling and the design of asphalt mixtures.

Chapter 3 – Impact of Mix Design Optimization on HMA Rutting Performance under Accelerated Pavement Testing. This paper provides the initial efforts to optimize the use of RAP by comparing two highly recycled mixes designed using different design compaction energy (65 and 50 gyrations respectively). The mixes were tested using laboratory characterization and APT to assess the potential of mix optimization by reducing the compaction energy. A reduced version of this paper was published in the Proceedings of the 6th APT Conference as part of the volume titled "*Accelerated Pavement Testing to Transport Infrastructure Innovation"*.

Chapter 4 – Application of Balanced Mix Design Methodology to Optimize Surface Mixes with High-RAP Content. This paper builds on the results of Chapter 3, and uses a balanced mix design methodology to produce highly recycled surface mixes capable of performing well in the field. The paper compares the BMD mixes with traditional mixes prepared in accordance with the requirements of the Virginia's volumetric mix design. This paper has been published in a special issue of the MDPI Materials journal titled *"Recycled Materials and By-Products for Pavement Construction"*.

Chapter 5 – Three-Level Performance Evaluation of High RAP Asphalt Surface Mixes. This paper further explores different BMD tests to evaluate the laboratory performance of four different highly recycled surface mixes to support their possible implementation in the state of Virginia. The mixes featured high RAP, different asphalt contents, the use of a WMA additive, and a rejuvenator. To analyze the mixes' performance in great depth, a three-level (base, intermediate, and advanced) testing framework was defined. Each level was characterized by an increasing degree of complexity and included tests to characterize both the cracking resistance and the rutting resistance. In addition, the study aimed at investigating the features of the various laboratory tests. This paper is going to be submitted to the "*Construction and Building Materials*" journal.

Chapter 6 – Summary, Conclusions, and Recommendations. This chapter provides a summary of the research conducted throughout this dissertation, highlights the main conclusions, and makes recommendations for future research.

1.6 ATTRIBUTIONS

This section clarifies the role of the doctoral candidate listing his contributions in each one of the papers included in the dissertation:

- Chapter 3 – Impact of Mix Design Optimization on HMA Rutting Performance under Accelerated Pavement Testing. Conduction of the literature review, preparation of the specimens, analysis (alongside Dr. Wenjing Xue), and writing of the original draft. All co-authors reviewed the final manuscript and provided feedback and recommendations.

- Chapter 4 – Application of Balanced Mix Design Methodology to Optimize Surface Mixes with High-RAP Content. Conduction of the literature review, conceptualization (alongside principal dissertation advisor Dr. Gerardo Flintsch), methodology (alongside all co-authors), testing, analysis, and writing of the original draft. All co-authors reviewed the final manuscript and provided feedback and recommendations.
- Chapter 5 – Three-Level Performance Evaluation of High RAP Asphalt Surface Mixes. Conduction of the literature review, conceptualization (alongside principal dissertation advisor Dr. Gerardo Flintsch), methodology (alongside all co-authors), testing (alongside Dr. Jhony Habbouche), analysis, and writing of the original draft. All co-authors reviewed the final manuscript and provided feedback and recommendations.

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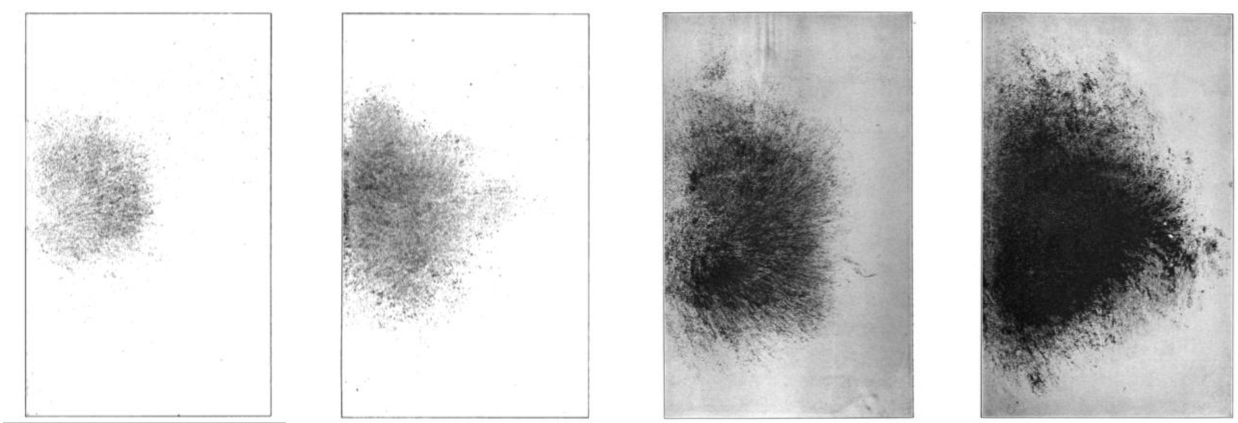
CHAPTER 2 – LITERATURE REVIEW

2.1 HISTORY OF ASPHALT MIX DESIGN

2.1.1 Early Empirical Approaches

One of the earliest examples of guidelines on how to prepare asphalt mixtures in order to build roads date back to 1890. During that year, E. G. Love published several articles on road construction and paving. In one of the articles, Col. F. V. Greene of the Barber Asphalt Paving Company illustrated how to produce a road surface mixture made of asphalt, sand, and pulverized carbonite of lime (Huber, 2013a).

At the start of the twentieth century, Richardson (1905) illustrated how the construction of asphalt pavements evolved over the years. He provided a thorough investigation of the properties, behaviors, and deterioration processes. One of the main insights was the differentiation between surface mixtures and asphaltic concrete. Surface mixtures were defined as a combination of asphalt binder, sand, and ground limestone. The reported average content of bitumen for such mixtures was around 10 percent and the aggregate gradation was 100 percent passing the sieve No. 10, which had an opening size of 2 mm (5/64″). The correct amount of binder was determined by the "pat-paper" test in which the area and intensity of bitumen stain determined the optimal asphalt content, as shown in Figure 2.



(a) (b) (c) (d)

Figure 2 Pat-paper test: (a) light stain, (b) medium stain, (c) strong stain, (d) heavy stain (Richardson, 1905)

Asphaltic concrete, instead, was formed by bitumen (between 6 and 9 percent), sand, filler, and a combination of aggregates passing sieve sizes of 6.35 mm, 12.5 mm, and 25 mm (¼", ½", and 1"). In order to design asphaltic concrete mixtures, the amount of voids in sand and mineral aggregates was calculated on compacted samples. This kind of mixture was intended as a base layer, ideal to support the surface mixture.

A big innovation came in the mid-1920s when P. Hubbard and F. C. Field of the Asphalt Institute determined a new test method to determine the optimum binder content of fine graded asphalt mixtures. At that time vehicles had solid rubber tires which, acting as concentrated loads, applied a punching effect on the roads (Kandhal, 2016). The Hubbard-Field Stability test was a punching shear type of test developed specifically for the testing of fine graded asphalt mixtures (Goetz, 1952). The test apparatus, which was quite heavy and bulky, is shown in Figure 3. The test measured the maximum load developed while extruding a mixture specimen of 50.8 mm (2 inches) in diameter and 25.4 mm (1 inch) in height through a smaller diameter orifice of 44.4-mm (1.75 inches). The laboratory samples were compacted with a hand hammer. A modified version of the Hubbard-Field test was developed for asphalt concrete. It featured test specimens of 152.4 mm (6 inches) in diameter which, after compaction, were pushed through a ring of smaller diameter. Like the original test method, in the modified version it was possible to record the maximum load sustained by the sample before the mixture passed through the orifice; this load was called Hubbard-Field Stability. In order to determine the optimum asphalt content, and in addition to the Stability value, the mixtures were analyzed in terms of volumetric properties, enriching the method first outlined by Richardson. The compacted samples were evaluated in terms of bulk specific gravity and maximum theoretical specific gravity, in order to calculate the air voids content. It can be noted that the process of asphalt absorption was not considered. In addition, the voids in the aggregate skeleton were also calculated and considered in the mix design (Huber, 2013a).

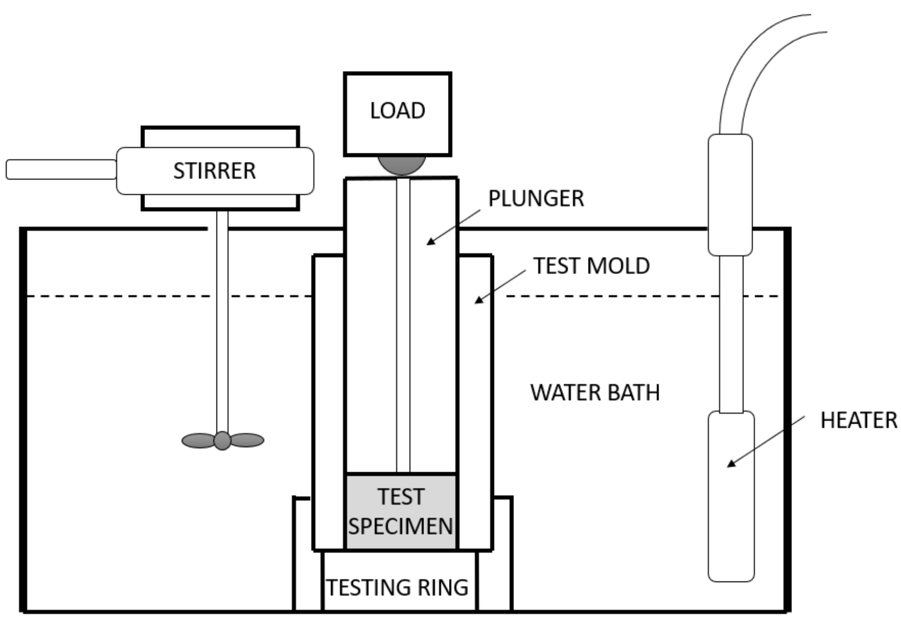


Figure 3 Hubbard-Field Stability test set-up (Asphalt Institute, 1956)

2.1.2 Traditional Mix Design Methodologies

A further step in the mix design evolution was made by Francis Hveem, who found a relationship between the gradation and the ideal amount of binder. The main representative property of the gradation was the surface area of the particles. Therefore, Hveem studied and implemented by 1932 a method to determine the optimum asphalt content based on the aggregate surface area. In addition, since the determination of the asphalt content was insufficient to guarantee enough shear strength and stability to resist deformation, Hveem developed a new stability test (Roberts et al., 2002). The test apparatus is shown in Figure 4.

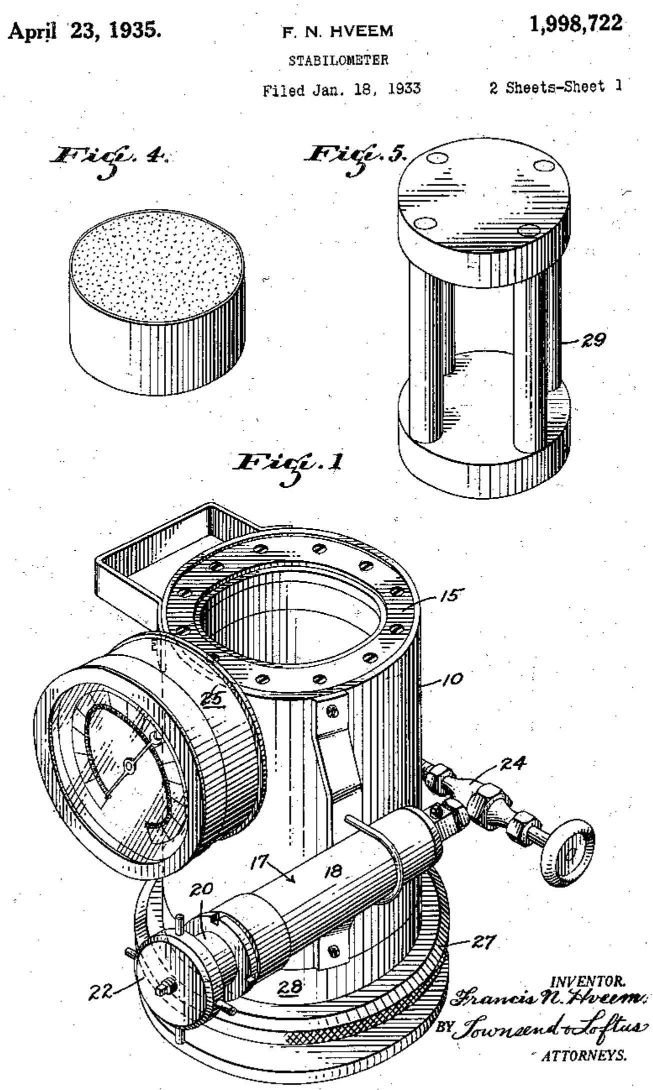


Figure 4 Drawings from the stabilometer patent (Hveem, 1935)

During the test, a vertical load was applied on a confined specimen and the resulting horizontal pressure was recorded. The intensity of the horizontal stresses indicated the degree of plasticity of the mixture being tested: a perfectly rigid solid would transmit zero pressure, while a perfect liquid would transmit a horizontal pressure equal to the vertical pressure. According to this criterion it was possible to identify stable and unstable pavement materials. The philosophy behind the selection of the design asphalt content is shown in Figure 5. In addition, a cohesiometer was used to measure the forces needed to break or bend the sample as a cantilevered beam. This information was intended to indicate the mixture resistance to raveling. It is interesting to note that air voids were not part of the Hveem mix design system. However, while the test was developed, comparisons between field cores and laboratory samples were made with particular reference to the density. It was possible to detect that the field densities were greater and therefore it was necessary to reproduce a kneading action in the laboratory similar to the one of the field rollers. The California Kneading Compactor was introduced in 1938.

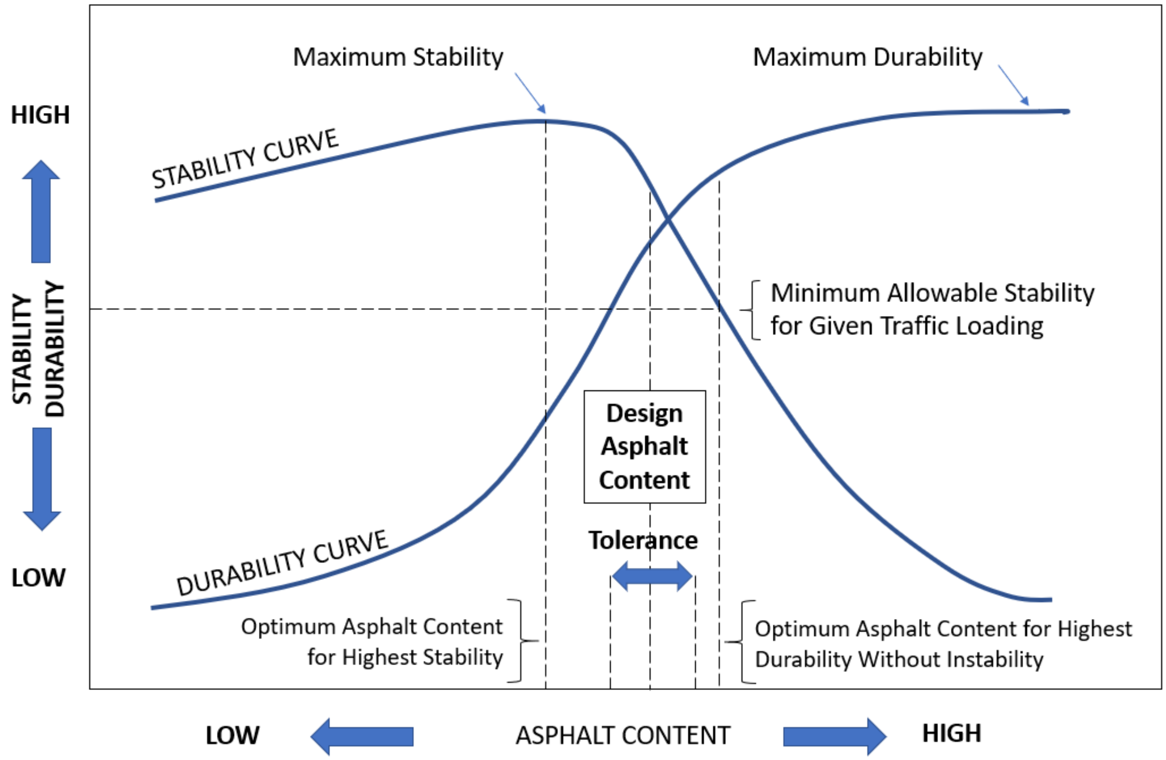


Figure 5 Stability and durability as functions of asphalt content (Brown et al., 2000)

While the Hveem method was implemented in California and other western states of the US, the majority of the states adopted another method, developed by Bruce Marshall of the Mississippi Department of Highways. The Marshall method was developed in the late 1930s to early 1940s and it is possible to say that it evolved from the Hubbard-Field procedure. In fact, the method included a laboratory test, which gave an indication of the mixture stability, and an analysis of the voids in the mixture. The Marshall test apparatus was inexpensive and portable, two important factors that contributed to its popularity.

The method featured an impulsive compaction, a water bath to cool the specimen, and the evaluation of maximum load reached in the Marshall compression device. In addition to the maximum load carried by the specimen (called Marshall stability), the corresponding vertical deformation (called flow) was recorded. The mixtures were prepared with different asphalt contents and the one which corresponded to the maximum stability was considered to be the optimum. In addition to stability and flow, other properties were checked: total specimen unit weight, aggregate unit weight, percent voids in the mineral aggregate (VMA), air voids in the compacted specimen, and percent voids filled with asphalt (Roberts et al., 2002).

After years from the implementation of both Hveem and Marshall methods, it was possible to see different trends relative to the mixtures designed in accordance with each of the procedures. In particular, it was possible to note that the Marshall compaction method was not adequate, and that the Marshall stability was not able to properly describe the shear strength of the mixture. These two factors contributed to the production of mixtures susceptible to rutting. On the other hand, the Hveem methodology required an expensive equipment, and the general concern was that the resulting mixtures had too little asphalt, therefore becoming more susceptible to cracking resistance (McGennis et al., 1995). These two mix design procedures represented the main mix design techniques until the mid-1990s, when the Superpave method was introduced.

2.1.3 Superpave

Superpave, which stood for Superior Performing Asphalt Pavements, was an asphalt mix design system which was developed as part of the Strategic Highway Research Program (SHRP) from 1987 to 1992. SHRP was a 5-year, $150 million research program which aimed at improving the United States highway system and improve the safety of users and workers. It focused on four major topics:

- Asphalt
- Concrete and structures
- Highway operations (maintenance and work-zone safety)
- Pavement performance (long-term pavement performance study)

On the subject of asphalt, the three main objectives were to develop a performance-based asphalt binder specification, an asphalt mix design system, and a series of tests for performance prediction (Huber, 2013b).

With respect to the first goal, the Performance-Graded (PG) asphalt binder specification was a major result of the research. As far as the other two objectives, the Superpave system included three levels of increasing complexity referred as Level 1, Level 2, and Level 3 mix design. The latter two included the performance-based tests which eventually, because of cost and complexity, were not implemented. Instead, the Level 1 mix design method became very successful.

Level 1 mix design was based on empirical performance-related aggregate properties, such as crushed aggregate faces and gradation, and mixture properties, such as air voids and voids in mineral aggregate. These properties were used as surrogate mixture properties to ensure adequate performance (Cominsky et al., 1994). For this reason, Level 1 design was designated as volumetric design. The main volumetric properties specified by the method were:

- Air voids
- Voids in mineral aggregate (VMA)
- Absorbed asphalt volume (V_{ba})
- Asphalt content (P_b)
- Effective asphalt volume (V_{be})
- Voids filled with asphalt (VFA)

The volumetric design method was formed by three main phases:

A. Selection of materials
B. Selection of a design aggregate structure (expressed as an aggregate gradation)
C. Selection of a design asphalt content

When selecting the aggregates, the Superpave mix design method required the observance of four aggregate properties: coarse aggregate angularity, fine aggregate angularity, flat and elongated particle determination, and clay content.

Once the different aggregates were mixed together, any trial blend gradation had to pass between predetermined control points which varied according to the Nominal Maximum Aggregate Size (NMAS) of the mixture. NMAS was defined as the one sieve dimension larger than the first sieve to retain more than 10 percent of the aggregate. The maximum sieve size was defined as one sieve larger than NMAS. The gradation control points were used to control the minimum and maximum percent passing the 75 μm (dust), the 2.36 or 4.75 mm (sand), the nominal maximum sieve size, and the maximum sieve size. For example, Figure 6 shows the gradation control points and the so called restricted zone for a 12.5 mm NMAS gradation. The restricted zone had two purposes: limiting the amounts of natural sands that could produce "humps" in the gradation curve at the 600 μm range and avoiding that gradations close to the maximum density line, which could results in inadequate voids in mineral aggregate (Kennedy et al., 1994).

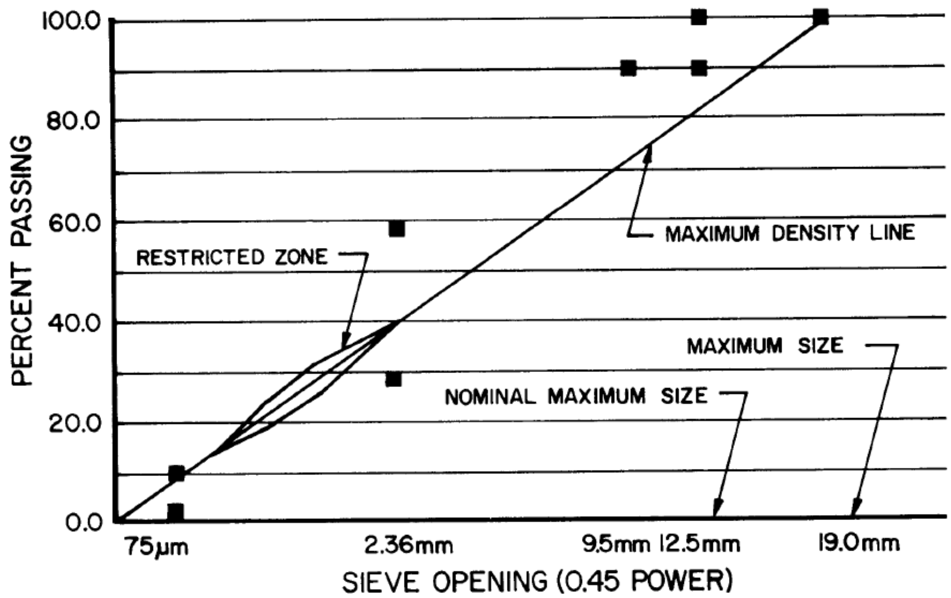


Figure 6 Superpave requirements for a 12.5 mm NMAS gradation (Kennedy et al., 1994)

A huge innovation introduced by Superpave was the gyratory compactor. The main feature of the compactor was the ability to replicate in the laboratory the properties of field cores constructed with the same asphalt aggregate combination. The Superpave gyratory compactor, shown in Figure 7 and Figure 8 had the following characteristics:

- Angle of gyration of 1.25 + 0.02°
- Rate of 30 gyrations per minute
- Vertical pressure during gyration of 600 kPa
- Capability of producing 150 mm in height by 150 mm in diameter specimens

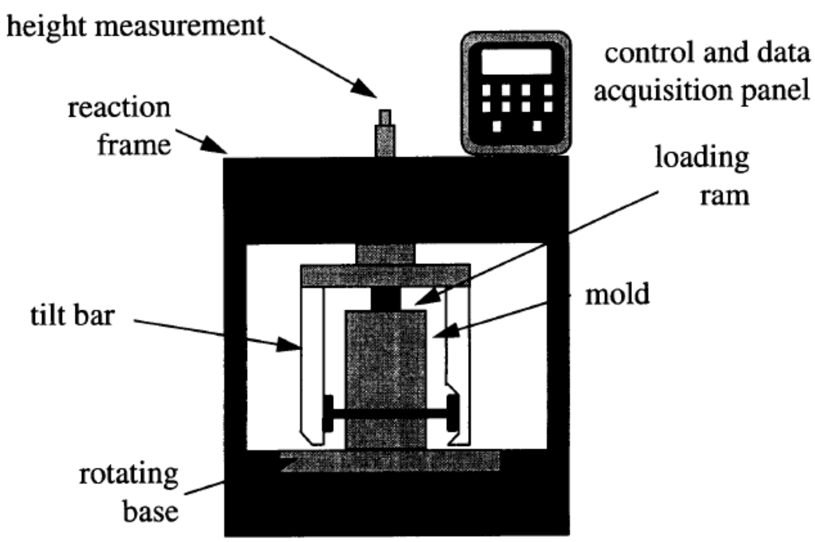


Figure 7 Gyratory compactor schematic (Kennedy et al., 1994)

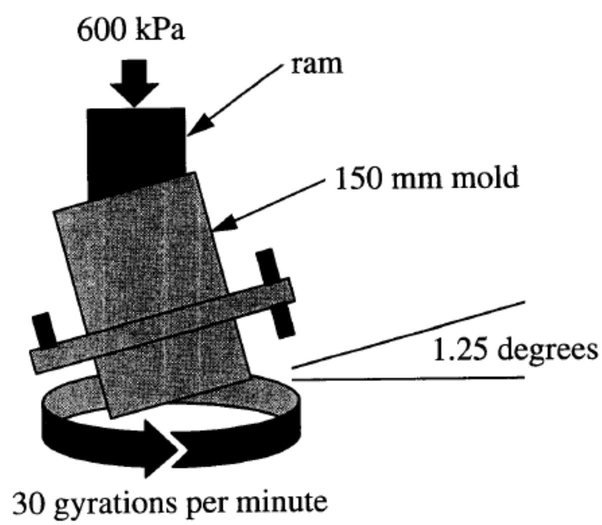


Figure 8 Gyratory compactor principle of operation (Kennedy et al., 1994)

As the compaction went on, the density of the specimen was monitored with particular reference to three points of interest. The points of interest were determined by the estimated traffic level and the environmental characteristics of the project, and were defined as:

- N_i = initial number of gyrations
- N_d = design number of gyrations
- N_m = maximum number of gyrations

The mix aggregate structure and asphalt content needed to be selected so that a compacted specimen presented 4% air voids at the design number of gyrations (N_d). Also, the mix needed to achieve at least

2% air voids at N_m and at least 11% air voids at N_i. Table 1 and Figure 9 show respectively the selection of the compaction points of interest and typical compaction-density curve.

Table 1 - Number of gyrations at the three point of interest for different combinations of traffic levels and maximum temperature (Cominsky et al., 1994)

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

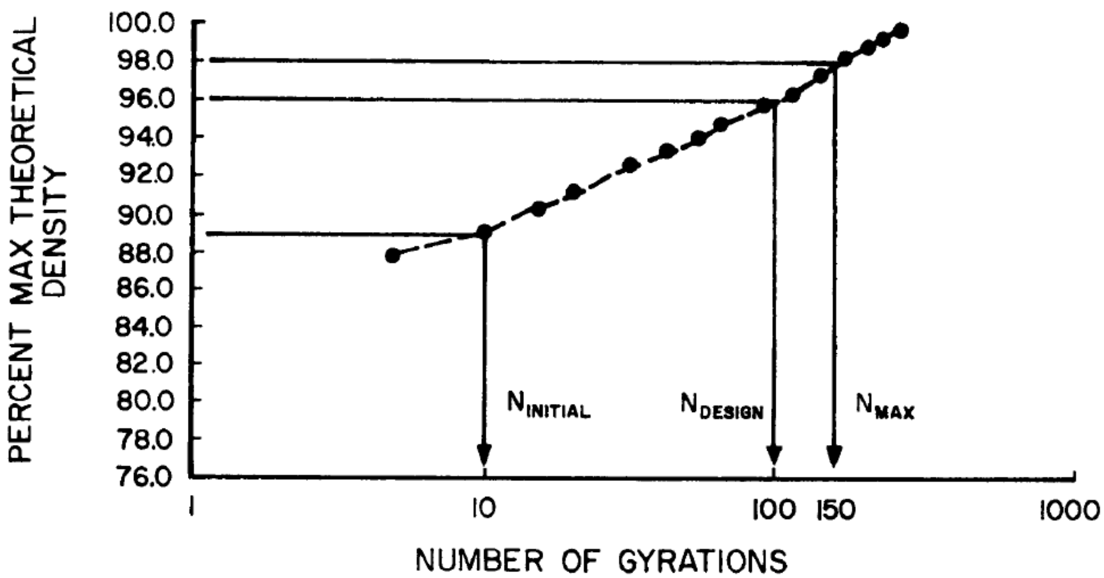


Figure 9 Typical Densification Curve (Cominsky et al., 1994)

Instead of air voids, the requirements could also be expressed in terms of required density as shown in Table 2.

Table 2 Compaction Requirements (Cominsky et al., 1994)

Compaction Level	Required Density (% of Theoretical Maximum Specific Gravity)
N_{init}	$C_{init} < 89$
N_{design}	$C_{design} = 96$
N_{max}	$C_{max} < 98$

In addition to the air voids content, Superpave defined acceptance intervals for the VMA and the VFA, as shown in Table 3 and Table 4.

Table 3 VMA Requirements (Cominsky et al., 1994)

Nominal Maximum Size	Minimum Voids in Mineral Aggregate (%)
9.5 mm	15.0
12.5 mm	14.0
19.0 mm	13.0
25.0 mm	12.0
37.5 mm	11.0
50.0 mm	10.5

Table 4 VFA Requirements (Cominsky et al., 1994)

Traffic Level (ESALs)	Design Voids Filled with Asphalt (%)
$< 3 \cdot 10^5$	70 – 80
$< 3 \cdot 10^6$	65 – 78
$< 1 \cdot 10^8$	65 – 75
$> 1 \cdot 10^8$	65 – 75

Another requirement was the ratio of dust to effective asphalt content. Dust was defined as the aggregate finer than the 0.075 mm sieve. An acceptable dust/asphalt ratio needed to be in the range 0.6 to 1.2 for all types of mixes. The final requirement for the Level 1 Superpave mix design was related to moisture susceptibility and the test used was described by AASHTO T 283: "Resistance of Compacted Bituminous Mixtures to Moisture Induced Damage". The test intended to measure the change of tensile strength due to the effects of water saturation and a freeze–thaw cycle. The results could indicate susceptibility of the asphalt mixture to stripping and the necessity of adding antistripping agents (AASHTO, 2014). The minimum tensile strength ratio (TSR) was set at 80 percent.

The volumetric design was intended for low traffic volumes, i.e. less than 10^6 design equivalent single axle loads (ESALs). For higher traffic volumes Level 2 and Level 3 were supposed to come into play, as shown by Table 5.

Table 5 Recommended Design Traffic For Level 1, 2, and 3 Mix Designs (Cominsky et al., 1994)

Design Level	Design Traffic (80 kN ESALs)
1 (low)	$\leq 10^6$
2 (intermediate)	$\leq 10^7$
3 (high)	$> 10^7$

Both Level 2 and Level 3 mix design procedures started with the selection of a design asphalt content in accordance with the Level 1 volumetric design procedure. The mixes were prepared at the optimal asphalt content, as well as a higher and a lower asphalt content. The three resulting mix designs were subjected to a series of performance-based tests. Then, the test results were used as an input for the Superpave software which would provide an estimate of the field pavement performance in terms of rutting, fatigue cracking, and low-temperature cracking. The final optimum asphalt content was determined from the software predictions. Two testing equipment were required by Level 2 and Level 3:

- Superpave Shear Tester (SST), shown in Figure 10
- Indirect Tensile Test Device (ITTD), shown in Figure 11

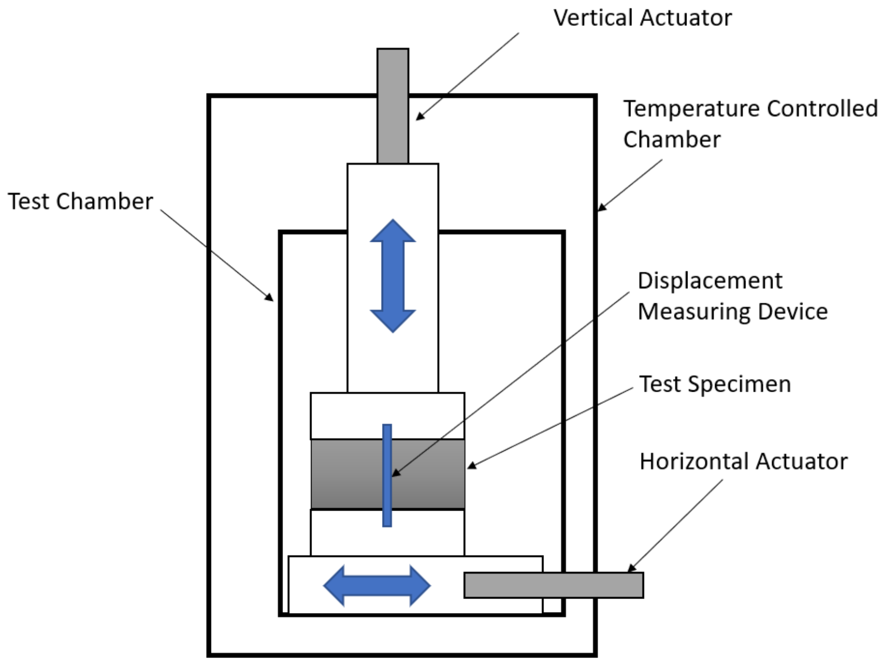


Figure 10 Superpave Shear Tester (Stuart & Mogawer, 2002)

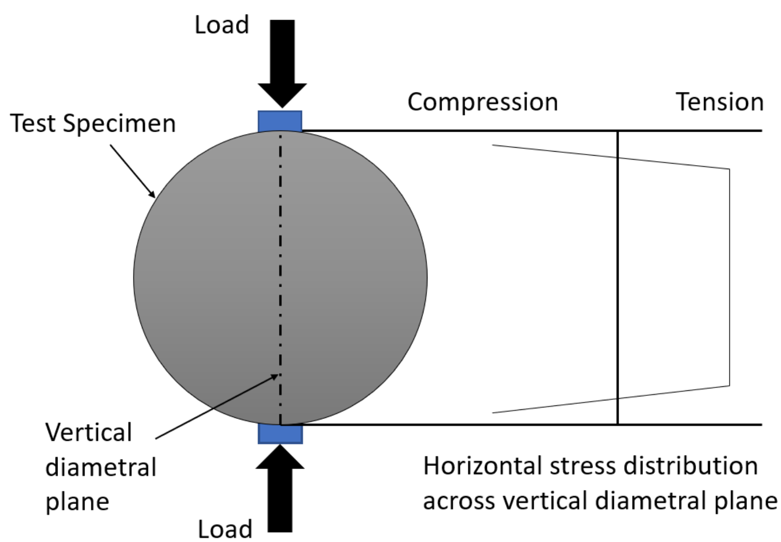


Figure 11 Indirect Tensile Test Device (Kennedy et al., 1994)

The SST was invented to simulate the high stresses that exist in the field on the pavement surface at the edges of the vehicle tires. The SST had the capability of applying on the specimens vertical and horizonal loads at the same time, to simulate both the compression and shear forces caused by the tires. The apparatus was composed by a load frame, vertical and horizontal actuators, temperature-controlled chamber, confining pressure control system, and a data acquisition system. The ITTD was a loading frame capable of applying a compressive load along the vertical diametral axis of a cylindrical specimen. As shown in Figure 11, the goal was to achieve a nearly uniform state of tensile stress across the specimen vertical section.

Overall, the tests required by Superpave Level 2 were the following (after each test, the corresponding necessary equipment is reported between parenthesis):

- Repeated shear test at constant stress ratio (SST)
- Frequency sweep at constant height (SST)
- Simple shear at constant height (SST)
- Indirect tensile creep (ITTD)
- Indirect tensile strength (ITTD)

The results obtained from the tests were then to be combined into the Superpave software with traffic, temperature, pavement structure, and subgrade support information. The software output consisted in a prediction of the field performance in terms of permanent deformation (rut depth, mm), fatigue cracking (area of fatigue cracking, %), and low-temperature cracking (spacing between cracks, m).

Level 3 required to perform all the Level 2 tests over a broader range of temperatures. Also, two additional tests were required in order to measure the nonlinear elastic behavior of the aggregate skeleton:

- Uniaxial strain (SST)
- Hydrostatic state of stress (SST)

Finally, if a higher degree of design reliability was required, supplemental proof testing could be conducted with the following tests: rolling wheel compaction, wheel-tracking device, flexural beam fatigue test.

2.1.4 Mix Design Evolution after Superpave

After Superpave's publication, it became clear that the level of complexity of Levels 2 and 3 was very high. Also, the SST apparatus was very expensive and required a long training of the operators. Therefore, most agencies and contractors adopted only the Level 1 volumetric mix design while the

SST was used primarily as a research equipment and was not considered a standard test in mix design and quality assurance testing.

The National Cooperative Highway Research Program (NCHRP) started in 1995 a new project (NCHRP 9-19 "Superpave Support and Performance Models Management") which aimed at developing simple performance tests to incorporate permanent deformation and fatigue cracking tests in the Superpave volumetric mix design method (Witczak et al., 2002). The laboratory tests underwent a field validation and specification between 2001 and 2005. The selected simple performance tests for asphalt mixes were dynamic modulus (E*), flow number (F_n), and flow time (F_t). Also, the research agency prepared a specification to determine a critical minimum E* value for HMA, which is based on project-specific information such as climate, traffic, and pavement structure. The specification was based on a series of pavement design examples solved using the mechanistic-empirical (M-E) pavement design guide developed by NCHRP Project 1-37A "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures: Phase II" (Witczak, 2005).

It must be noted that, soon after the implementation of Superpave specifications, most state agencies started making adjustments to the Superpave standards in order to improve the durability of their mixes. One of the main concerns was that the Superpave mixes were low in asphalt content, which resulted in poor durability especially with respect to cracking resistance and therefore had shorter service lives; by consequence, the Federal Highway Administration (FHWA) suggested that transportation agencies were to perform an independent evaluation in order to adjust the mix design process, for instance by modifying the number of design gyrations (FHWA, 2010). Also, with the development of new asphalt technologies, it was observed that Superpave could not adequately evaluate mix variables such as recycled materials, warm-mix additives, polymers, rejuvenators, and fibers. The main adjustments introduced to improve the original design method were the following:

- Lowering gyrations levels – Prowell and Brown (2007) conducted a study on the mixes' densification and, after collecting material from 40 field projects, recommended a reduction of the gyratory compaction levels in order to allow higher asphalt contents. The FHWA also suggested that transportation agencies perform an independent evaluation in order to modify the number of gyrations (FHWA, 2010).
- Changing design air voids and VMA limits – West et al. (2013) observed that lowering the target air void content allows more binder in the mix only if the VMA criteria are kept unchanged. However, VMA limitations are hard to validate because of the poor reproducibility of the aggregate bulk specific gravity (G_{sb}).

- Air voids regression approach – In this approach, mixes are first designed using the standard criteria that indicates a target air void content of 4.0%. The asphalt content is then selected through regression to achieve a target of 3.5% or 3.0% air voids. This approach typically allows achieving an increased binder content by 0.3 to 0.4% for a target of 3.0% air voids (West et al., 2018).

Also, binder modification, i.e. with polymers, has been shown as a valid way of improving the mix performances with respect to all kinds of distress. Polymer modification began in the 1980s and by 1997 was used by all but three states. However, Superpave specifications were designed for neat binders therefore they were inappropriate for polymer modified binders (Yildirim, 2007). The most popular polymers used to modify asphalt have been styrene–butadiene–styrene (SBS), styrene–butadiene rubber (SBR), and rubber.

- SBS – R Roque et al. (2004) evaluated the effect of SBS modification on the characteristics of Superpave mixes. The mixes were tested with Superpave IDT in terms of resilient modulus, creep, repeated load fracture, healing, and strength. The tests showed a reduced rate of micro-damage accumulation that allowed to achieve a better cracking resistance both in terms of crack initiation and propagation.
- SBR – Kim et al. (1999) tested SBR-modified mixes with respect to Hveem and Marshall stability, tensile strength, resilient modulus, and moisture susceptibility. The results showed how SBR provided improved resistance to rutting and cracking.
- Rubber – Rubber-modified mixes are dependent on rubber type, asphalt composition, size of rubber crumbs, and time and temperature of reaction (King et al., 1999). Generally, natural rubber improves rutting resistance and ductility but is sensitive to decomposition while tire rubber can improve both rutting and reflective cracking (Yildirim, 2007).

Part of the mixes’ unexpected behavior was also due to the introduction in the mixes of materials that originally were not considered in Superpave, such as:

- Recycled materials (i.e. RAP, RAS) – When adding recycled materials to an asphalt mix, there are multiple factors that will affect the long-term performance of the pavement, in particular: properties of RAP aggregates, properties of RAP binder, the interaction between virgin and RAP binder, and RAP inherent variability. Boriack et al. (2014) investigated HMA mixtures containing different amount of RAP and recommended that volumetric mix design should be supplemented by laboratory performance testing. The original Superpave method did not include recommendations for recycled mixes. Through two following NCHRP Projects, guidelines were provided, as summarized in Table 6.

Table 6 Summarized timeline of specifications evolution for recycled mixes

Year	Program	Highlights
1993	Strategic Highway Research Program (SHRP)	Superpave introduction: no provision on RAP use
2000	NCHRP Project 9-12: Incorporation of Reclaimed Asphalt Pavement in the Superpave System	Established a design method to allow RAP in Superpave mixes (McDaniel et al., 2000)
2012	NCHRP Project 9-46: Improved Mix Design, Evaluation, and Materials Management Practices for Hot Mix Asphalt with High Reclaimed Asphalt Pavement Content	Revised and improved practices for high-RAP content mixes (West et al., 2013)
2018	NCHRP Project 20-07/Task 406: Development of a Framework for Balanced Asphalt Mixture Design	Development of a framework for Balanced Mixture Design (BMD)

- Warm mix asphalt (WMA) – There are three main types of WMA technologies: asphalt foaming, organic additives, and chemical additives. Their use allows lowering the temperature of production and paving of asphalt mixes. By consequence, WMA enables to lower energy demand, reduce emissions, and increase allowable haul distances. During NHCRP Project 9-47A, “Properties and Performance of Warm Mix Asphalt Technologies”, West et al. (2014) compared performances of WMA pavements and corresponding regular HMA using both laboratory tests and field sections. Rutting tests (FN and HWTT) showed that WMA mixes were more susceptible to rutting while, with respect to the field performances, WMA and HMA performed similarly. Fatigue cracking tests (S-VECD) indicated an equal or better fatigue life for WMA mixes, while the field cracking was negligible for both.
- Recycling agents – There are two main kinds of recycling agents: softening agents and rejuvenators. Softening agents are only capable of lowering the viscosity of RAP binder, while rejuvenators are used to recover the properties of aged binders and reconstitute the chemical composition to ensure durability. Zaumanis and Mallick (2015) evaluated the effect of six different recycling agents on 100% recycled mixes and compared the performances with a virgin mix. Even if general trends were noticeable (e.g. improved rutting resistance, similar fatigue resistance) every agent impacted the performances in a different measure, highlighting the insufficiency of a purely volumetric design.

In order to overcome the limits of the initial mix design requirements and to continue exploiting the potential of non-conventional materials, state agencies adjusted the Superpave design specifications and implemented multiple changes over the years. Figure 12 shows the results of a survey conducted across the US regarding the most popular mix design modifications (R. West, 2018). The four most common measures included the use of softer binders for recycled mixes, changes in gyrations number, increasing the required voids in mineral aggregate (VMA), and the addition of performance tests.

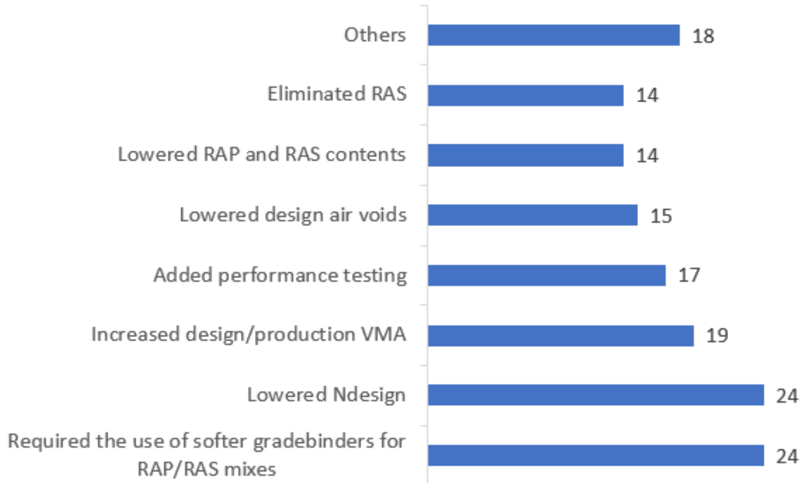


Figure 12 Survey on DOTs specification changes (R. West, 2018)

2.1.5 Balanced Mix Design

The need for a performance-based mix design stemmed from the absence of performance testing in the mix design systems commonly used across the US, which usually referred to the first level of Superpave (originally three levels of increasing complexity were conceived for mix design). Also, the increasing use of recycled materials and a growing number of new technologies changed the traditional way of designing mixes. Therefore, there was a need for a reliable method to properly characterize the mixes, allowing to supplement the traditional volumetric requirements while increasing the confidence in the final performances of the mixes. Multiple agencies and contractors started using laboratory tests to verify the resistance of the mixes to a variety of distresses. However, no common consensus was reached on which tests to use and how to set appropriate requirements. To this end, the ideal asphalt mix should behave appropriately with respect to the two main distress types, namely rutting and cracking. Hence the definition of "balanced" mix design (BMD), which should aim at designing mixes through the research of the best compromise between rutting resistance and cracking resistance. Zhou et al. (2006) first introduced the concept of BMD using the HWTT to evaluate rutting resistance and the Texas OT to evaluate cracking resistance. Subsequently, BMD was formally defined by the FHWA as an "asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate and location within the pavement structure”.

An easy way to report the performances of a given asphalt mix is through performance diagrams. Such illustrations allow showing the results in terms of performances with respect to two or more distresses. As an example, a performance diagram of rutting resistance (provided by an APA) and cracking resistance (provided by the CT Index cracking test) is shown in Figure 13.

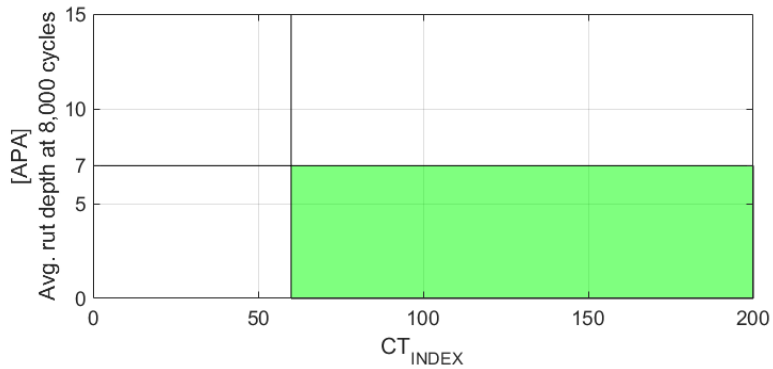


Figure 13 Example of performance diagram based on APA and CT index test

It is possible to observe how test results which belong to the highlighted green area imply that the respective mixes have appropriate resistance to both rutting and cracking, more specifically:

- Asphalt Pavement Analyzer to test rutting resistance (AASHTO T 340): maximum of 7 mm at 8,000 cycles
- CT Index to test cracking resistance: minimum value of 60

If three performance tests are included in the design procedure, it is possible to visualize the zone of acceptance through a 3D performance diagram. Figure 14 shows a performance-based mix design procedure which features the following requirements:

- Asphalt Pavement Analyzer to test rutting resistance (AASHTO T 340): maximum of 7 mm at 8,000 cycles
- Overlay Tester (OT) to test reflective cracking resistance (Texas DOT Tex-248-F): minimum of 300 cycles to failure (load reduction of 93%)
- Flexural Beam Fatigue to evaluate fatigue life (AASHTO T 321): minimum of 100,000 cycles

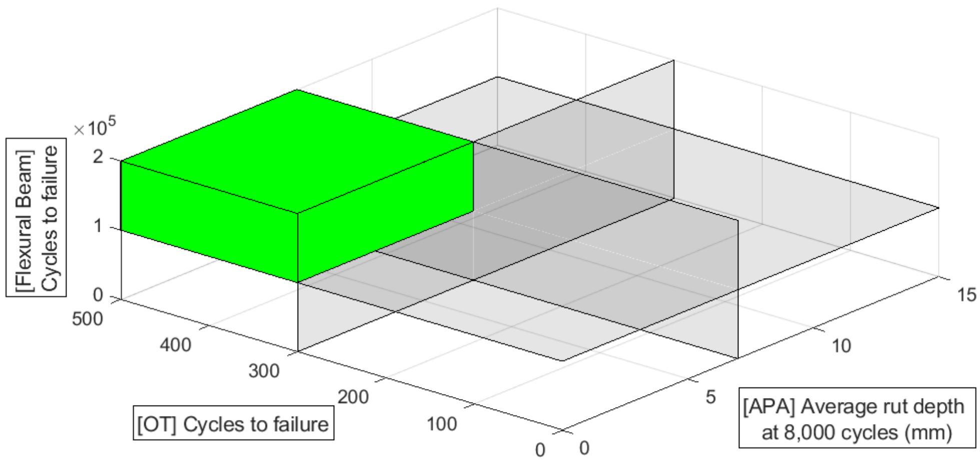


Figure 14 3D performance diagram

Today, many states implement performance tests in their design procedures.

<u>California Department of Transportation (Caltrans)</u>

Caltrans, which is the California DOT, uses a performance-based pavement design system (Table 7) which features specifications related to the use of:

- Superpave Shear Tester (SST) to determine the design binder content (AASHTO T 320) through the Repeated Shear Test at Constant Height (RSCH)
- Flexural Beam Fatigue to evaluate fatigue life (AASHTO T 321)
- Hamburg Wheel Tracking Test (HWTT) to evaluate the moisture damage resistance (AASHTO T 324)

Table 7 Caltrans performance-based requirements.

Design parameter	Test method	Requirement
Permanent deformation	Superpave Shear Tester (SST)	Minimum 360,000 repetitions
Fatigue	Repeated flexural bending	Minimum no. of repetitions variable with strain level
Moisture	Hamburg Wheel-Track Test (HWTT)	Minimum 20,000 repetitions

In addition to these tests, Caltrans uses CalME (Caltrans Mechanistic-Empirical Design Program) to support the mix design. Also, the California mix design method features the use of conventional Hveem requirements such as: air void content, aggregate specifications, voids in the mineral aggregate, voids filled with asphalt, dust proportion and tensile strength ratio (Harvey et al., 2014). Between the main challenges identified by Harvey et al. (2014) to develop the performance-based mix design system there are:

- Selection of baseline materials used to develop the specifications and procurement of representative materials, in particular RAP.
- Writing of specifications, with particular reference to reliability and requirements for each layer of the pavement.
- Relationship with expected distress modes.
- Communication and assistance to contractors.
- Differences between laboratory and plant-produced specimens.

Caltrans is aiming at implementing a modified version of the Superpave mix design procedure which, instead of the SST uses the repeated load triaxial (RLT) rutting test using the asphalt mixture performance tester (AMPT). A study conducted by He et al. (2016) found that the unconfined RLT test could be used as a substitute for the repeated shear test for mix design and quality assurance, while either the confined or unconfined RLT could be used for ME characterization.

New Jersey Department of Transportation (NJDOT)

The New Jersey DOT established a mix design procedure which features at least one of the following three laboratory performance tests:

- Asphalt Pavement Analyzer to test rutting resistance (AASHTO T 340)
- Flexural Beam Fatigue to evaluate fatigue life (AASHTO T 321)
- Overlay Tester (OT) to test cracking resistance (Texas DOT Tex-248-F)

In most cases, both rutting and cracking are tested; the choice of the fatigue test depends on the type of cracking expected for the specific application. Currently, the New Jersey DOT requires the abovementioned procedure only for five types of mix, including mixes with high binder content and a mix with high RAP content (Bennert et al., 2014). Each performance-based kind of mix has different requirements related to the laboratory test output. The reason for different thresholds resides in the acknowledgment that different mixes need different requirements in relation to the application or need of the specific project. Along with performance requirements, adjustments were made to the conventional volumetric requirements.

Performance-based mixes in New Jersey need to be tested in multiple phases of the paving process: design, test strip, and project construction. After more than ten years since their first implementation, this kind of mixes showed excellent performance in the field and oftentimes outperformed conventional mixtures. Bennert et al. (2014) reported that at first asphalt contractors were skeptical towards this new mix design method, but eventually came to embrace it and also the design methodology became more efficient. The New Jersey DOT has an approach which highlights how each type of sub-structure (e.g. bridge deck) requires overlays of different asphalt mixture type (e.g. different asphalt content, different volumetric requirements, and different performance levels). As an example, Table 8 reports the performance requirements in terms of rutting and cracking resistance for mixes with high RAP content.

Table 8 High-RAP mix performance requirements (Bennert et al., 2014)

	Surface course		**Intermediate course**	
	PG 64-22	**PG 76-22**	**PG 64-22**	**PG 76-22**
APA @ 8,000 cycles	< 7 mm	< 4 mm	< 7 mm	< 4 mm
Overlay tester	> 150 cycles	> 175 cycles	> 100 cycles	> 125 cycles

Texas Department of Transportation (TxDOT)

Texas DOT features the use of a balanced mix design procedure with the following laboratory tests:

- Hamburg Wheel-Track Test (HWTT) to evaluate the rutting resistance (AASHTO T 324)
- Overlay Tester (OT) to test cracking resistance (Texas DOT Tex-248-F)

Zhou et al. (2014) reported that the most common form of rehabilitation in Texas is the placement of an asphalt overlay. However, when trying to improve the rutting resistance of the mixes, the pavement designers had to face negative impacts on the cracking resistance. Also, with an increasing use of recycled materials, the mixes were likely to become more susceptible to cracking. Therefore, the Texas DOT started moving towards a mix design supported by performance tests. The authors report that these tests should be characterized by project-specific requirements and consider the climatic zone of the project.

Louisiana Department of Transportation and Development (LaDOTD)

Cooper III et al. (2014) studied the feasibility of modifying the mix design specifications for the Louisiana Department of Transportation and Development. The balanced mix design procedure featured the use of the Hamburg Wheel-Track Test (HWTT) and semicircular bending (SCB) test. Through this study it was possible to conclude that volumetric mix design can be successfully complemented by performance tests; in particular, it was possible to highlight the performance differences between polymer modified and regular binders.

<u>Other states</u>

Mogawer et al. (2014) studied the development of a performance-based mix design procedure for high-performance thin overlays. Mixtures from different projects (Minnesota, New Hampshire, and Vermont) were tested with the following experiments:

- Overlay Tester (OT) to test cracking resistance (Texas DOT Tex-248-F)
- Asphalt Pavement Analyzer to test rutting resistance (AASHTO T 340)
- Thermal Stress Restrained Specimen Test (TSRST) to evaluate thermal cracking resistance
- Flexural Beam Fatigue to evaluate fatigue life (AASHTO T 321)

The authors provided pilot specifications, which are reported in Table 9. Additional tests featured the HWTT for rutting and SCB test for evaluating cracking resistance.

Table 9 Pilot specifications for laboratory mixture performance criteria

Design parameter	Test method	Requirement
Thermal cracking	TSRST	±6°C from the low-temperature PG of the binder
Cracking	OT	≥ 300 cycles
Fatigue Life	Flexural beam	≥ 100,000 cycles
Rutting	APA	≤ 4 mm at 8,000 cycles

2.2 PAVEMENT RECYCLING

2.2.1 History

Even though the first reports of asphalt pavement recycling date back to 1915, the practice became more popular in the 1970s (Epps, 1990). Two main factors supported the diffusion of pavement recycling. First, the 1973 Arab Oil Embargo which resulted in a sharp increase of the asphalt binder cost. Figure 15 shows the evolution over time of the West Texas Intermediate (WTI) crude oil price per barrel. Secondly, the introduction in 1975 of large scale cold planing equipment which, with the implementation of easily replaceable tungsten carbide milling tools, provided a practical way to grind the pavements (ARRA, 2001). At that time, contractors and agencies started examining mixes with high reclaimed asphalt pavement (RAP) contents (even up to 80%), but the equipment of the time was not adequate to produce such high RAP mixes.

As oil prices went back to normal, the need for designing highly recycled mixes became less urgent and the average RAP content decreased to 20%. This trend lasted through the period during which Superpave was developed and released. In the 2000s a new increase in the price of crude oil sparked new interest in increasing the amount of RAP in the mixes and this interest continues today (Newcomb et al., 2016). Over time, agencies became increasingly aware of the benefits of recycling besides the economic savings. Pavement recycling turned out to be a very valuable tool for many other reasons, which contributed to building a sustainable transportation infrastructure. Besides reducing the initial cost of asphalt mixes, it has been demonstrated that the use of RAP helps saving natural resources, reducing landfill, and lowering emissions. Such benefits are crucial to the environmental concerns of today, and many countries all over the world require by law the use of certain minimum percentages of recycled pavement materials in the production of asphalt mixes.

Figure 15 WTI crude oil price per barrel adjusted for inflation (Macrotrends, 2020)

2.2.2 Pavement Recycling Benefits

Reclaimed asphalt pavement (RAP) is commonly obtained through the operations of resurfacing, rehabilitation, and reconstruction of roads. This material contains asphalt binder and aggregates and can be considered as a substitute for aggregates and virgin asphalt binder. Also, RAP can be used in granular bases, subbases, and embankments.

In 2019, the estimated quantity of RAP used in asphalt mixtures was 89.2 million tons. This amount corresponds to the saving of 4.5 million tons (24 million barrels) of asphalt binder and more than 84 million tons of virgin aggregate; the average content of RAP contained in asphalt mixtures increased from 15.6 percent in 2009 to 21.1 percent in 2019 (Williams et al., 2020). Through the replacement of the expensive virgin aggregates and binder it is possible to achieve significant economic savings in terms of materials cost. This cost category, represents about 70 percent of the cost to produce asphalt mixes, given that the other categories are production, trucking, and paving (Copeland, 2011).

With respect to the environmental performance, the impact of pavement recycling can also be seen in terms of energy and greenhouse gas (GHG) savings. Aurangzeb et al. (2014) reported that reductions between 17% and 28% were observed in both energy consumption and GHG emissions for pavements with 30%, 40%, and 50% RAP in the binder course. This lower environmental and energy impact was associated with the reduction of asphalt binder needed to be extracted from crude oil and the ensuing reduction in processing, transportation, blending, and storage. By contrast, the contribution of the construction phase to the reduction of GHGs and energy was found to be minimal. With respect to feedstock energy, a reduction of 26%, 33%, and 40% was observed for the mixtures with 30%, 40%, and 50% RAP, respectively. Yang et al. (2015) reported that, compared to mixes produced using virgin materials, recycled mixes showed consistent trends of reductions (up to approximately 20%) in energy and global warming potential (GWP) as the RAP content increased (up to 42%).

2.2.3 Design of Recycled Mixes

On the subject of mix design and more specifically with respect to the inclusion of RAP in hot mixes, the FHWA formed a Superpave Mixtures Expert Task Group in 1997 to develop interim guidelines. These guidelines provided the first distinction between “black rock” behavior (i.e. the aged binder does not combine with the virgin binder in a significant way and does not change the binder properties) and more complex interactive behavior. The overall mix performance was found to be dependent on the percentage of RAP incorporated in the mix and the condition of the old binder included in the RAP could determine a reduction in the needed quantity of virgin binder.

The inclusion of RAP in hot mix asphalt was formalized through the NCHRP Project 9-12 “Incorporation of Reclaimed Asphalt Pavement in the Superpave System” in which the authors reported the results of three main studies (McDaniel et al., 2000):

- Black rock study
- Binder effect study
- Mixture effect study

The first study aimed at defining the role of RAP in an asphalt mix discussing the blending level between the old RAP binder and the virgin binder. Two possible extreme cases were defined: “black rock” and “total blending”. These results showed that neither RAP acted like a black rock nor it was reasonable to think that total blending of the RAP binder and virgin binder occurred. Partial blending appeared to be the most appropriate behavior. Therefore, at high RAP contents, the old hardened RAP binder had to be considered in the selection of the virgin binder. In this regard, blending charts could be used for determining the virgin binder grade or the maximum amount of RAP that can be included in the mix. In order to develop the blending charts the RAP binder needs to be extracted and the suggested extraction-and-recovery method featured either toluene and ethanol or an n-propyl bromide solvent.

The second study was focused on the effect of the hardened RAP binder on the blended binder properties. The research team recommended to use the dynamic shear rheometer (DSR) and rolling thin film oven (RTFO) to test the recovered RAP binder and be able to use the blending charts. Based on the RAP binder stiffness a three-tier system could be defined. At low RAP contents, the effect of the RAP binder could be considered negligible. For intermediate RAP contents the use of a softer virgin binder was recommended. For high RAP content a blending chart should be used.

In the third study, shear tests, indirect tensile tests, and beam fatigue testing were conducted to define the impacts of RAP on the mix performance. At high RAP contents, RAP binder had a stiffening effect on the mix and caused a decrease in shear deformation. Therefore, high RAP contents would result in a better resistance to rutting while lowering the fatigue life and low-temperature cracking resistance. These results supported the idea of using a softer virgin binder with high RAP contents in order to compensate the increased mix stiffness.

An important accomplishment of NCHRP Project 9-12 was the development of the current mix design recommendations included in AASHTO M 323 specification "Superpave Volumetric Mix Design". The design of hot recycled mixes was dependent on the RAP binder grade and the RAP content, as shown in Table 10.

Table 10 Binder selection guidelines for RAP mixtures provided by NCHRP Project 9-12 (McDaniel et al., 2000)

Recommended Virgin Asphalt Binder Grade	**RAP Percentage**		
	Recovered RAP Grade		
	PG xx-22 or lower	***PG xx-16***	***PG xx-10 or higher***
No change in binder selection	< 20%	< 15%	< 10%
Select virgin binder one grade softer than normal (i.e., select a PG 58-28 if a PG 64-22 would normally be used)	20 – 30%	15 – 25%	10 – 15%
Follow recommendations from blending charts	> 30%	> 25%	> 15%

The mix design guidelines for hot recycled mixes were further revised and improved through a following study, included in the NCHRP Project 9-46 "Improved Mix Design, Evaluation, and Materials Management Practices for Hot Mix Asphalt with High Reclaimed Asphalt Pavement

Content". The objectives of the research were to develop a mix design procedure that would allow achieving satisfactory long-term performances of high-RAP mixes and propose changes to the AASHTO standards, R 35 and M 323 accordingly (West et al., 2013). The research team reiterated the effects of RAP on the mix performances: increased dynamic modulus, increased rutting resistance (measured through the flow number test), and lower fracture energy. In order to improve the long-term cracking resistance, care needed to be taken in the selection of the performance grade of the virgin binder.

With this study, the RAP Binder Ratio (RAPBR) concept was introduced. RAPBR allowed distinguishing mixes containing RAP by the proportion of RAP binder to the total binder (instead of RAP content described as the percentage by weight of the RAP aggregate in the total aggregate blend). The high-RAP content mixes were classified as the ones with RAPBR higher than 0.25 as shown in Table 11.

Table 11 Binder selection guidelines for RAP mixtures provided by NCHRP Project 9-46 (AASHTO, 2017)

Recommended Virgin Asphalt Binder Grade	RAPBR
No change in binder selection	< 0.25
Follow recommendations from Appendix X2	> 0.25

The research team recommended the use of moisture susceptibility test for recycled mixes, regardless of the RAP content. In the case of high RAPBR, rutting tests were considered unnecessary, unless those mixes featured the use of a softer grade virgin binder or rejuvenators. Based on the climate of the project thermal cracking tests would also be required. Load-related cracking tests were recommended to provide supplemental information, but not to define acceptance or rejection of the mixes because of the lack of a well-established relationship between the property and the field performance. Finally, the need for a fatigue test was highlighted and suggested as a topic for further research (West et al., 2013).

One key takeaway from the NCHRP Project 9-46 was that volumetric properties alone were not be able to adequately characterize the suitability of a mixture. Recycled mixtures that met the standard volumetric criteria, showed significant differences in terms of laboratory performance tests. Therefore, one challenge the asphalt industry needed to face was the identification of practical and reliable laboratory performance tests that could correlate to field performance (Willis & West, 2014).

2.2.4 Challenges and Impact on Performances

Several studies have been conducted to investigate the effect of RAP on the performance of asphalt mixes. Traditionally, the RAP inclusion has been linked to an improve of the rutting resistance and a decline of the cracking resistance. When including high contents of RAP, proper adjustments of the asphalt mix designs, such as the use of a softer grade virgin asphalt binder, were recommended to prevent reduced fatigue life (Yang et al., 2015). However, due to the variety of mix types and the inherent variability of the RAP there isn't a clear and definitive set of rules on how the recycled material is going to affect the final mix performances. Chehab and Daniel (2006) underlined the impossibility of making a universal statement on how increasing RAP affects the performance. Generally, one effect is the increase of the dynamic modulus due to the effect of the stiff RAP binder. In order to determine the impact of the RAP on the mix stiffness, it is important to first determine the amount of blending that occurs between the RAP and virgin binder. In fact, the assumption that 100% of the RAP binder is working could result in under-asphalted mixtures; on the contrary, assuming that the recycled pavement material only acts as a "black rock" would have the opposite outcome (i.e., over-asphalted mixtures). The inability to accurately characterize the binder properties prevents the use of high percentages of RAP (Al-Qadi et al., 2009).

Over time, conflicting results and outcomes contrary to the expectations have been observed. A summary of the main findings from different studies is reported in Table 12.

- Huang et al. (2004) conducted a laboratory study and reported that up to 20% RAP improved the tensile strength and fracture resistance, while 30% RAP changed the performance significantly.
- Tabakovic et al. (2006) found that an increase in RAP content in the mix corresponds to an increase in stiffness, an improved fatigue life, and a decrease in moisture damage resistance.
- Kim et al. (2007) reported that mixes with more RAP showed higher rutting resistance and lower fracture energy. However, for high-RAP mixes, the use of softer binders could reduce the rutting performance.
- Widyatmoko (2008) indicated that mixtures containing RAP tend to have lower stiffness, lower resistance to permanent deformation, and better resistance to fatigue than equivalent mixtures without RAP. This behavior would be explained by the use of softer binders and rejuvenating agents.

- Tabaković et al. (2010) tested a series of binder course mixes designed with varying percentages of RAP. The authors found that the RAP addition provided an improvement in many mechanical properties. In particular, the fatigue resistance was improved, and the stiffness was increased. Instead, the water resistance deteriorated.
- Apeagyei et al. (2011) reported similar rutting performances at the lower (0%) and higher (25%) RAP contents, while the best performances corresponded to mixtures that contained intermediate amounts of RAP (10% and 15%).
- Al-Qadi et al. (2012) reported that, as RAP content increased, complex modulus, tensile strength, moisture damage resistance, fatigue life, and rutting resistance increased. On the contrary, the thermal cracking resistance decreased.
- McDaniel et al. (2012) found that RAP increased the stiffness of the mixes while improving the fatigue life and not affecting the thermal cracking resistance.
- Izaks et al. (2015) observed how high RAP content mixtures had a higher resistance to rutting when compared to reference traditional mixes. The recycled mixes exhibited similar mechanical properties, such as resistance to fatigue and stiffness.

Table 12 Impact of RAP addition to asphalt mixes

Study	Modulus	Rutting resistance	Fatigue life	Moisture damage resistance	Cracking resistance
Huang et al. (2004)	/	Improved	/	/	Improved
Tabakovic et al. (2006)	Increased	/	Improved	Worsened	/
Kim et al. (2007)	/	Improved	/	/	Worsened
Widyatmoko (2008)	Lowered	Worsened	Improved	No effect	/
Tabaković et al. (2010)	Increased	/	Improved	Worsened	/
Apeagyei et al. (2011)	/	Improved	/	/	/
Al-Qadi et al. (2012)	Increased	Improved	Slightly improved	Improved	Worsened (thermal)
McDaniel et al. (2012)	Increased	/	Improved	/	No effect
Izaks et al. (2015)	No effect	Improved	No effect	/	/

It is possible to observe that there is no final agreement between researchers with respect to the impact of RAP on rutting resistance, cracking resistance, moisture susceptibility, and fatigue life. Many factors contribute to the properties of an asphalt mix and by consequence do not allow making generalized conclusions on the RAP effects. The main elements that have a distinct impact on a mixture's final performances are the following:

- Quality of the virgin bitumen and the properties of the virgin aggregates.
- RAP inherent variability.

- Uncertain role of the recycled binder in the new mix; the recycled binder is aged, oxidized, and stiffer than the virgin one.
- Degree of blending between recycled and virgin binder.
- Use of additives and rejuvenators.

For this reason, performance-based mix design should be preferable to the traditional volumetric one. Proper testing, performed on project-specific materials and mixes, can provide a better prediction and understanding of the recycled mixes and by consequence support in a more effective way the decisions which transportation agencies have to make when trying to adopt more sustainable practices.

2.2.5 Current Use and Limitations

Today, more than 99 percent of the recovered RAP is being reused. Of this amount of available RAP, over 97 percent is included in mixes to use in new pavements while the remaining 3 percent is being used in other civil engineering applications, like unbound aggregate bases. In 2018, the total amount of RAP used in asphalt mixtures was estimated at 82.2 million tons, which represented a 46.8 percent increase from 2009 (Williams et al., 2019).

Over time, an average between 15 and 20 percent RAP content has become a standard practice for the production of asphalt mixtures worldwide (Tarsi et al., 2020). In some countries, like the Netherlands and Japan, these recycling levels have been largely exceeded. In the Netherlands, it is common practice to include 50% RAP in most asphalt mixtures, for both base and surface layers (Mohajeri, 2015). Most significantly, in 2008 a new specification system for asphalt mixtures was implemented, in line with the European standards (EN13108 series). In this new scenario, the Dutch contractors had the freedom to select their own mix composition and design procedure as long as the final performance requirements were fulfilled. In 2013, Japan reported an average of 47 percent RAP content (West & Copeland, 2015). The main measures taken by the Japanese industry included RAP fractionation, moisture control, and use of rejuvenators. In addition, specifications were developed with respect to testing both directly on the RAP and the mix at the design stage.

It is clear how one of the keys to increase the recycling levels with confidence has been the use of performance tests on the mixes. Figure 16 shows the responses of state asphalt pavement associations (SAPAs) to a survey on the best ways for increasing the use of recycled materials. The top solution was indeed the use of mixture performance testing and balanced mix design (29% of respondents), followed by increasing the recycled material content in lower pavement layers (23%) and the use of RAP fractionating (18%). Updates of the design specification, improved quality control of recycled materials, and binder grade bumping were the other responses (Williams et al., 2019).

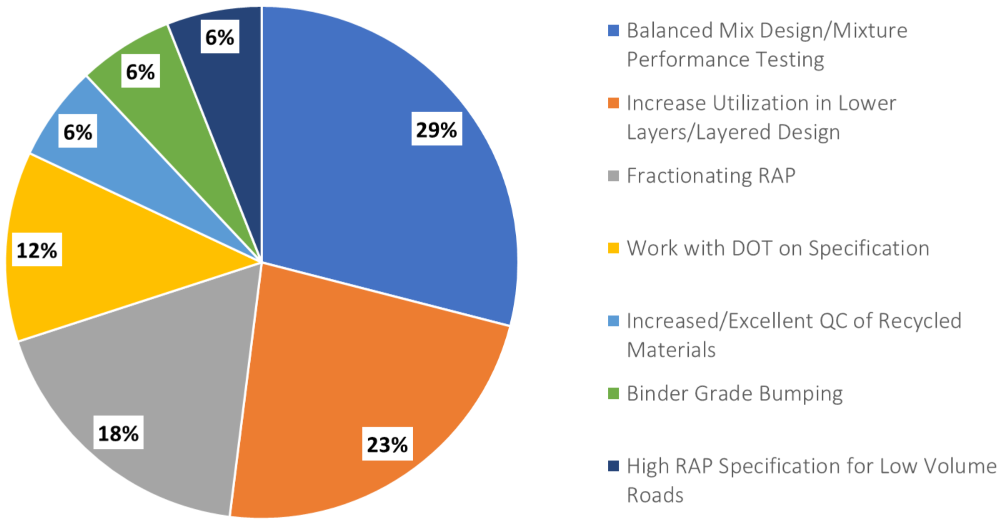


Figure 16 Reported possible means for increasing recycled materials use (Williams et al., 2019)

To support the mix design and quality acceptance processes, many transportation agencies have adopted a very wide selection of tests varying in complexity, cost, testing time, and operator's training required. Many American states like Texas (Zhou et al., 2014), California (Harvey et al., 2014), and New Jersey (Bennert et al., 2014) have already developed performance-based mix design systems. At the federal level, the NCHRP Project 20-07/Task 406 aimed at developing a framework that evaluated various approaches to incorporate performance testing and criteria at the design stage and implement balanced mix design procedures (R. West, 2018).

Also, the gradation of RAP needs to be studied. RAP fractionation is the process of separating the reclaimed material in different sizes (at least two) in order to have control over the consistency of the mix gradation. For RAP ripped off pavements, the gradation shows little or no degradation, while for RAP recovered from milling, the gradation becomes finer than the original aggregate (Kim et al., 2007). Across the US, 24 percent of the RAP used is fractionated (Williams et al., 2019).
The quality of the RAP materials needs to be checked over time. In fact, once a pavement is reclaimed, RAP aging continues during the stockpiling process due to exposure to air and oxidation (McMillian and Palsat 1985).

2.2.6 Rejuvenators

There are three main physical processes that happen during the asphalt aging process (Petersen, 1984):

- Loss of the oily components of asphalt because of volatility or absorption by porous aggregates
- Changes in the chemical composition of asphalt molecules from reaction with atmospheric oxygen
- Molecular structuring that produces thixotropic effects (steric hardening)

According to Petersen (1984) the second mechanism is the main contributor to the pavement hardening. As the pavement ages, the maltene phase transforms into the asphaltene phase leading to increased viscosity and decreased ductility (Corbett, 1975). In order to compensate the properties of the aged RAP binder and achieve satisfying pavement performance, the recycled material can be mixed with soft virgin binders or recycling agents, which differentiate between rejuvenators and softening agents. Rejuvenators are chemical agents capable of restoring the physical and chemical properties of the old binder. They differ from the softening agents that lower the viscosity of the aged binder (Roberts et al., 1991). Rejuvenators, such as lubricating and extender oils, contain a high proportion of maltene constituents. Instead, softening agents include asphalt flux oil, lube stock, and slurry oil.
The main benefits of using rejuvenating agents instead of soft binders are (Zaumanis et al., 2014):

- Cheap storage, since usually heating is not required
- Simple addition to the mixture using pump or liquid additive dosage system
- Precise addition based on the RAP binder properties and level of aging
- Potential ability to dose directly on RAP
- RAP contents from 0 to 100 % with the same product
- Often lower costs

The main concerns are related to the possible incomplete diffusion into the binder film. Carpenter and Wolosick (1980) described the rejuvenation as a two-stage process. The process starts with the formation of a low viscosity layer the aggregates coated with the aged binder. Then the rejuvenator penetrates the aged binder layer and slowly softens the old asphalt binder. The inability to activate the aged binder homogenously could cause (Zaumanis et al., 2014):

- Plastic deformations early in pavement life due to the increased dose of recycling agent on outer layer of binder film
- “Black rock” behavior of RAP which would increase the risk of cracking failures

Today, a lot of uncertainties remain over the properties of the rejuvenated RAP binder, therefore RAP is often used in lower level applications without fully exploiting the value of the asphalt binder available in the RAP (Zaumanis & Mallick, 2015). Haghshenas et al. (2016) studied the effects of three different rejuvenators and observed some common effects: increase of the rutting potential, increase of the mixture ductility, improvement of the cracking resistance, and increase of moisture susceptibility. However, differences were still significant between the different kinds of recycling agent. Therefore, it is important to select the right rejuvenating agent, based on its compatibility with the aged binder (Al-Qadi et al., 2007). Typically, rejuvenating agents with low saturate content and high aromatic content were found to be usually compatible with aged binder (Dunning & Mendenhall, 1978).

2.3 EVOLUTION OF PERFORMANCE TESTING

The first example of performance testing for asphalt mixes date back to 1920s with the Hubbard-Field Stability test which was intended to investigate the resistance of the mix to the punching stresses applied on the roads by the vehicles' tires. Then, in the 1930s the Hveem Stabilometer test was introduced and the analysis, through a pseudo-triaxial test, was intended to discern between stable and unstable pavement materials. The Stabilometer measured the resistance to deformation of a compacted HMA sample by measuring the lateral pressure developed from applying a vertical load. Approximately in the same years, the Marshall method was introduced and featured two parameters, obtained through a compression test, to analyze the mixes: stability and flow. After years from their first implementation it was possible to observe how differently the mixes designed with Hveem and Marshall method performed. Hveem mixes showed a tendency towards cracking issues, while the Marshall mixes had more rutting problems (McGennis et al., 1995).

In order to evaluate the resistance to both the distresses, the Superpave method, introduced in the 1990s, included a series of performance tests to be performed on the mixes at the design stage. Through the Superpave system it was possible to obtain thorough information on the mix performance in terms of rutting, fatigue cracking, and low-temperature cracking. This high level of information came at the cost of low practicality and high economic commitment (the Superpave Shear Tester reportedly had a price of $300,000), therefore Superpave testing did not become popular in routine operations of mix design (Button, 2004). Superpave mixes, which in order to provide durability were designed according to volumetric properties, were found to be susceptible to early cracking problems (Watson, 2003).

To support the Superpave volumetric mix design method, simple performance tests were introduced in the 2000s. The Simple Performance Test (SPT) machine was a relatively simple and low-cost (between \$50,000 and \$80,000 in 2005) equipment to characterize the HMA (Brown, 2007). The SPT machine had the capability of computing dynamic modulus, flow number, and flow time which were parameters suitable to describe rutting and cracking resistance through a performance prediction platform (FHWA, 2016). During the NCHRP Project 09-29 “Simple Performance Tester for Superpave Mix Design” the FHWA assumed responsibility for the development of the SPT machine and the Asphalt Mixture Performance Tester (AMPT) was developed (Bonaquist, 2011). The AMPT details are covered in section 1.1.6.

Over the years, alongside the development of the SPT machine and the AMPT, multiple additional laboratory test methods have been designed to study the performance of asphalt mixes in relation to specific pavement distresses. Particular emphasis has been given to the development of cracking and rutting tests, which are covered in sections 1.1.7 and 1.1.8 respectively.

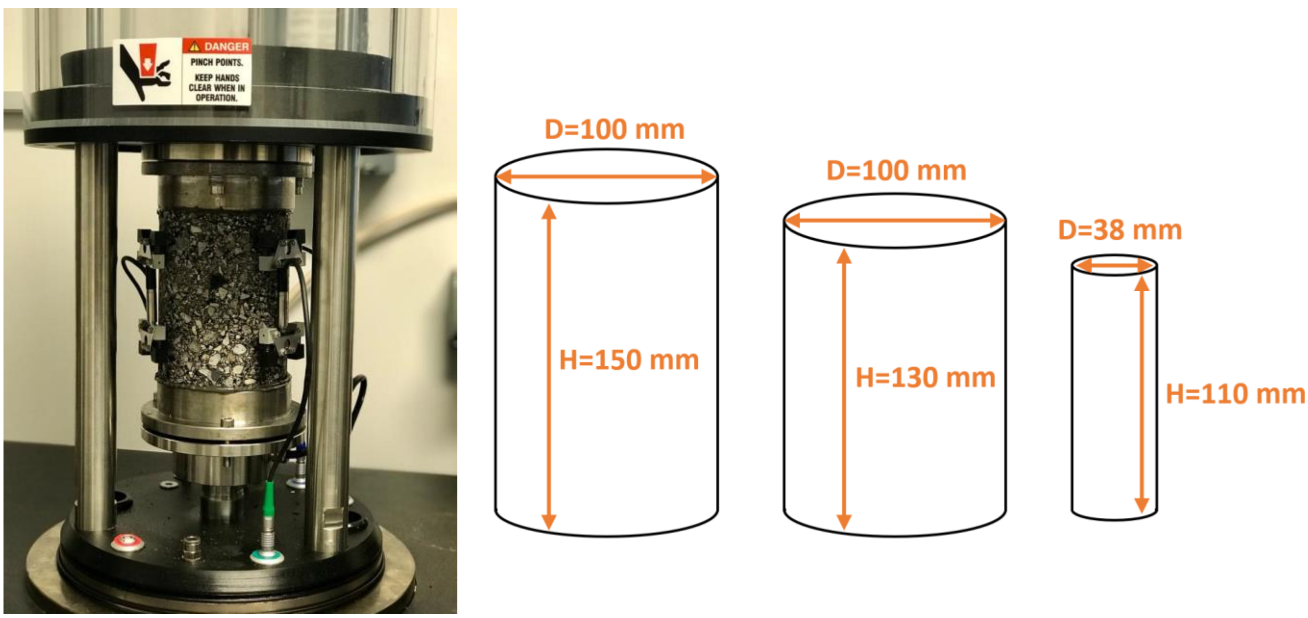
2.3.1 Asphalt Mixture Performance Tester (AMPT)

The AMPT, shown in Figure 17, is a servo-hydraulic testing system equipped with a computer controller. It features a temperature-controlled chamber capable of cooling or heating a specimen between 4°C and 60°C (FHWA, 2016). The AMPT is capable of performing several performance tests, including dynamic modulus, flow number (FN), stress sweep rutting (SSR), and S-VECD fatigue test. The typical AMPT specimens are cut and cored from laboratory compacted specimens and have a diameter of 100 mm and a height of either 150 mm or 130 mm. In order to test specimens extracted from the field, specimens of smaller sizes, 38 mm diameter by 110 mm height, can be tested. The two different samples sizes are visible in Figure 18. The following AASHTO methods have been published to standardize the testing procedures:

- AASHTO R 83, specimens’ preparation
- AASHTO T 378, dynamic modulus and flow number
- AASHTO TP 107, S-VECD fatigue test
- AASHTO TP 134, stress sweep rutting

The AMPT has been promoted for the ability to use the same tests for both mixture evaluation and structural design and the FHWA has adopted the AMPT in the development of performance-related specifications. The test results constitute the primary inputs for pavement design software such as FlexMAT™, FlexPAVE™, and AASHTOWare® Pavement ME Design.

Figure 17 Asphalt Mixture Performance Tester



(a) **(b)**

Figure 18 (a) AMPT specimen instrumentation, (b) AMPT specimen sizes

2.3.1.1 Dynamic Modulus

The dynamic modulus test is performed through the application of a controlled sinusoidal (haversine) compressive stress at different temperatures and frequencies. The applied stresses and axial strains are measured as a function of time to calculate dynamic modulus and phase angle. The dynamic modulus represents the stiffness of the mix at given temperature and loading frequency. The phase angle shows the extent of viscous and elastic behavior of the material.

2.3.1.2 Flow Number

The flow number (FN) test features a repeated haversine axial compressive load pulse of 0.1 second every 1.0 second. Throughout the test, which is conducted at constant temperature, the permanent axial strains are measured as a function of the load cycles. FN is defined as the number of load cycles corresponding to the minimum rate of change of permanent axial strain, which coincides with the start of the tertiary region, as shown in Figure 19. In other words, FN indicates the onset of shear deformation in asphalt mixtures which constitutes a meaningful indicator of the rutting resistance. In accordance with AASHTO T 378, the FN test can be performed in both confined and unconfined conditions.

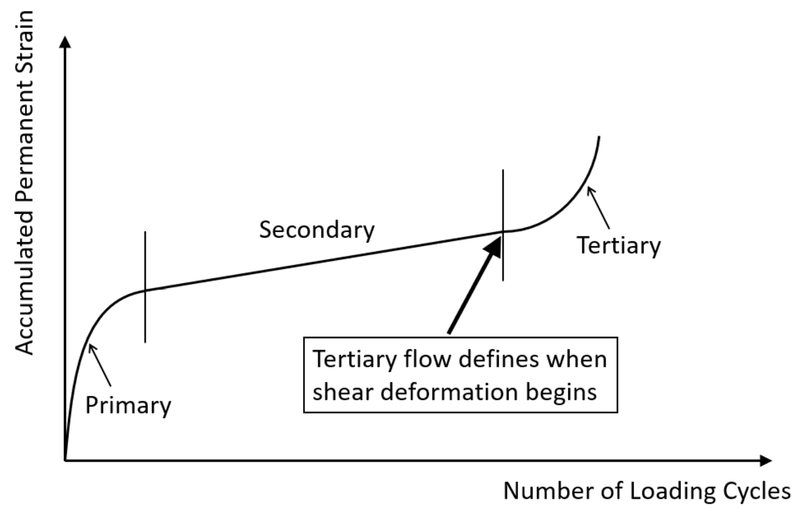


Figure 19 Typical test results from static creep and repeated load tests, such as the flow number test (Kaloush et al., 2003)

2.3.1.3 Stress Sweep Rutting

The stress sweep rutting (SSR) test follows a similar procedure to the FN test, but takes into account the effects of temperature, loading time, and deviatoric stress on the rutting resistance. In contrast to the FN test, which can represent the resistance to permanent deformation only in terms of ranking at a certain testing condition, SSR generates a rutting master curve which describes the distress potential at various temperature and traffic combinations (Kim & Kim, 2017).

SSR tests are performed on four test specimens at two temperatures: high (T_H) and low (T_L). The selection of appropriate temperatures is defined in AASHTO TP 134: T_H is calculated from the degree-days parameter obtained using LTPPBind Version 3.1, while T_L is selected based on the PG grade. In both cases, the specimens are subjected to three 200-cycle loading blocks of three deviatoric stress levels and constant confining pressure of 69 kPa.

The main SSR output is the shift model for rutting, which is composed of one permanent strain master curve and two shift functions. The shift model has been incorporated into the pavement performance prediction program, FlexPAVE™ (AASHTO, 2019).

Also, based on the SSR test outcome, the Rutting Strain Index (RSI) was developed. Since the field performance is a consequence of rutting across all layers of the pavement system, three different reference structures have been defined to study all kinds of asphalt mixtures: surface, intermediate, and base layers (Ghanbari et al., 2020). To determine the RSI, the average permanent strain due to a predefined traffic level at the end of a 20-year pavement service life (240 months) is calculated.

2.3.1.4 Cyclic Fatigue Test

The cyclic fatigue test is based on the Simplified Viscoelastic Continuum Damage (S-VECD) fatigue theory. Damage is only manifested through tensile stress (pull–pull loading scheme). The main output of the test is the damage characteristic curve (C-S curve), which relates material integrity to microstructural damage. Gibson and Li (2015) observed how the C-S curves are not designed to rank and judge performance but still represent the behavior of the different mixes. In general, higher curves can sustain more microcracking with less change in pseudo-stiffness (C), therefore better performance.

The S-VECD test procedure uses the AMPT to first conduct a dynamic modulus test to determine the linear viscoelastic properties of the mix. Afterwards, the direct tension cyclic test is performed to develop the damage characteristic curve. Together, the viscoelastic material properties and damage characteristic curve can be used to obtain the fatigue behavior of an asphalt mixture. A few AMPT cyclic fatigue tests at a single temperature are sufficient to obtain the S-VECD model (Underwood et al., 2012).

A traditional way of evaluating the fatigue life of an asphalt mixture specimen is to consider the number of cycles corresponding to a 50% reduction of the initial stiffness. Instead, Li et al. (2017) studied the use of the number of cycles corresponding to the maximum phase angle versus actuator strain level as indicator to evaluate the fatigue life. Furthermore, to define the asphalt mixtures' fatigue performance, a fatigue cracking index parameter, referred to as apparent damage capacity (S_{app}), has been introduced (Wang et al., 2020). The goal was to allow pavement engineers to make rapid decisions, based on the S-VECD test results, about mix design and mix quality control.

2.3.2 Cracking Tests

Cracking tests can be classified by the type of cracking which is investigated, namely fatigue cracking, thermal cracking, and reflection cracking. Through the NCHRP Project 09-57 "Experimental Design for Field Validation of Laboratory Tests to Assess Cracking Resistance of Asphalt Mixtures", Zhou et al. (2016) conducted a thorough review and classification of the most used and promising cracking tests, based on the type of cracking investigated.

Another way of categorizing the cracking tests is by the tests' loading conditions. Generally, it is possible to separate the tests which investigate the materials' response to repeated loads and the tests which evaluate the fracture energy needed to break a specimen in response to a monotonic load. In addition, some tests, like the Thermal Stress Restrained Specimen Test (TSRST), feature unique characteristics.

Table 13 summarizes the most popular cracking tests highlighting the type of cracking investigated and the loading conditions. It is possible to observe how some tests can be used for multiple cracking types, i.e. the Semi-Circular Bend (SCB) test and the Disk-Shaped Compact Tension (DCT) test, or in multiple loading conditions, i.e. the Direct Tension (DT) test.

Table 13 Overview of HMA cracking tests

CRACKING TYPE	LOADING CONDITIONS		
	Repeated Load	**Monotonic Load**	**Other**
Thermal	• N/A	• DCT • SCB • IDT	• TSRST
Fatigue	• Bending Beam • Trapezoidal Beam • DT • HWTT • Texas OT • S-VECD	• Fénix • DT • SCB-LTRC • SCB-IL • IDT • CT_{Index}	• N/A
Reflection	• Texas OT • WRC	• DCT	• N/A

2.3.2.1 Thermal Cracking Tests

Thermal cracking of asphalt mixes is caused by the tensile stresses induced by the contraction of the materials, which happens when temperature decreases. Usually it manifests as transversal cracks on the road since there is more restraint along the longitudinal direction of the pavement. Also, thermal cracks are likely to start from the surface of the mix because of the higher exposure to the cooling environment. The thermal cracking mechanism is shown in Figure 20.

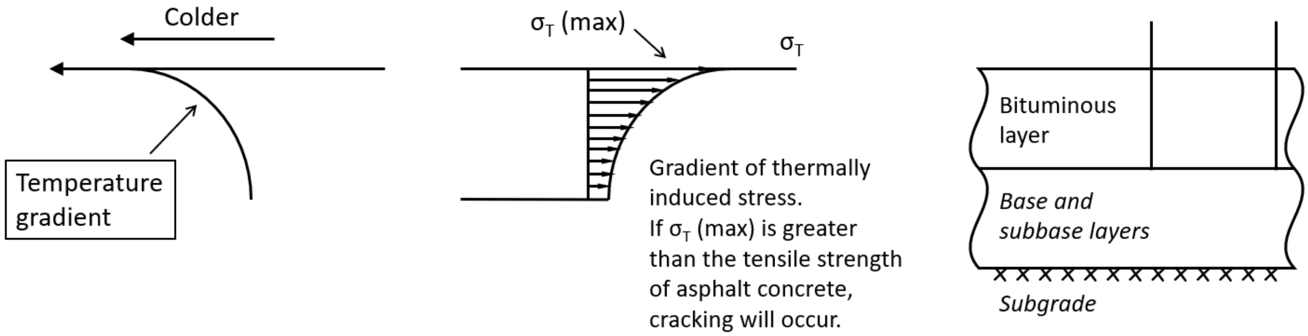
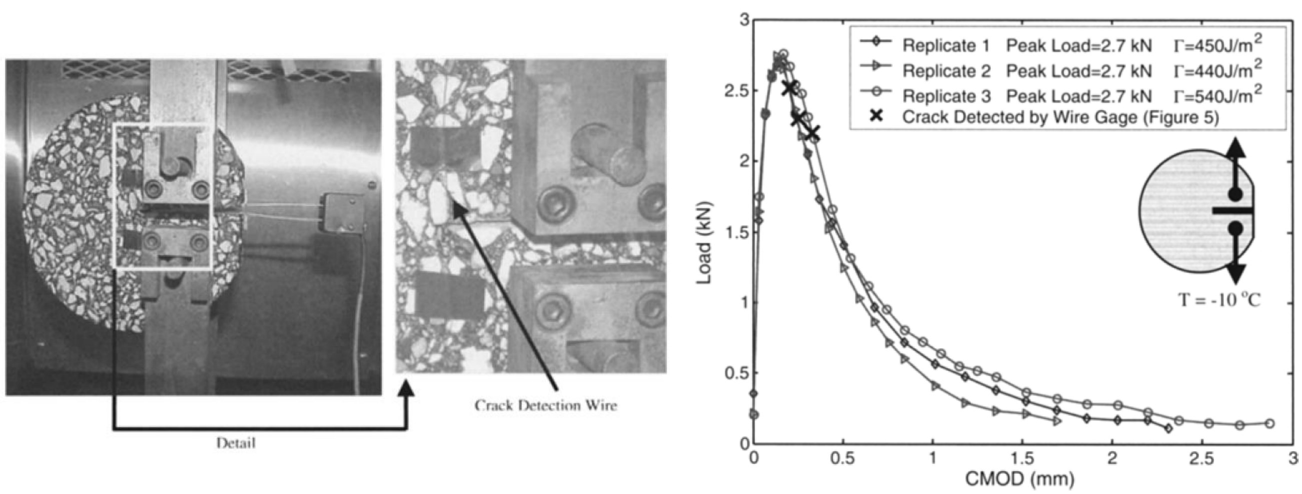


Figure 20 Cross section of cold pavement showing temperature and thermal stress gradients (Haas et al., 1987)

Disk-Shaped Compact Tension (DCT) Test

The test features a disk-shape specimen with a 62.5 mm (2.46 inch) notch. The specimen is pulled at crack mouth opening displacement (CMOD) rate of 1 mm/min. During the test, the temperature is usually maintained constant at 10 ºC warmer than the PG low temperature grade. The fracture energy (G_f) is calculated as the area under the load-CMOD curve normalized by the initial specimen section. Larger fractures energies indicate better thermal cracking resistance. The test setup and the typical output are shown in Figure 21.



(a) (b)

Figure 21 (a) DCT experimental setup, (b) typical load-CMOD curve (Wagnoner et al., 2005)

Semi-Circular Bend (SCB) Test

The SCB test features a semi disk-shape specimen with a 15 mm (0.6 inch) notch. The SCB test is run in a CMOD mode at a 0.03 mm/min. The temperature and fracture energy calculation are kept analogous to the DCT test. The test setup is shown in Figure 22.

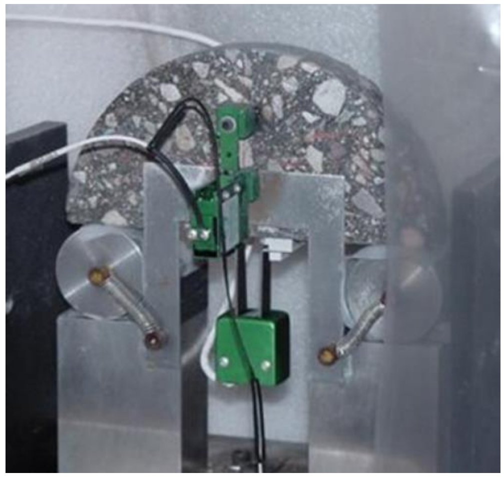


Figure 22 SCB experimental setup (Marasteanu et al., 2012)

Indirect Tensile (IDT) Test

The test consists of a compression test on a cylindrical specimen. According to the AASHTO standard T 322 the test can measure both creep compliance and tensile strength (AASHTO, 2016). For the creep test, a load level that produces a horizontal deformation between 0.00125 mm and 0.019 mm is maintained constant for 1,000 seconds. In order to calculate creep compliance and stiffness as a function of time, the horizontal and vertical deformations are recorded during the loading process. The tensile strength is calculated loading the specimen at a constant rate of 12.5 mm/min until failure. To calculate the failure strength the peak load and the specimen dimensions are used (Zhou et al., 2016).

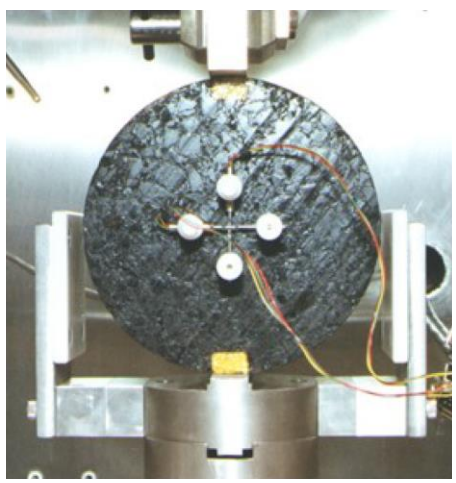
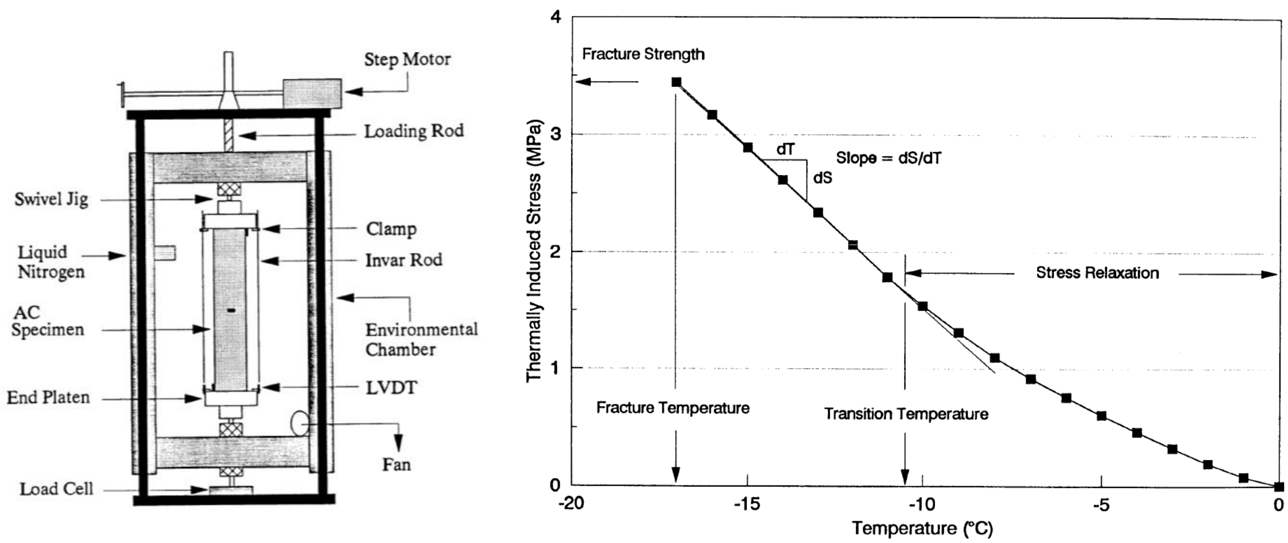


Figure 23 IDT experimental setup (Marasteanu et al., 2012)

Thermal Stress Restrained Specimen Test (TSRST)

The TSRST features the use of a long beam with a square section of 5 cm by 5 cm, and 25 cm long (2 inch by 2 inch, 10 inch long). The specimen is placed in an environmental chamber and restrained from contracting. The specimen is subjected to a decreasing temperature at a rate of 10°C/hour until fracture. Throughout the experiment the tensile stresses and the temperature are monitored in order to define the fracture temperature and the fracture strength. Similar to the TSRST is the Uniaxial Thermal Stress and Strain Test (UTSST) developed by University of Nevada at Reno which, instead of the beam specimen, features a cylindrical laboratory compacted specimen with a in a typical size of 5.7 cm (2.25 inch) diameter and a 14 cm (5.5 inch) height. The test setup and the typical output are shown in Figure 24.



(a) (b)

Figure 24 (a) Schematic of TSRST setup; (b) typical test result (Jung & Vinson, 1994)

2.3.2.2 Fatigue Cracking Tests

Asphalt pavement fatigue cracking can be differentiated between bottom-up and top-down mechanism. Bottom-up fatigue cracking is the result of repetitive tensile strains applied by traffic loads being applied to the pavement surface. The entity of the stains is usually lower than the level that would cause fracture due to a single load application. Bottom-up fatigue cracking consists of the two phases: crack initiation and crack propagation. During the first phase, microcracks grow from microscopic size until the length of 7.5 mm (Lytton et al., 2018). In the propagation phase the cracks grow until reaching the surface of the pavement. Top-down fatigue cracking starts on the surface and propagates downward in the asphalt layer. It develops in the longitudinal direction both within the wheel path and outside of the wheel path (Harmelink & Aschenbrener, 2003). Top-down cracking consists of the three phases: appearance of single short longitudinal crack outside of the wheel path, development of parallel companion cracks, and formation of short transverse connection cracks. Top-down cracking can be either construction-related tor load-related. Construction-related top-down cracking is caused by the segregation of aggregate in the asphalt mixture during construction. Load-related top-down cracking is produced by bending-induced surface tension away from the tire (Roque et al., 2010).

<u>Flexural Bending Beam Test</u>

The test features the use of a 380 mm long beam specimen with a section of 50 mm by 63 mm (2 inch by 2.5 inch, 15 inch long). During the test, the beam is held by four clamps and subjected to a repeated haversine (or sinusoidal) loading through the two inner clamps whereas the outer clamps provide a reaction load. This configuration produces a constant bending moment over the center part of the beam between the two inside clamps (Zhou et al., 2016). The test is usually run in strain-controlled mode with a frequency of 10 Hz and is performed at intermediate temperature, usually 20°C (68°F). The test apparatus is shown in Figure 25. The test is standardized in AASHTO T 321 “Determining the Fatigue Life of Compacted Asphalt Mixtures Subjected to Repeated Flexural Bending”. It must be noted that the beam preparation is quite time consuming and the test machine is usually a universal test machine which costs above $150,000 (Zhou et al., 2016).

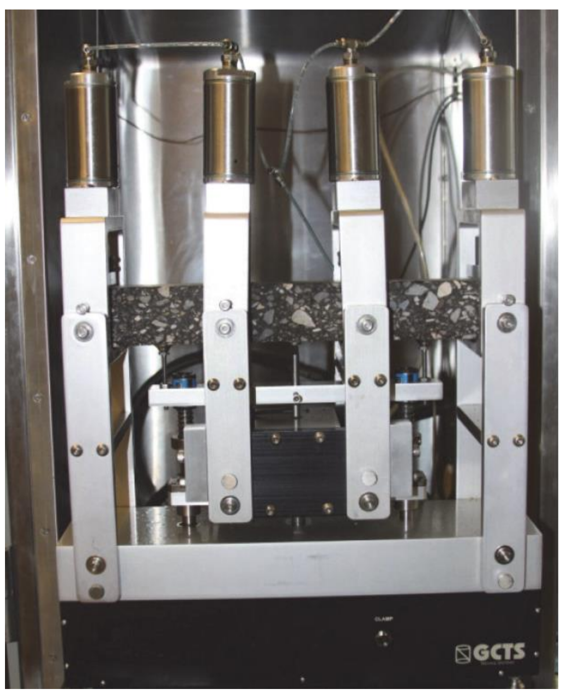


Figure 25 Flexural bending beam test setup (Islam & Tarefder, 2015)

Trapezoidal Beam Fatigue Test

The trapezoidal beam fatigue test features a specimen 250 mm (10 inch) tall, 25 mm (1 inch) thick, while the small and large base of the trapeze are 25 mm (1 inch) and 55 mm (2.2 inch) respectively. The specimen is glued to a bottom plate in an upright position and a thin metal plate is glued on top of the specimen and connected to a load cell, as shown in Figure 26. The test is usually run in strain-controlled mode with a frequency of 15 Hz and is performed at intermediate temperature, usually 20°C (68°F). The fatigue life is usually determined as the point in which the stiffness modulus reaches 50 percent of its initial value.

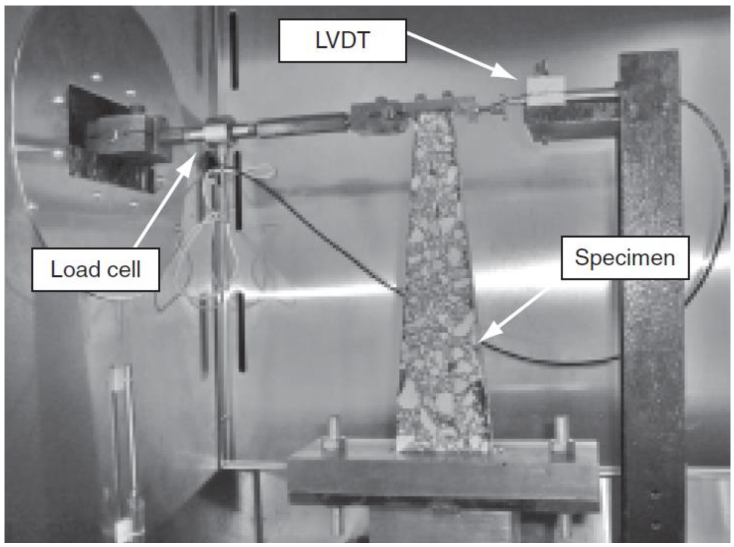


Figure 26 Trapezoidal beam fatigue test setup (Cocurullo et al., 2008)

Fénix Test

The Fénix test features a semi disk-shape specimen with a 6 mm (0.24 inch) notch. The specimen is glued to two steel plates and put in tension at a constant displacement rate of 1 mm/min at a specified temperature until failure. The dissipated energy is calculated as the area under the load displacement curve divided by the specimen section. In addition, the tensile stiffness index is calculated as half the peak load (F_{max}) divided by the displacement before peak load Δm (Pérez-Jiménez et al., 2010). The test setup and the typical output are shown in Figure 27.

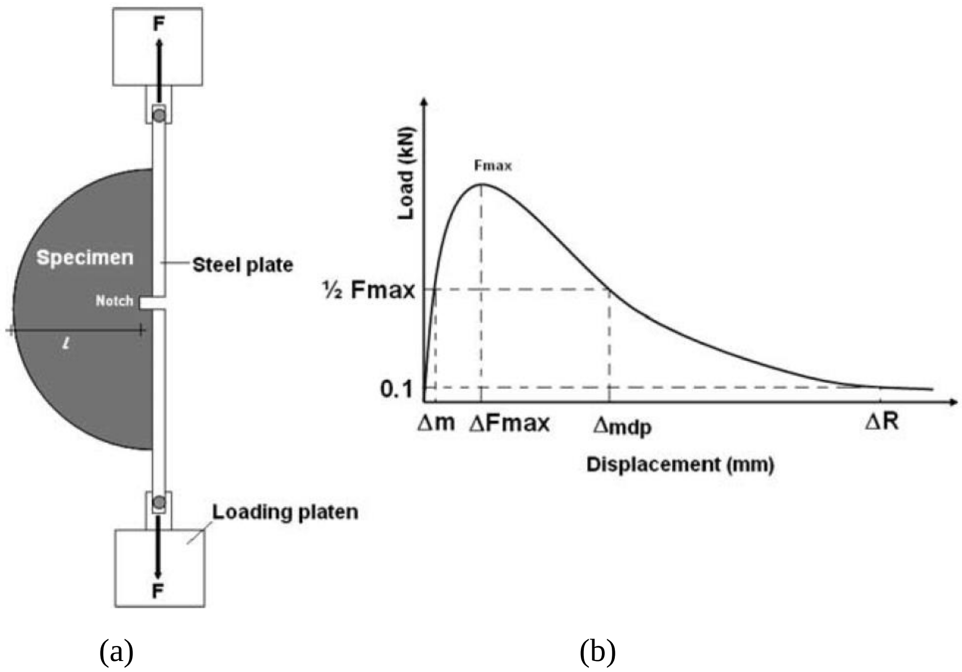


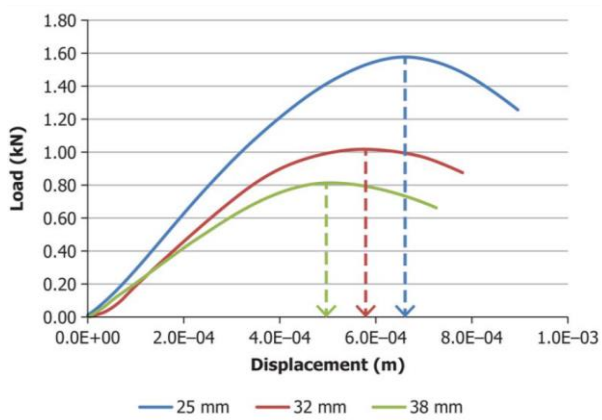
Figure 27 (a) Fénix test setup; (b) typical output curve (Pérez-Jiménez et al., 2010)

Direct Tension (DT) Test

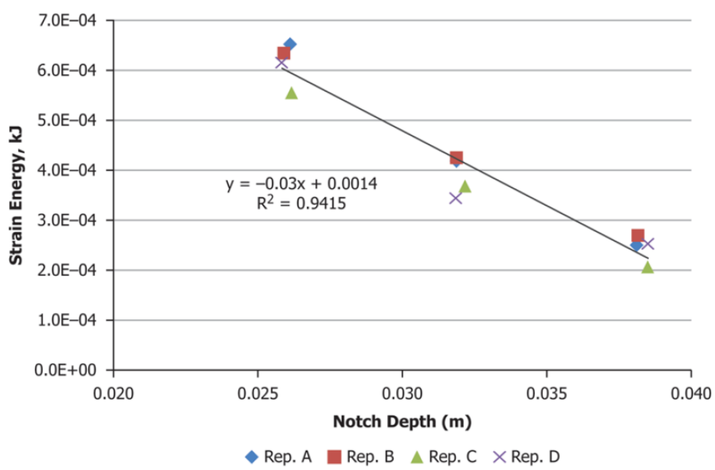
The DT test is conducted on cylindrical specimens glued to end plates and subjected to uniaxial tensile loading. The test can be performed either in repeated or monotonic loading. Both viscoelastic properties and fracture energy information can be obtained. In the first case, the test can be used to evaluate top-down fatigue cracking, while in the second the test can determine the tensile modulus of the asphalt mix (Luo et al., 2013). In order to test field cores Lytton et al. (2018) tested rectangular specimen instrumented with linear variable differential transformers (LVDTs) used to measure the vertical deformation. The DT test can be performed using the AMPT or the Material Test System (MTS).

Semi-Circular Bend-Louisiana (SCB-LTRC)

This test was developed by the Louisiana Transportation Research Center (LTRC). It features the use of a semi disk-shape specimens with three different notch depths: 25 mm, 32 mm, and 38 mm (1.0, 1.25, and 1.5 inch). Each specimen is loaded at a cross-head controlled rate of 0.5 mm/min. The strain energy to failure is calculated for each notch depth and it is calculated as the area under the loading portion of the load versus deflection curve, up to the maximum load measured. The three loading curves are shown in Figure 28 (a). The energy values are then plotted against the notch depths to calculate the critical strain energy release rate, J_c. J_c is calculated as the slope of the linear regression line, shown in Figure 28 (b), divided by the average sample thickness. This parameter can be used to rank the resistance of asphalt mixtures to cracking (ASTM, 2016).



(a)

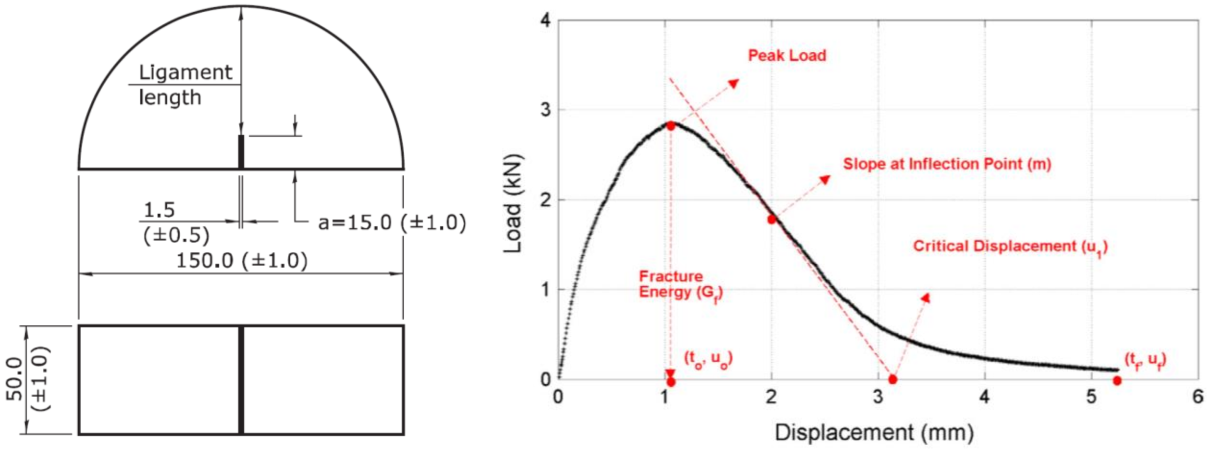


(b)

Figure 28 (a) Load versus displacement plots for each notch depth; (b) Example of notch depth versus strain energy plot used to determine J_c (ASTM, 2016)

<u>*Semi-Circular Bend-Illinois (SCB-IL)*</u>

This test was developed by the University of Illinois. It features semicircular specimens with a notch depth of 15 mm, as shown in Figure 29 (a). The specimens are loaded along the vertical radius of the specimen at a constant load line displacement (LLD) rate of 50 mm/min. The main output of the test is the Flexibility Index (FI) which supports the identification of brittle mixtures, that may be prone to premature cracking, and indicates an asphalt mixture's overall capacity to resist cracking related damage (AASHTO, 2018b). Fracture energy (G_f), post-peak slope (m), displacement at peak load (u_0), critical displacement (u_1), and a FI are calculated from the load and LLD results, as shown in Figure 29 (b).



(a) (b)

Figure 29 (a) SCB-IL specimen configuration, dimensions in millimeters (AASHTO, 2018b); (b) Typical outcome of the SCB test illustrating the parameters derived from the load-LLD curve (Ozer et al., 2016)

The test requires a significant amount of time with respect to preparation of the specimens. It starts with the compaction of a gyratory pill with a height of 160-mm, from which two middle discs of 50-mm thickness are extracted as shown in Figure 30. The air voids content for each disc has to be 7.0 ± 1.0 percent. The discs are then cut in half and a 15-mm notch is cut in each specimen, as shown in Figure 31.

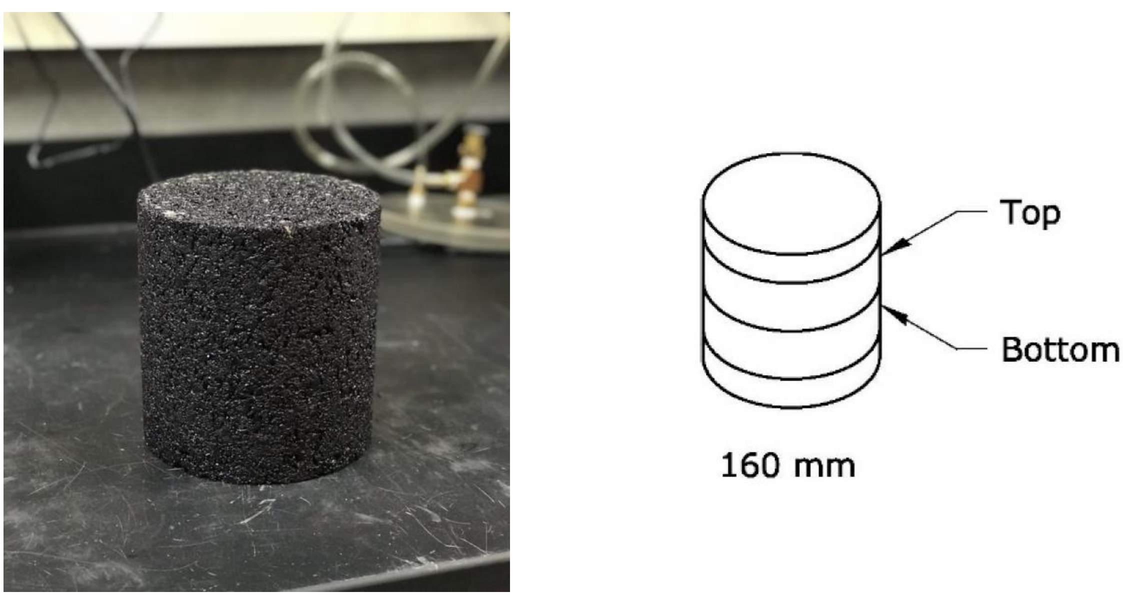
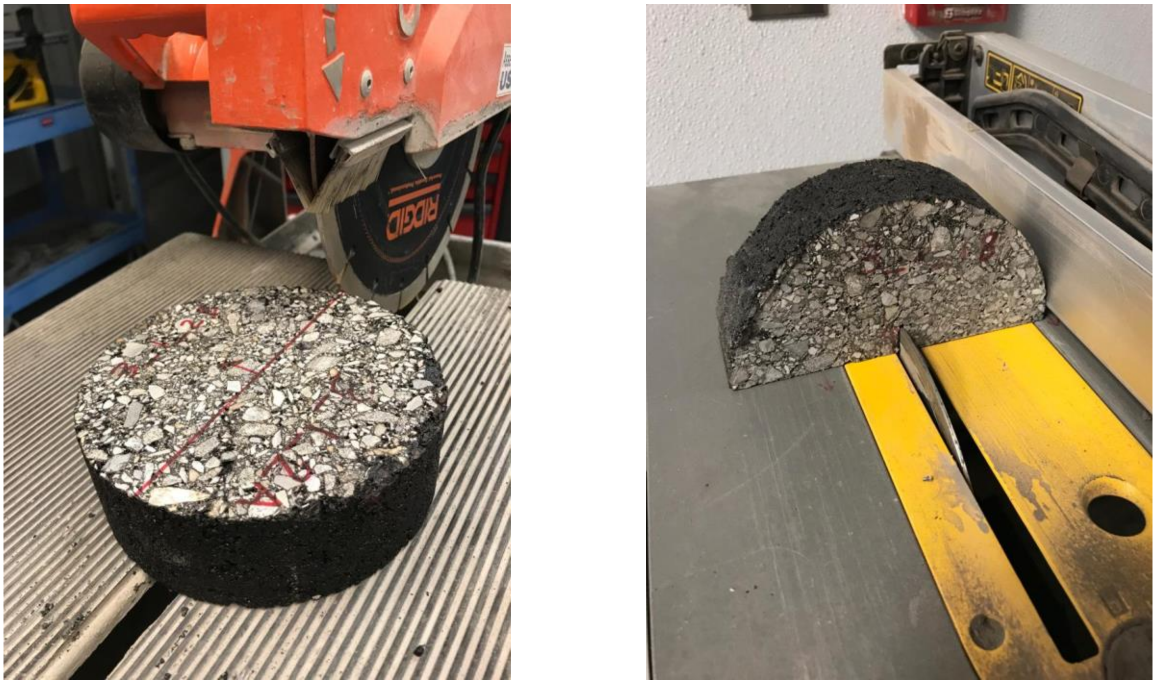


Figure 30 (a) 160-mm gyratory pill; (b) top and bottom slices (AASHTO, 2018b)



(a) (b)

Figure 31 Cutting (a) and notching (b) operations

Indirect Tensile (IDT) Test

The IDT test, conducted at room temperatures of 20°C to 25°C (70°F to 77°F), was studied by Kim and Wen (2002). The authors found a very good correlation between field fatigue cracking percentage in the test track and the fracture energy defined by the area under the stress-strain curve in the loading portion (Figure 32). The IDT was performed as a monotonic test with a loading rate of 50 mm (2 inch) per minute until failure. Reynaldo Roque et al. (2004) developed, at the University of Florida, the use of the IDT test specifically for top-down fatigue, using three tests (resilient modulus, creep compliance, and tensile strength test) a temperature of 10°C (50°F).

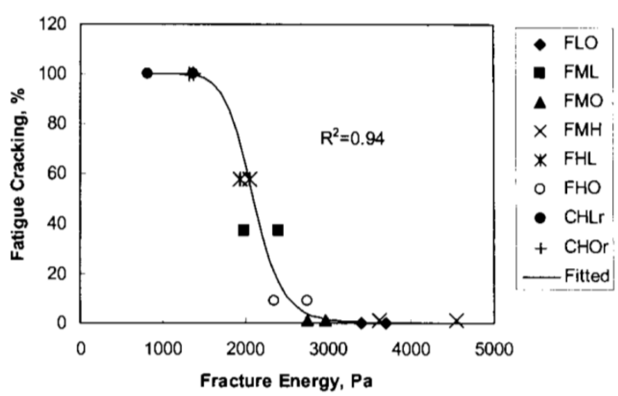


Figure 32 Relationship between field fatigue performance and fracture energy (Kim & Wen, 2002)

CT_{Index}

This test shares a similar setup to the IDT test, shown in Figure 33. A cylindrical specimen is loaded at a constant load-line displacement (LLD) rate of 50.0 mm/min. For the entire duration of the test, both the load and LLD are measured to calculate the CT_{Index} (ASTM, 2019). The CT_{Index} is calculated as a function of failure energy G_f, post-peak slope m_{75}, LLD at 75% the peak load after the peak, and specimen geometry (Figure 34). The test is run at room temperature (25°C) and usually is completed in less than 1 minute. The main benefits of the test reside in its simplicity (there is no need for instrumenting, cutting, and notching of specimens) and practicality (the required training is minimum). In addition, the test is sensitive to key parameters such as RAP and recycled asphalt shingles (RAS) content, asphalt binder type, binder content, aging conditions, and air voids (Zhou et al., 2017).

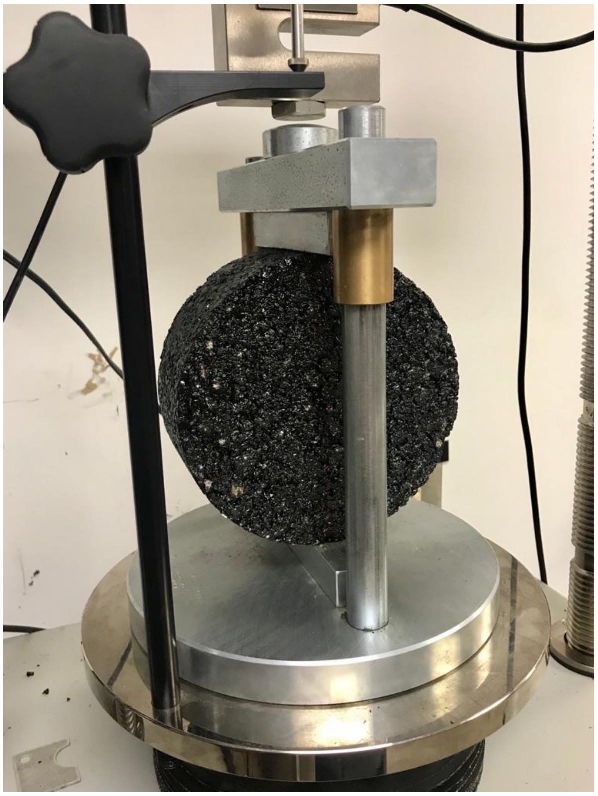


Figure 33 Indirect tensile cracking test setup for CT_{Index} calculation

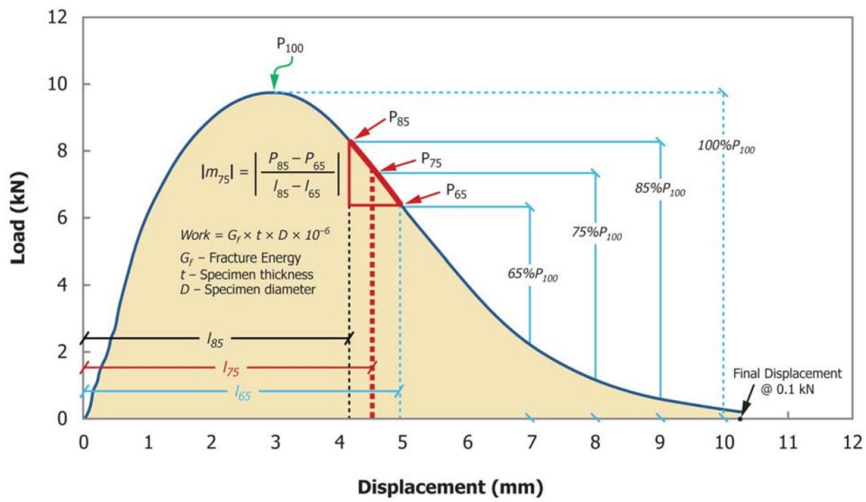


Figure 34 Recorded load versus LLD curve (ASTM, 2019)

<u>*Hamburg Wheel Tracking Test (HWTT)*</u>

The HWTT was designed for evaluating the rutting resistance and stripping potential of asphalt mixtures. It has been observed how wheel tracking devices could be used to evaluate resistance to top-down cracking as well. In fact, surface cracks can accompany the rutting in the field as well as in the laboratory test. The HWTT and other accelerated loading tests can produce surface cracking in specimens during the rutting tests (De Freitas et al., 2005).

2.3.2.3 Reflective Cracking Tests

Reflective cracking consists in the formation of cracks in an HMA overlay placed on existing joints or cracks. The presence of joints or cracks results in stress concentrations at the bottom of the asphalt overlays which can lead to the formation and growths of cracks in the overlay. The stress concentrations are usually due to movements in the underlying pavement which can be induced either by traffic loads or temperature changes.

<u>*Texas Overlay Test (OT)*</u>

The Texas OT apparatus applies repeated direct tension loads to the test specimens. It consists of two steel blocks to which the asphalt mix specimen is glued. One plate is fixed and the other is movable to simulate the opening and closing of joints or cracks beneath an overlay (TxDOT, 2009). The OT specimen size is obtained cutting a gyratory specimen. The final dimensions are 15 cm (6-inch) long, 7.6 cm (3 inch) wide, and 3.8 cm (1.5 inch) high. The Texas OT is a cyclic displacement-controlled test and the number of load cycles to failure is reported as well as fracture properties.

Wheel Reflection Cracking (WRC) Device

The WRC device was developed by Gallego and Prieto (2006). It features 305×305×60 mm (12×12×2.4 inch) test specimens cut from slabs. The specimens are positioned on top of two plates, one fixed and the other one is displaced horizontally. The specimens are loaded by a moving wheel and the deflection is simulated by placing a rubber block under the rocker support. The test setup is shown in Figure 35.

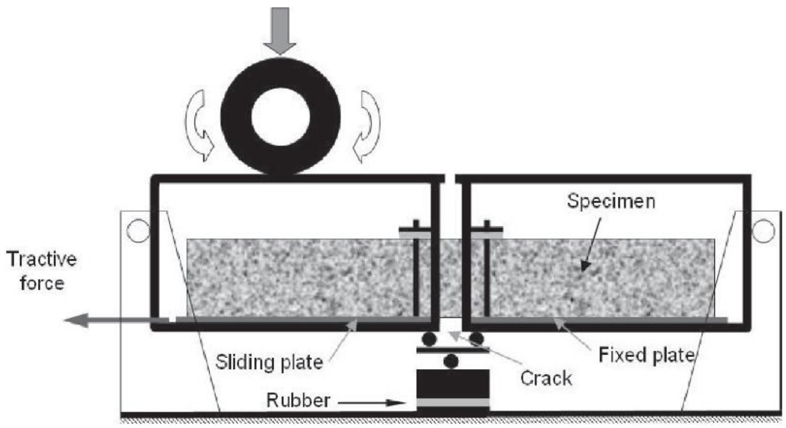


Figure 35 WRC setup scheme (Gallego & Prieto, 2006)

2.3.3 Rutting Tests

Rutting consists of the accumulation of permanent deformation in the pavement structure. It usually occurs as longitudinal surface depressions in the vehicles' wheel paths possibly accompanied by pavement uplifts on the sides. In a typical asphalt road structure, rutting can occur both in base/subgrade layers and in the asphalt layers, as a consequence of traffic loading. In the first case, the quality of the subgrade material could be poor, or the overlying asphalt layers could be of insufficient thickness. In the second case, rutting can be caused by inadequate compaction, which results in a post-compaction of the material, or improper mix design, which results in shearing deformation. This last factor can be investigated by specific laboratory devices, of which the main ones are described below.

French Loaded Wheel Tester (LWT)

The French LWT was developed at the Laboratoire Central des Ponts et Chaussées (LCPC) and is known also as the French Rutting Tester (FRT). The FRT tests HMA slabs 180 mm wide, 500 mm long, and 100 to 200 mm thick. The testing consists of a controlled loading of 5,000 N through a pneumatic tire inflated to 600 kPa. The load is applied at a rate of one cycle per second with each cycle composed of a back and forth stroke. Before testing, the slabs are first thermally conditioned for 12 hours at a temperature between 35°C and 60°C. After the conditioning, the slabs are loaded for 30,000 cycles, which correspond to approximately 9 hours. Rut depth are taken at growing intervals: after 100, 300, 1000, 3000, 10000, and 30000 cycles (Kandhal & Cooley, 2003).

Hamburg Wheel Tracking Test (HWTT)

The HWTT was developed in Hamburg, Germany. Originally, the HWTT tested slabs of HMA 260 mm wide, 320 mm long, and 40 mm thick. Today, a new configuration with cylindrical specimens can be used, as shown in Figure 36. The specimen is submerged in a temperature-controlled water bath and repetitively loaded using a steel wheel. The use of the HWTT is specified by the AASHTO method T 324, "Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)". Through the application of 20,000 cycles of loading, the rut depth evolution is evaluated against the number of passes and the main test parameters (stripping inflection point and number of passes to failure) are obtained.

Figure 36 (a) HWTT test apparatus

Asphalt Pavement Analyzer (APA)

The original version of the APA was developed in the 1980s and was called Georgia Loaded Wheel Tester (GLWT). The GLWT was a modified version of a wheel tracking device for slurry seals. The main purpose of the GLWT was to provide a reliable laboratory test for HMA rutting performance (Kandhal & Cooley, 2003). The GLWT was then modified in 1996 and the APA was developed. It consisted of a loaded wheel on top of a pressurized linear hose and moved back and forth over a test specimen. The test specimens can be laboratory produced, plant produced, or roadway cores. The specimens must be 150 mm (5.91 in.) in diameter by 75 ± 2 mm (3.0 ± 0.1 in.) tall. Today, the use of the APA is specified by the AASHTO method T 340, “Determining Rutting Susceptibility of Hot Mix Asphalt (HMA) Using the Asphalt Pavement Analyzer (APA)”. During the test 8,000 cycles of loading are applied, which correspond to approximately 2 hours of testing time. The test temperature should be set to the high temperature of the standard Superpave performance-graded (PG) binder (AASHTO, 2018a). Figure 37 shows the loading apparatus (a) and the test cabin (b).

(a) (b)

Figure 37 (a) APA loading wheels and hoses; (b) APA cabin

It can be observed how all the listed rutting tests do not yield a fundamental material property that can be used in a pavement model for distress prediction. Therefore, it is fundamental to investigate the relationship between test results and field performance. However, at the same time, it is important to keep in mind that the field performance depends also on the properties of the whole pavement structure such as subgrade support, thicknesses, traffic levels, and environmental conditions.

2.4 FULL SCALE ACCELERATED PAVEMENT TESTING

2.4.1 Overview

Accelerated Pavement Testing (APT) is a destructive test procedure that allows determining the performances of a full-scale pavement system through the controlled application of a wheel loading. APT has been defined as the “vital link between the laboratory evaluation of materials used in pavement layers and the field behavior of these materials when combined into pavement structures” (Steyn, 2012). In fact, concerning the time necessary to obtain information and results, laboratory testing takes weeks while long term monitoring takes years. APT gives results over the course of a few months. At the same time, APT offers more reliable analyses than laboratory testing and is less demanding in terms of time and cost than monitoring. Figure 38 shows a general framework for the main methods used to evaluate pavement materials.

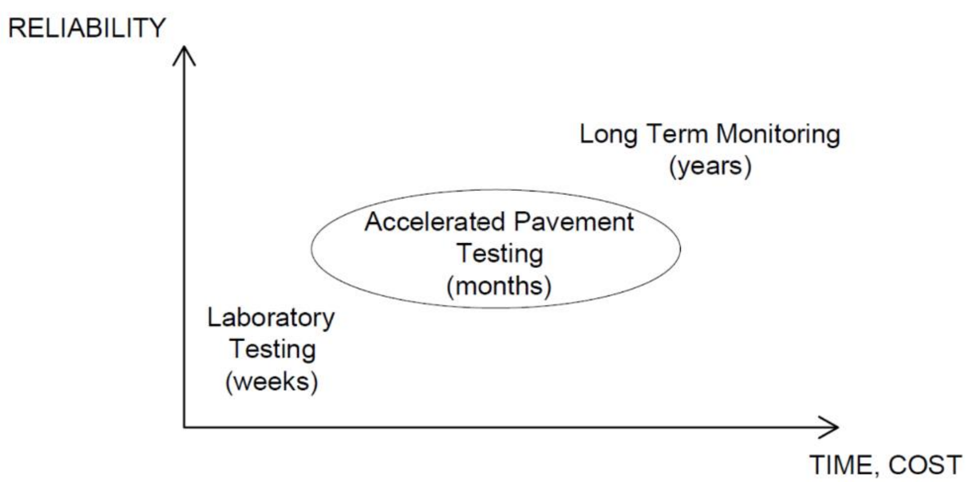


Figure 38 Comparison between pavement evaluation methods

2.4.2 Virginia Tech Transportation Institute (VTTI) Facility

The Virginia Department of Transportation (VDOT) and the Virginia Tech Transportation Institute (VTTI) started the APT program in November 2015 with the goal of speeding up pavement testing and evaluating in a compressed amount of time the performances of different types of pavement to various traffic loads and environmental conditions. The Heavy Vehicle Simulator (HVS) is the centerpiece of the APT facility and works 24 hours a day 7 days a week, with brief interruptions due to routine maintenance operations. The HVS has been developed by Dynatest and the model installed at VTTI is the Mark VI, shown in Figure 39. It consists of a 54 tons mobile machine designed to be easily moved to and from the testing sites. It features two standard commercial trailers and a main test beam so that the unit can be transported in two sections. The beam supports a hydraulic system that manages to move and lift the load which consists of dual truck wheels. The wheels are driven back and forth over a length of pavement section which can vary from 6 m (20 ft) to 12 m (40 ft). The section can be subjected to unidirectional or bidirectional loading, the latter can maximize productivity but at the same time does not simulate properly the actual road traffic. Also, to obtain a better simulation the wheel can move with a programmable wander between passes in increments of 25 to 75 mm over 0.8 m. The HVS Mark VI can apply load levels between 30 and 100 kN (7 and 22.5 kips) and the maximum testing speed is 12.8 km/h (8 mph). At VTTI the standard load configuration consists of 40 kN (9000 lbf, 9 kips), which correspond to 1 Equivalent Single Axle Load (ESAL). The speed is equal to 6.44 km/h (4 mph). As the testing on a specific cell progresses, the load is usually increased to 53 kN (12 kips) and 67 kN (15 kips) in order to speed up the deterioration process.

Figure 39 Dynatest HVS: Mark VI

2.4.3 Review of HMA Surface Mix Studies

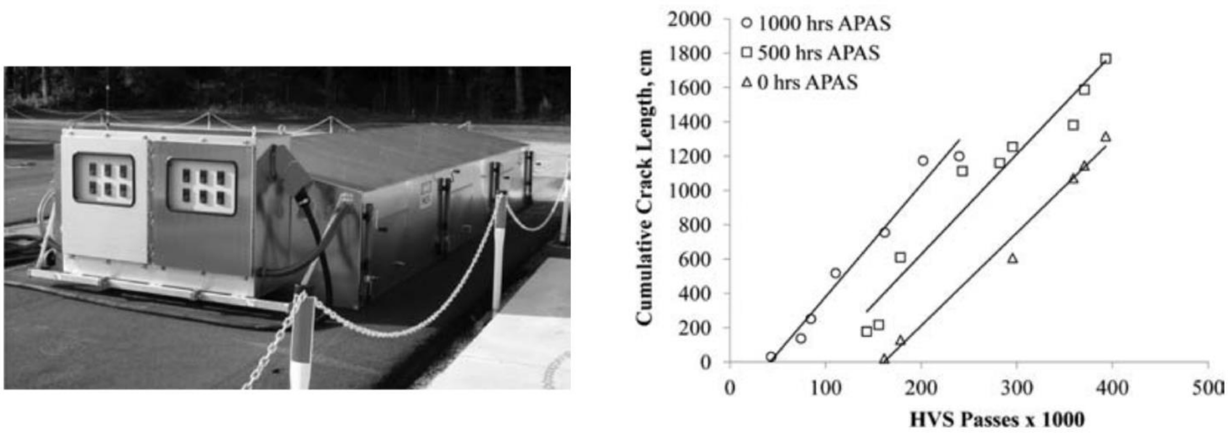
APT can be used to investigate multiple types of distresses. The correct definition of loading and environmental conditions is critical in order to investigate appropriately the deterioration processes. For example, rutting can be favored by keeping high temperature and slow loading speeds during the testing. Instead, the evaluation of fatigue cracking requires a colder environment and extensive trafficking which generally corresponds to longer testing times and higher costs (Hugo et al., 2012). This section has the objective of reviewing some of the most significant studies conducted in the past years to examine the behavior of HMA surface mixes, with respect to different distresses.

Federal Highway Administration (FHWA)

The FHWA used the Accelerated Loading Facility (ALF) located at Turner-Fairbank Highway Research Center to compare different surface mixtures in terms of fatigue cracking. The goal was to identify a new way of specifying the performances of virgin binder to supplement $|G^*|\sin(\delta)$ in the Superpave performance Grading (PG) system (Gibson et al., 2012). The surface mixes were laid with a thickness of either 100 mm (4 cm) or 150 mm (6 cm). The total thickness of asphalt mixes and subgrade was 660 mm (26 in.). The load of 71 kN (16,000 lb.) was applied by a 425 super single tire inflated to 827 kPa (110 psi). The lateral wandering was programmed with a standard deviation of 133 mm (5.25 in.). The test was performed under a temperature of 19°C (66.2°F) kept constant by radiant heaters. To compare the performances of the mixes, the evolution over time of fatigue cracking can be evaluated in terms of cumulative crack length and percentage of area cracked.

Florida DOT (FDOT)

The Florida DOT (FDOT) started an APT program in 2000 to investigate flexible, rigid, and composite pavements. In order to investigate cracking resistance, FDOT used an accelerated pavement aging system (APAS) which was capable of rapidly aging the pavement and limiting the healing capacity (Greene & Choubane, 2012). The APAS featured long radiant heaters that heat the pavement up to 90°C (194°F) and water nozzles used to cool the pavement from 90°C to 30°C. The APAS was used to study both top-down and bottom-up cracking under APT loading. Figure 40 shows the APAS equipment and the effect on the cracking performance of the pavement.



(a) (b)

Figure 40 (a) APAS, (b) fatigue cracking due to different aging times (Greene & Choubane, 2012)

California Department of Transportation (Caltrans)

Since 1995 the California Department of Transportation (Caltrans) used two HVSs to fill the gap between laboratory testing and pavement field performance. With the main objective being the calibration of the mechanistic-empirical models of a computer program known as CalME (Ullidtz et al., 2008), different goals were pursued and by consequence different conditions were set for the APT experiments on surface mixes:

- Goal 1 - Fatigue cracking: dry conditions, intermediate temperature (20°C)
- Goal 2 - Reflective cracking: overlays placed on previously cracked layers, dry conditions, and intermediate temperature (20°C)
- Goal 3 - Rutting: dry conditions, 40°C and 50°C at 50 mm depth
- Goal 4: Use of permeable bases: wet conditions (water introduced into base layers) and intermediate temperatures, 20°C

Texas Department of Transportation (TxDOT)

Because of concerns relative to the cracking resistance of the mixes routinely used on Texas highways, an APT program was employed to compare the performances of traditional and performance-optimized mixes (Walubita & Scullion, 2013). The goal was to test the mixes in terms of resistance to rutting, reflective cracking and fatigue cracking.

The rutting study was conducted during the fall, when the average temperature was 23.6°C (74.5°F). The reflective cracking study was conducted during the winter months and the average air temperature was 9°C (48°F). The fatigue cracking was conducted during the spring with and average temperature of 73°C (22.7°F).

While the first two studies correlated well with the laboratory prediction, the last one performed unexpectedly as all sections accumulated high rutting. The research team inferred that this behavior was likely to be related to the high temperatures of the test, underlining once again the importance of the definition of the right testing parameters, especially the temperature.

2.5 SUMMARY OF LITERATURE REVIEW

Through the literature review, it was possible to observe how the methods of designing asphalt mixtures evolved over time. Since the first procedures, the importance of testing the mixtures at the design stage was highlighted. Only through the introduction of very detailed requirements (through Level 1 of Superpave) it was possible to achieve reliable field performances without using laboratory performance tests as part of the standard design procedure. However, over time, the Superpave guidelines showed the need of being adjusted and, at the same time, the diffusion of practices such as pavement recycling called for further changes to the volumetric-based design method. Therefore, performance tests were again identified as the most appropriate supporting tools. The literature review explored the variety of tests that were developed over the years, from laboratory experiments to full-scale testing facilities. It was possible to see how the majority of these tests can today be considered by state agencies and contractors to upgrade their design procedures and to optimize the production of asphalt mixtures.

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CHAPTER 3 – IMPACT OF MIX DESIGN OPTIMIZATION ON HMA RUTTING PERFORMANCE UNDER ACCELERATED PAVEMENT TESTING

Fabrizio Meroni

Graduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Wenjing Xue, Ph.D.

Research Associate
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Gerardo W. Flintsch, Ph.D., P.E.

Professor, Charles Via, Jr. Department of Civil and Environmental Engineering, Virginia Tech
Director, Center for Sustainable Transportation Infrastructure,
Virginia Tech Transportation Institute

Brian K. Diefenderfer, Ph.D., P.E.

Principal Research Scientist
Virginia Transportation Research Council

3.1 ABSTRACT

To build durable asphalt pavements, one of the main concerns that many transportation agencies have shared in recent years is the low binder content of mixes designed according to Superpave. In order to add more binder and still be able to achieve satisfying performance, adjustments to the design compaction energy and mix aggregate structure can be made. The Virginia Department of Transportation started an Accelerated Pavement Testing program at the Virginia Tech Transportation Institute in 2015. Two test lanes were used to investigate the performance of two dense-graded surface mixtures designed with different design gyration levels (50 and 65). Laser profiler, Multi-Depth Deflectometer, and pressure cells were used to monitor pavement response and permanent deformation throughout the experiments. This paper analyzes the instrumentation responses and permanent deformation collected during the APT experiments. In parallel, pavement cores were tested in the laboratory and analyzed in terms of dynamic modulus, flow number, and rutting resistance. The results showed that the optimized surface mixture, which was designed with 50 gyrations, was able to achieve a similar or better rutting resistance than the mixture designed at the traditional 65 gyration level.

Keywords

Accelerated Pavement Testing, Gyrations, Compaction, Rutting

3.2 INTRODUCTION

The Superpave mix design method was introduced in 1993 with the purpose of creating asphalt mixtures with sufficient binder content for long-term performance and durability (FHWA, 2010). However, soon after the nationwide application of Superpave, many transportation agencies found that the mixes they produced using the Superpave design method were low in asphalt content, which resulted in poor durability, especially with respect to cracking resistance, and had shorter service lives. Prowell and Brown (2007) conducted a study on the mixes' densification and, after collecting material from 40 field projects, recommended a reduction of the gyratory compaction levels in order to allow higher asphalt contents. The Federal Highway Administration also suggested that transportation agencies perform an independent evaluation in order to modify the number of gyrations (FHWA, 2010).

The Virginia Department of Transportation (VDOT) introduced the Superpave design system in 1997, replacing the Marshall method. The original Superpave N_{design} compaction levels were implemented with 86, 95, and 109 gyrations for low, medium, and high traffic levels, respectively. In 2002, the design gyrations number was reduced to 65 to increase binder content and durability. Maupin (2003) conducted multiple laboratory tests and concluded that an increase in asphalt content would result in improved mix properties. The addition of more binder should improve the resistance to cracking without detrimental effects on rutting performance (Maupin & Diefenderfer, 2006). Katicha and Flintsch (2016) evaluated the impact of gyration levels and binder contents on the rutting and cracking performances of dense-graded mixtures using the unconfined Flow Number test and indirect tension strength test. The results showed that binder content could be increased by reducing the number of gyrations from 65 to 50 and reducing the design air voids (i.e., VTM) and that the amount of binder affected the flow number test significantly for all mixtures and the indirect tension strength test for some mixtures.

In addition to the changes in the design gyrations number, many state agencies started including additional specification criteria based on the test results obtained with devices such as the Asphalt Mixture Performance Tester (AMPT) and the Asphalt Pavement Analyzer (APA). Currently, the suitability of performance tests to support mix design is part of the balanced mix design framework, in which the mix performance is evaluated at the design stage both in terms of cracking and rutting resistance (West et al., 2018). Khosla and Ayyala (2013) studied asphalt mixes with different asphalt contents and design gyration levels of 50, 75, 100 and 125. The authors evaluated the balance between rutting and fatigue performance, recommending an intermediate N_{design} value of 85. Aguiar-Moya et al.

(2007) evaluated the performances of mixes designed with 50, 75, 100, and 125 gyrations and found that the interval between 55 and 85 gyrations is the optimal level for the performance of the asphalt mixes.

In 2015, VDOT started an Accelerated Pavement Testing (APT) program at the Virginia Tech Transportation Institute, employing a Heavy Vehicle Simulator (HVS) as its technological centerpiece. Two test lanes (lane 3 and 4) in this APT facility were designed to study the field performance of optimized dense-graded surface mixtures with different gyration levels and binder contents. The two lanes had the same structure with respect to the thicknesses and materials, with the only difference being in the surface layer mix. The surface mixture of lane 3 was designed with 50 gyrations, while lane 4 was designed with 65 gyrations.

3.3 OBJECTIVE

The objective of the study was to compare the performance of two dense-graded surface mixtures, mainly with respect to rutting resistance. One mixture was traditional, acting as the control mix, and one was optimized in order to enhance durability. The two mixes were designed using different design gyration levels (65 for the traditional, 50 for the optimized).

The two testing tools used to evaluated the rutting performance were:

- Laboratory testing – The two mixes were characterized and tested with the AMPT to determine their dynamic modulus and flow number and with the APA.
- APT – To monitor the performance of the two mixtures over time laser profiler, Multi-Depth Deflectometer (MDD), and pressure cells were used.

3.4 MATERIALS LABORATORY CHARACTERIZATION

Since the only difference between lane 3 and 4 was the surface layer, the laboratory testing focused on the material characterization of the surface mixtures.

The primary volumetric properties of the two mixtures are reported in Table 14. In order to better characterize these mixtures, the corresponding asphalt binder film thickness was calculated. It should be noted that a PG 64-22 asphalt binder from the same supplier was used to produce both asphalt mixtures.

Table 14 Volumetric Properties of the Evaluated Mixes

Mixture ID	Lane 3	Lane 4
Composition		
RAP (% by weight)	26	26
PG of Virgin Binder	64-22	64-22
Property		
Gyration Level	50	65
AC (%)	5.9	5.5
G_{mm}	2.478	2.482
G_b	1.03	1.03
VTM (%)	2.9	2.6
VMA (%)	15.8	15.3
VFA (%)	81.5	83.0
Dust proportion	1.17	1.11
G_{mb}	2.405	2.418
G_{sb}	2.696	2.698
G_{se}	2.699	2.701
P_{ba}	0.04	0.04
P_{be}	5.53	5.39
Asphalt film thickness (microns)	9.4	9.4

The composition of both the control mix (lane 4) and the optimized mix (lane 3) is reported in Figure 41, which illustrates how the optimized mix was characterized by a higher asphalt content and a lower natural sand content.

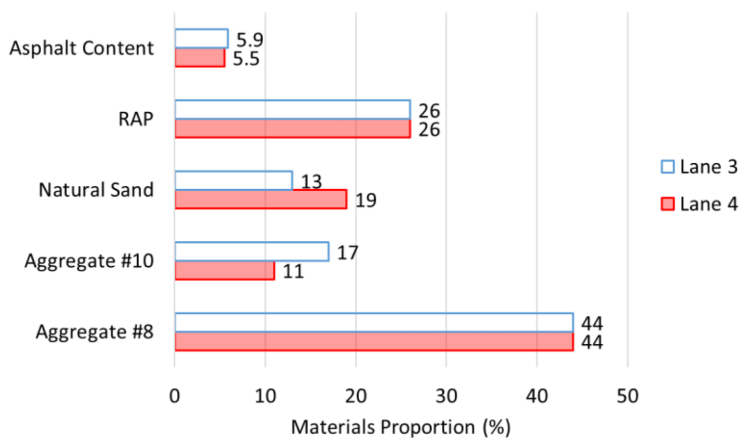


Figure 41 Composition of Mixes

The fundamental engineering properties of the asphalt mixtures were characterized using specimens cored from the testing lanes. Specifically, the dynamic modulus and the flow number tests were performed on small-sized specimens with the AMPT in accordance with AASHTO T-378. The results of the dynamic modulus test are presented in Figure 42 and the results of the flow number test are given in Figure 43.

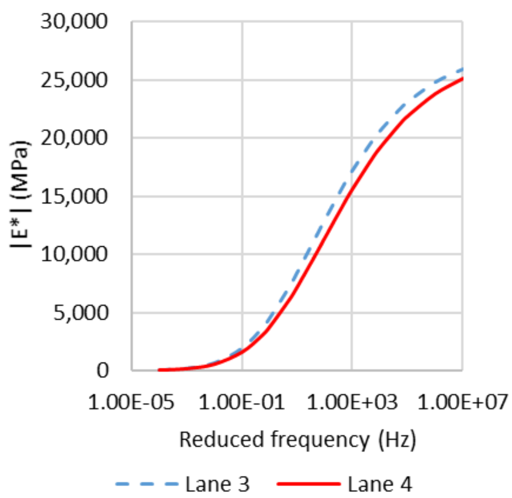


Figure 42 Dynamic Modulus Test Results

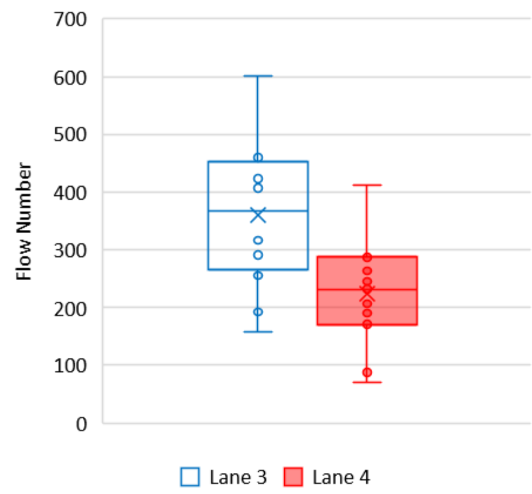


Figure 43 Flow Number Test Results

Figure 42 shows that the optimized surface mixture in lane 3 had a higher modulus than that in lane 4. With respect to the flow number, Figure 43 shows that the optimized mix went into the tertiary zone with a larger number of load cycles compared to the traditional mix, which indicates that the mixture from lane 3 demonstrated a better rutting resistance than lane 4. The results obtained from the two tests were consistent though unexpected. Surface mixtures with higher design gyration levels are expected to have a better rutting resistance. The overall gradation of the mix, particularly the sand content, was believed to be the possible reason for such behavior.

To further check the rutting performance, full depth cores were tested with the APA. The results are shown in Figure 44.

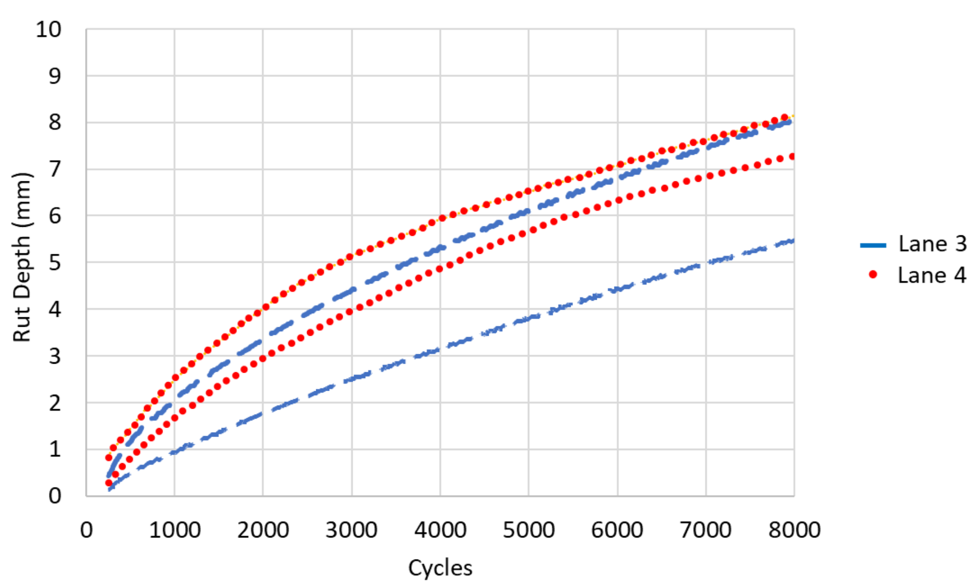


Figure 44 APA Test Results of Surface Mixtures

The optimized mixture (lane 3) had an average rutting depth of 6.81 mm and the traditional mixture (lane 4) had an average rutting depth of 7.75 mm. Although the difference is not statistically significant, the traditional mixture presented a rutting depth 13.8% higher than lane 3. This result was consistent with the dynamic modulus and flow number tests.

3.5 APT FACILITY

The layout and structure of the two test lanes are presented in Figure 45, Figure 46, and Figure 47. The structure was built over a 68.6-cm subgrade layer placed over a rigid foundation. Both lanes featured a 17.8-cm 21-B aggregate subbase, a 10.2-cm IM-19.0 mm (normal maximum aggregate size) intermediate layer, and a 7.6-cm 9.5-mm (NMAS) dense-graded surface layer. The surface mix in lane 3 was designed using 50 gyrations while the mix in lane 4 was designed using the current standard of 65 gyrations.

One load cell was installed on the centerline of each cell; the locations of these are shown in Figure 46 and Figure 47. In addition, an MDD, which is an integration of several linear variable differential transformers (LVDTs), was installed in cells 3A and 4B in order to keep track of the vertical displacement at various depths within the pavement structure. Because the LVDTs couldn't be too close to each other, a solution that featured two MDDs for each test cell was selected to ensure that all the critical depths were covered, as shown in Figure 46 and Figure 47.

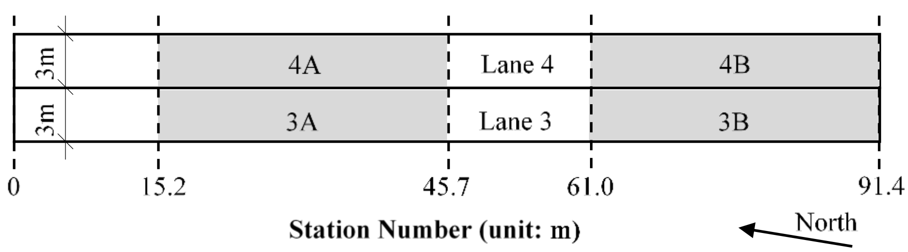


Figure 45 Section Layout

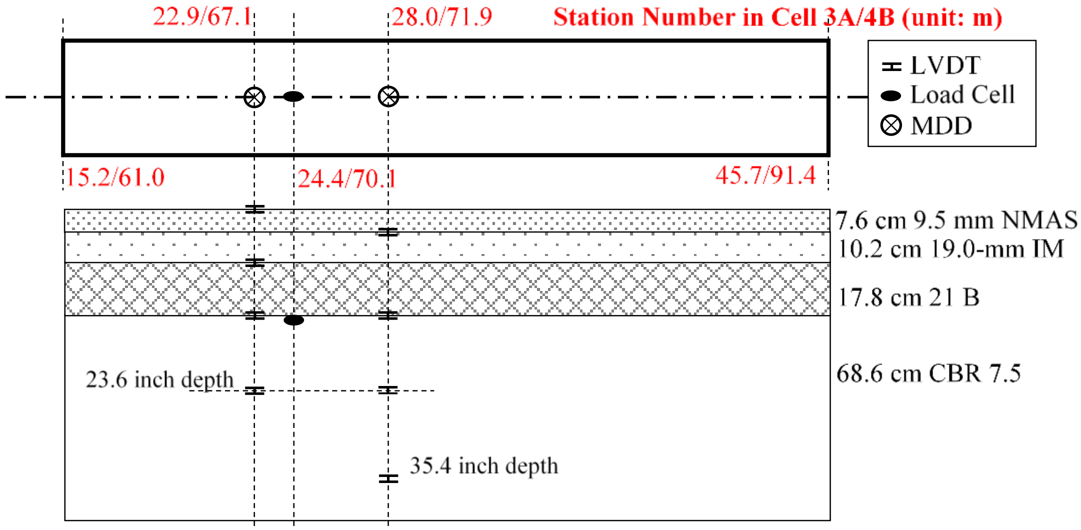


Figure 46 Structure and Instrumentation in Cells 3A and 4B

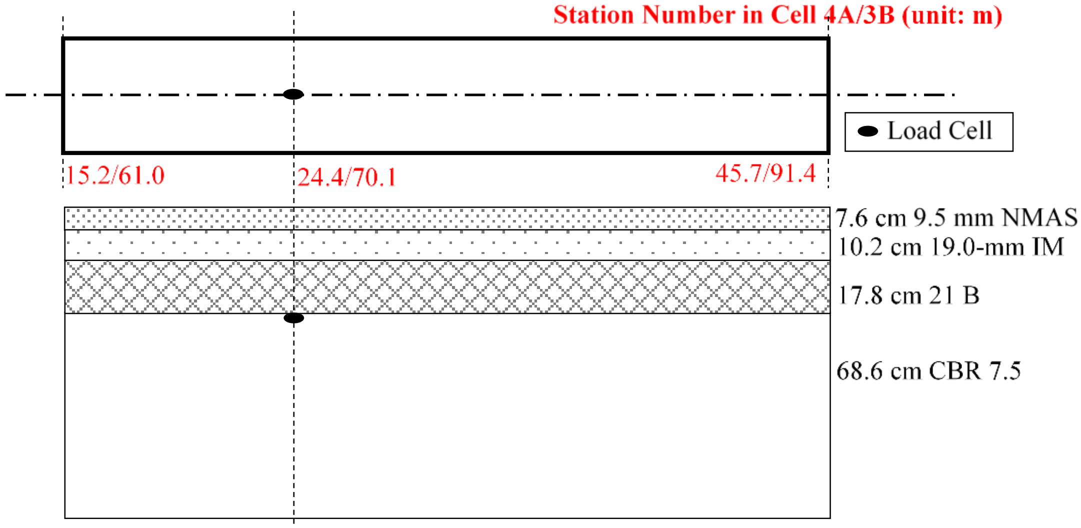


Figure 47 Structure and Instrumentation in Cells 4A and 3B

The heavy vehicle simulator (HVS) used at the Virginia APT facility is a model Dynatest Mark VI. This model has a maximum wheel speed of 20 km/h (± 3.2 km/h) for loads that range from 30,025 kN to 100,085 kN. It can achieve 24,000 bi-directional passes or 12,000 uni-directional passes in 24 hours (Cooke, 2015). The unit features an environmental chamber that allows maintaining a constant temperature at the loaded area. The pavement surface is heated with infrared heaters located along the edge of the test lane within the environmental chamber.

A laser profiler mounted on the HVS carriage was used on a daily basis to scan the pavement surface and measure the vertical permanent deformation at the pavement surface. The rut depth measurements were collected across the full width of the wheel path for a distance of 203.2-cm (101.6 cm inches on either side of the center of the wheel path) and along the length for a distance of 5.4-m.

The spacing of the measurements was 4 inches in the longitudinal direction and 10.2-cm in the transverse direction.

3.6 METHOD

The four test cells were loaded one at a time through a designed loading experiment. The two test cells in each lane were considered as replicates. During each test, the HVS operated continuously with the exception of regular daily maintenance and occasional repairs. The temperature of the surface layer was held at 40°F as monitored by a thermocouple embedded at a depth of 5.1-cm from the surface. The loading sequence consisted of applying a number of passes at progressively higher load levels of 40 kN (9,000 lbf), 53.4 kN (12,000 lbf), and 66.7 kN (15,000 lbf). The loading was applied through a dual tire system, inflated at 7.58 bar (110 psi). The assembly was running uni-directionally at a constant speed of 6.44 km/h (4 mph). The 40 kN (9,000 lbf) load level was intended to simulate half of an 80 kN (18,000 lbf) standard axle load. The wheel loads were transformed into equivalent single axle loads (ESALs) using Equation 1:

$$ESALs = \left(\frac{wheel\ load}{9{,}000}\right)^{4.2} \tag{1}$$

The loading timeline and number of ESALs applied to the pavement within the testing period are provided in Table 15.

Table 15 Loading Timeline for Lane 3 and Lane 4

Cell	Start Date	End Date	# of passes	# of ESALs
3B	1/9/2017	3/7/2017	273,110	704,163
4B	3/20/2017	5/13/2017	278,274	641,141
4A	5/24/2017	7/31/2017	295,131	850,014
3A	8/12/2017	10/7/2017	296,393	829,029

3.7 INSTRUMENTATION RESPONSE

3.7.1 Pressure

The pressure response from the test cells was recorded at each one of the three load levels (40, 53.4, and 66.7 kN). Cell 3B was tested during a continuous extreme snowstorm, and the measurements from the pressure cell were possibly dominated by the changing underground water table instead of the wheel load. For the sake of clarity, only the pressure responses from cells 3A, 4B, and 4C were compared, as shown in Figure 48.

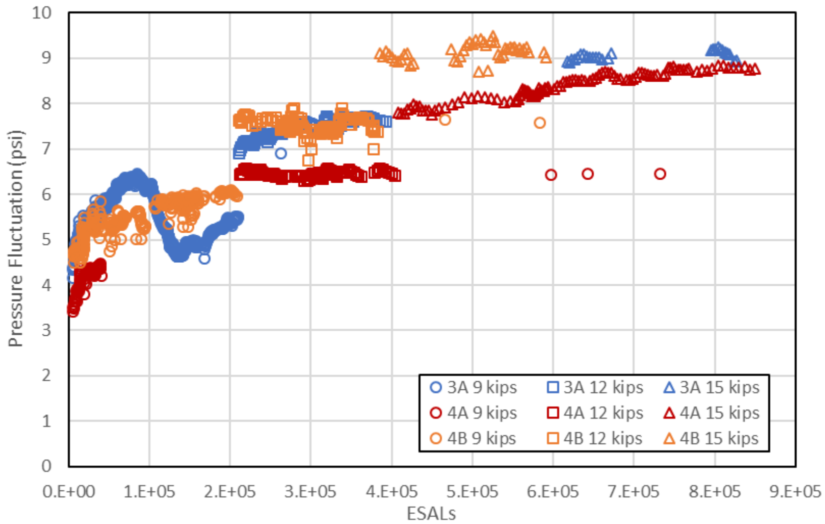
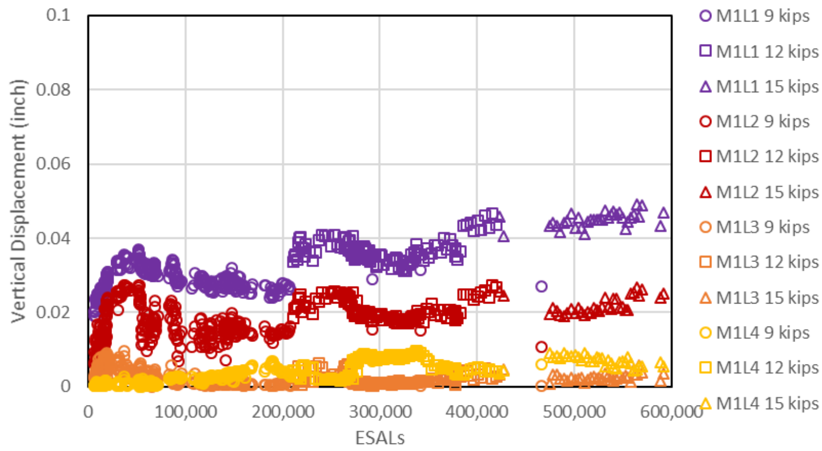


Figure 48 Pressure vs. ESALs in 3A, 4A and 4B.

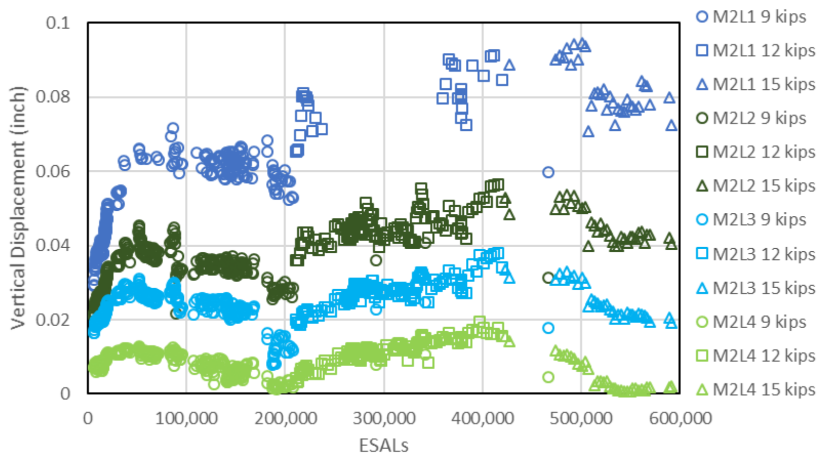
It is possible to observe that all three the test beds went through a consolidation process at the beginning of the experiment (from approximately 0 to 40,000 ESALs), and then stayed at constant level under each wheel load. The pressure fluctuation increased as the wheel load increased, but not proportionally. The pressure values fluctuated for two reasons: (1) the noise of pressure measurements was relatively high because the pressure cells were embedded on top of the subgrade layer, which was 380 mm (14 in) deep, and at that depth, the stresses are relatively small; (2) the pressure measurements were probably affected by a fluctuating underground water table, which changed with the extreme weather in 2017. For example, the first third of the experiment on section 3A saw frequent periods of precipitation, and the pressure measurement was completely determined by the underground water table. When the wheel load on 3A was increased to 12 kips, the weather was consistently clear, and then the corresponding pressure was constant for each wheel load, undisturbed by other factors.

3.7.2 Vertical Displacement

The displacement at the layer interphases at multiple depths was measured via the LVDTs integrated into the MDDs (Xue et al., 2020). The measurements of LVDTs were found to be sensitive to the water table within the pavement structure. For example, the MDD 1 in cell 3A was seriously affected after extreme precipitation and could not provide any meaningful signals. The displacement fluctuations from the two MDDs (eight LVDTs in total) in cell 4B are presented in Figure 49. The dynamic displacements at different depths within the pavement structure changed following a similar trend as the wheel passes accumulated. In addition, it was possible to observe that, as the location was closer to the pavement surface, the displacement increased. Also, the fluctuation over time was likely to increase; as shown by the LVDT M2L1 on the surface of the test bed 4B, it had a much larger variation than the other LVDTs. This behavior was presumably due to the greater level of interaction with the moving wheel and the environment compared to the other LVDTs.



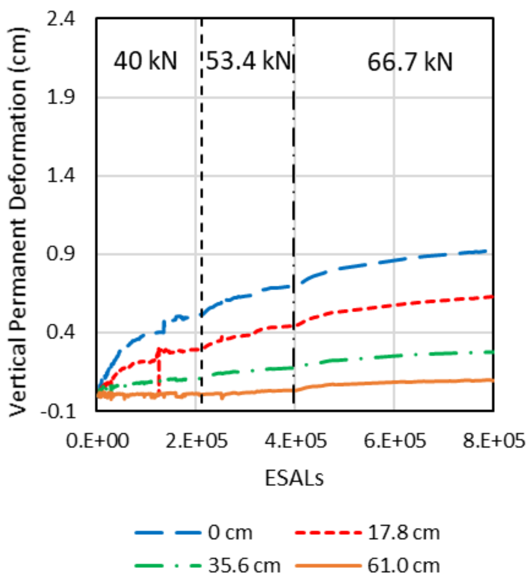
(a) MDD 1



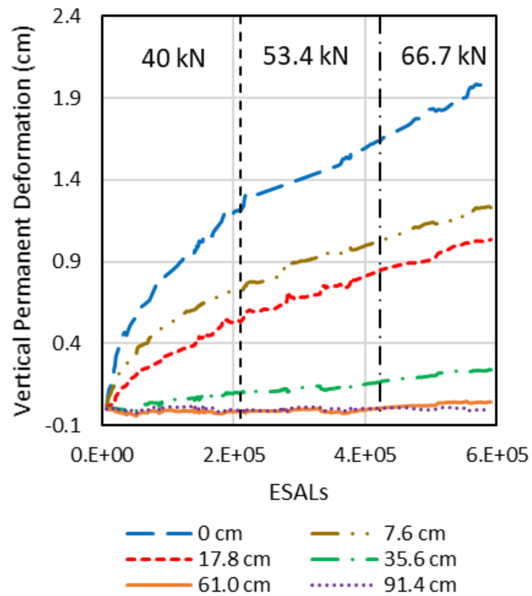
(b) MDD 2

Figure 49 Dynamic fluctuations of LVDTs on the MDDs in cell 4B

The permanent deformations of all the functional LVDTs are summarized in Figure 50, which shows how the MDD installed in lane 3 measured lower deformation levels with respect to the top three layers when compared to lane 4. The subgrade, however, recorded similar levels of deformation in both lanes.



(a) 3A



(b) 4B

Figure 50 Vertical permanent deformation at multiple depths measured by MDDs

3.8 RUTTING PERFORMANCE

In order to evaluate the rut depths of the testbeds, two different methods were used:

- The laser profiler mounted on the HVS carriage
- The LVDT on MDD installed at the surface level

The laser profiler recorded successive transversal planes along the longitudinal direction (one every 10 cm). An example of the profiler output is shown in Figure 51.

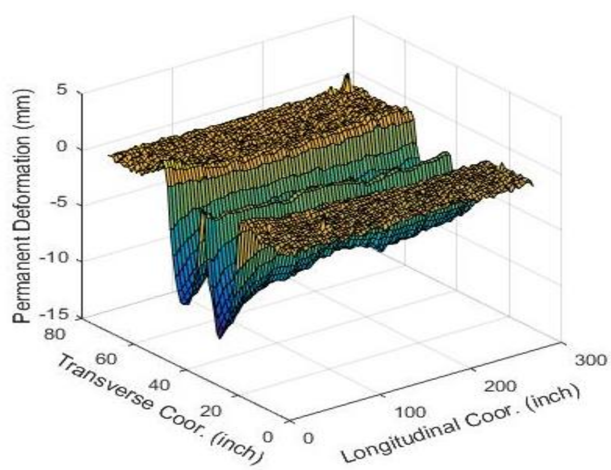


Figure 51 Example of laser profiler measurements (cell 4A, July 31, 2017).

Each scanned point represents the surface vertical permanent deformation (SVPD). The maximum of the SVPDs in a specific transverse plane is denominated MSVPD, while the overall rutting depth includes the lifts and the shape of deformation in the transverse plane (Figure 52).

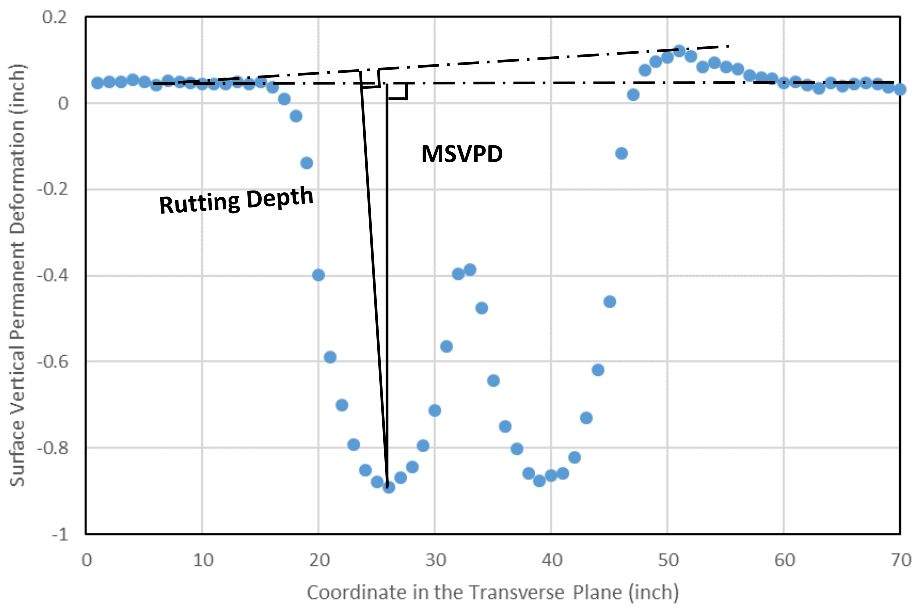


Figure 52. Calculation of rutting depth based on the scanned surface deformation

Compared to MSVPD, rutting depth provides a more appropriate indication of the pavement status and is more commonly used. By consequence, rutting depth was calculated from the profiler measurements for each transverse plane of the cell. To obtain the rutting performance of test cells under constant speed, the rutting depths were averaged for every day throughout the experiments, as shown in Figure 54.

With respect to the LVDT on MDD installed at the surface level, the reference line of each fluctuation represents the position of the LVDT, by subtracting the original location from the current location, the deformation can be computed. Compared to the laser profiler, this method is affected by some limitations when it comes to measuring the deformation of pavement surfaces, as shown by Xue et al. (2020). In fact, as shown in Figure 53, the LVDT only measures the pavement vertical permanent deformation at the specific location of the sensor while the profiler on the other hand scans the whole pavement surface to obtain further information about its rutting progression.

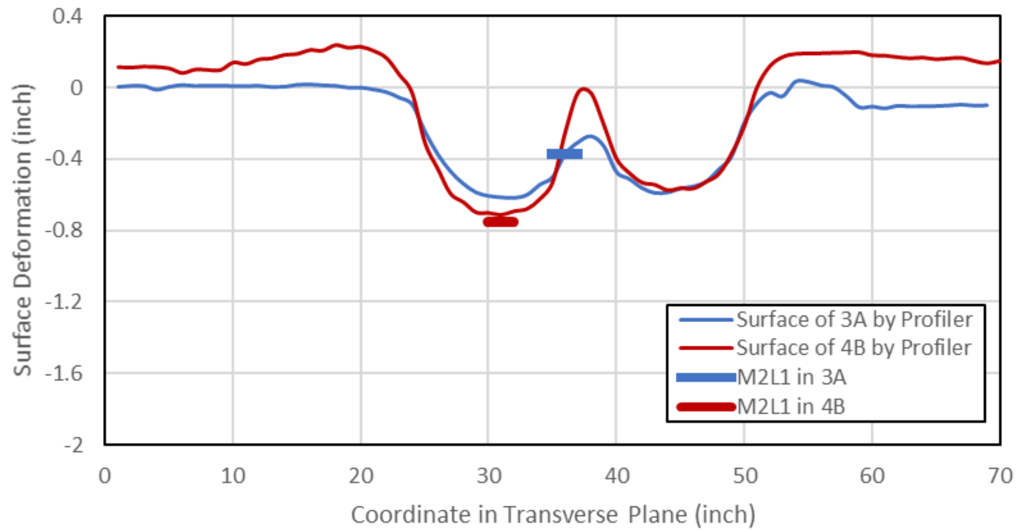


Figure 53 Measurements of profiler and M2L1 on the last days in the transverse plane of M2L1

Figure 53 shows that the locations of the MDD did not always capture the largest vertical deformation even though they were installed to be exactly under the wheel; this is mainly because it is very hard to accurately place the HVS at the expected location.

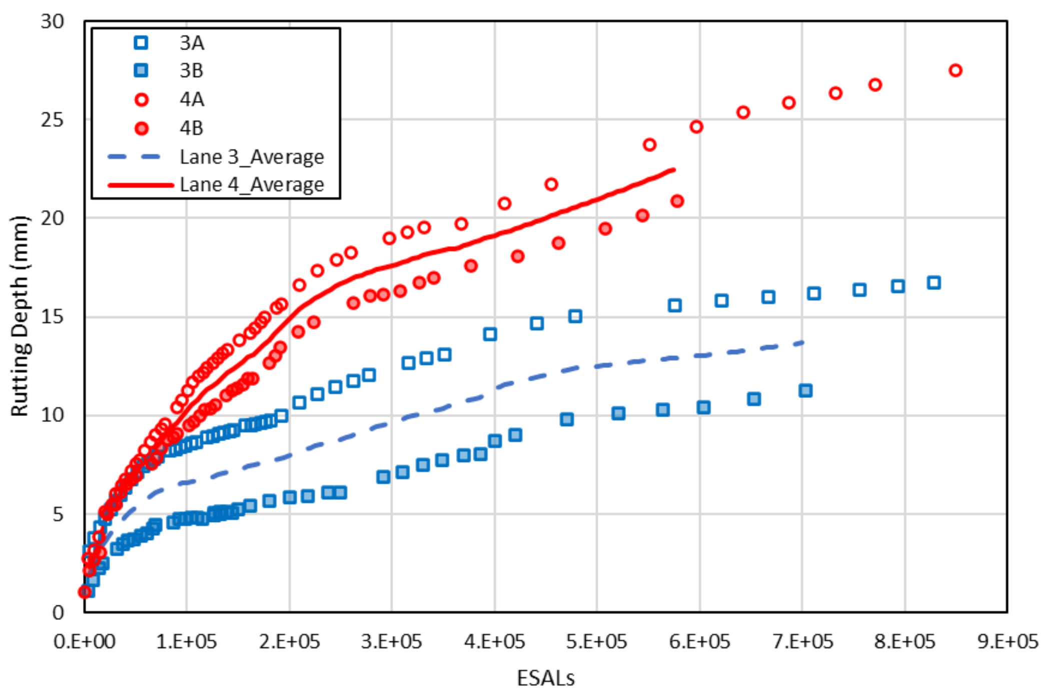


Figure 54 Rutting depths for the four test cells

Figure 54 shows that the shapes of the four curves were similar, but their magnitudes were quite different. The two cells in lane 3 (3A and 3B) had lower rutting depths than the two testbeds in lane 4

(4A and 4B). For example, the average rutting depth of 3A and 3B was about 12.5 mm (0.5 inches) when the accumulated ESAL was 500,000, and the corresponding rutting depth of 4A and 4B was 22.5 mm (0.9 inches). Considering the objective of the experiment, the comparison suggests that the optimized mix have similar or better performance than the traditional mix designed for the higher compaction energy. One possible explanation is that the lower compaction energy allowed the designer to achieve the required air voids with a lower percentage of natural sand and slightly less asphalt and this helped improve the rutting performance.

The test cells were trenched during the forensic investigation after the APT experiments to verify the deformation of each layer (Figure 55). The combination of the thickness measurements with the information relative to the surface deformation provides a way to quantify the final deformation of each layer.

(a) Overview

(b) Details

Figure 55 Pictures of the trenching of cells 3B and 4B

3.9 CONCLUSIONS

This paper documents the results of an APT experiment which aimed at assessing the potential of mix optimization with the objective of improving rutting resistance.

The study compared a control mix and an optimized mix designed using different design compaction energy. The control mix was designed with 65 gyrations, while the optimized mix was designed with 50 gyrations. The optimized mix had a lower natural sand content, which probably increased the aggregate interlock mechanism in the mix.

The testing showed no indication that the optimized mixes would have rutting problems, supporting the implementation of the reduction of the design compaction energy level. The optimized mix exhibited a similar or superior rutting resistance in the laboratory, in the full-scale setting, and in the forensic investigation.

These results also show that small differences in the aggregate structure, such as the different content of natural sand in the mixes studied, can greatly affect the mix performances.

3.10 RECOMMENDATIONS

Based on the results from this study, the following recommendations were made:

- The statewide implementation of the reduction in the design compaction energy should be pursued by the VDOT's Material Division. Both the laboratory and APT results confirmed that rutting resistant mixes could be designed at the 50-gyration compaction level.
- MDDs are valuable supporting tools of an APT experiment, since they allow to measure the vertical deformation within a pavement structure at multiple depths.

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CHAPTER 4 – APPLICATION OF BALANCED MIX DESIGN METHODOLOGY TO OPTIMIZE SURFACE MIXES WITH HIGH-RAP CONTENT

Fabrizio Meroni

Graduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Gerardo W. Flintsch, Ph.D., P.E.

Professor, Charles Via, Jr. Department of Civil and Environmental Engineering, Virginia Tech
Director, Center for Sustainable Transportation Infrastructure,
Virginia Tech Transportation Institute

Brian K. Diefenderfer, Ph.D., P.E.

Principal Research Scientist
Virginia Transportation Research Council

Stacey D. Diefenderfer, Ph.D., P.E.

Senior Research Scientist
Virginia Transportation Research Council

4.1 ABSTRACT

The most common use of reclaimed asphalt pavement (RAP) is in the lower layers of a pavement structure, where it has been proven as a valid substitute for virgin materials. The use of RAP in surface mixes is more limited since a major concern is that the high RAP mixes may not perform as well as traditional mixes. To reduce risks or compromised performance, the use of RAP has commonly been controlled by specifications that limit the allowed amount of recycled material in the mixes. However, the ability to include greater quantities of RAP in the surface mix while maintaining satisfying field performance would result in potential cost savings for the agencies and environmental savings for the public. The main purpose of this research was to produce highly recycled surface mixes capable of performing well in the field, verify the performance-based design procedure, and analyze the results. To produce the mixes, a balanced mix design (BMD) methodology was used and a comparison with traditional mixes, prepared in accordance with the requirements of the Virginia Department of Transportation's volumetric mix design, was performed. Through the BMD procedure, which featured the indirect tensile cracking test for evaluating cracking resistance and the Asphalt Pavement Analyzer (APA) for evaluating rutting resistance, it was possible to obtain a highly recycled mix (45% RAP) capable of achieving better overall laboratory performance than traditional mixes designed using volumetric constraints while resulting in a reduction in production cost.

Keywords

Pavement Recycling, RAP, Surface Mix, Balanced Mix Design, Laboratory Performance

4.2 INTRODUCTION

When the Superpave mix design system was introduced in 1993, it featured performance tests that supplemented a series of material specifications and volumetric requirements (Cominsky et al., 1994). However, due to the high cost and complexity that the testing required, the majority of state agencies adopted only the volumetric part of the design process. Over the years, as Superpave mixes were produced and constructed, each state adjusted the volumetric requirements to improve field performance. One of the most common concerns relates to the mixes' low asphalt content. The general concern was that the Superpave design resulted in dry mixes that were susceptible to cracking while showing satisfying performance in relation to rutting; to counteract this trend, transportation agencies have modified mix design parameters such as the number of gyrations to allow higher asphalt contents (FHWA, 2010). Also, to supplement the design process, simple performance tests, such as the triaxial dynamic modulus and triaxial static creep tests, were recommended (Witczak et al., 2002).

In parallel to concerns about low asphalt content, agencies started using larger amounts of recycled materials (i.e., RAP and reclaimed asphalt shingles [RAS]). This has been linked to mix deterioration due to cracking. In order to keep the cracking performance under control and allow for the inclusion of recycled materials in the mixes, many cracking tests have been evaluated (Al-Qadi et al., 2015; Mohammad et al., 2012; Yin et al., 2018). Moreover, other non-conventional materials, such as additives, rubber, and rejuvenators have been increasingly used and have further changed the typical mixes' behavior. Therefore, alongside cracking tests, evaluation of rutting at the design stage also began, and the idea of a balanced mix design (BMD) took shape (Zhou et al., 2006). BMD generally features at least two performance tests, one for cracking resistance and the other for rutting, to determine how a mix resists various distresses (West et al., 2018).

With consistently stagnant or shrinking budgets, agencies are aiming to include growing quantities of recycled and non-conventional materials in their mixes. However, the use of such mixes leaves open questions about the impact on long-term performances. To reduce those risks, agencies use specifications that limit the maximum percentages allowed. One of the tools that would support the extensive use of these mixes is BMD. Compared to the volumetric-only mix design, BMD allows obtaining an indication of mixture behavior by testing in the laboratory at the design stage. This is expected to provide more confidence about the expected mix performance in the field. For instance, the New Jersey Department of Transportation established a mix design procedure which features at least one of the following three laboratory performance tests: Asphalt Pavement Analyzer (APA) to test rutting resistance, flexural beam fatigue to evaluate fatigue life, and Overlay Tester (OT) to test cracking resistance (Bennert et al., 2014). Meanwhile, the Texas DOT uses a BMD procedure with the Hamburg Wheel-Track Test to evaluate the rutting resistance and OT to test cracking resistance (Zhou et al., 2011).

Currently, in Virginia, the maximum percentages of RAP allowed in asphalt mixtures are 30% for surface and intermediate mixes, 35% for base mixes, and 20% for stone matrix asphalt (VDOT, 2016). As a means to address interest in increased RAP percentages, the Virginia Department of Transportation (VDOT) is aiming to use three performance tests to evaluate performance at the mix design stage: indirect tensile cracking test for cracking resistance, APA for rutting resistance, and Cantabro for durability.

4.3 PURPOSE AND SCOPE

The main purpose of this research was to produce high recycled content surface mixes capable of performing well in the field, verify the performance-based design procedure, and analyze the results. To produce the mixes, a BMD methodology was used and a comparison with traditional mixes was performed. In particular, two mixes, which acted as control mixes, were prepared in accordance with the requirements of the VDOT volumetric mix design, which originated from the Superpave system. Two additional mixes were designed following a BMD procedure which featured the indirect tensile cracking test for evaluating the cracking resistance (ASTM, 2019) and the Asphalt Pavement Analyzer (APA) for evaluating the rutting resistance (AASHTO, 2018).

Additionally, the goal was to explore the feasibility of exceeding the limitations of the volumetric mix design, moving towards a performance-based mix design. From this perspective, the results of the laboratory tests would become the main indicators for how to increase the RAP content while obtaining satisfying performances.

4.4 SIGNIFICANCE OF THE STUDY

Today, the most established use of RAP is in the lower layers of a pavement structure, where it has been proven as a valid substitute for virgin materials. For instance, base layers can allow achieving excellent performances while including up to 100% RAP through a cold process in which virgin asphalt is added as a recycling agent (Diefenderfer et al., 2016; Diefenderfer & Link, 2014). On the contrary, the use of RAP in hot asphalt surface mixes is more limited, with a major concern being that high RAP mixes may not perform as well as traditional mixes (Nair et al., 2019). To reduce risks of compromised performance, the use of RAP has commonly been controlled by specifications that limit the allowed amount of recycled material in the mixes (Al-Qadi et al., 2015).

Compared to the bottom layers, the surface mix has to better withstand higher stresses and aging. Current specifications typically require that the mix components of surface layers (both aggregates and virgin binder) need to be composed of higher percentages of virgin materials as they are thought to be of higher quality. Being able to include greater quantities of RAP in the surface mix while maintaining satisfying field performance would support the achievement of potential cost savings for the agencies (Copeland, 2011) and environmental savings for the public. A performance-based mix design procedure, such as BMD, is one of the most promising tools that agencies can use to expand the use of RAP and reduce the risks of using high recycled content mixtures. In fact, BMD allows control of the recycled mixture resistance to distresses and compares it to mixes having lower recycled contents at the design stage. It must be considered that in the light of an initial low cost, it is the mixture's ability to resist distresses over its life cycle that eventually determines the overall costs of a recycled pavement and its potential to achieve economic savings over its entire life span.

4.5 METHODOLOGY

First, the research team defined a high-RAP-content control mix which met the current VDOT specifications. The mix, which will be referred as 30-Superpave, contained 30% RAP by weight of total mixture, which is the current upper limit for surface mixes (VDOT, 2016). An additional control mix, referred as 45-Superpave, was designed to match the aggregate structure and binder content of 30-Superpave but at a RAP content of 45% by weight of total mixture.

Based on the performance information (obtained through the indirect tensile test and APA rut test) of the afore-mentioned mixes, changes to the mix composition were made. Two more mixes were designed with a different aggregate structure, which did not meet the VDOT specification requirements but met the current tentative BMD thresholds (2019). The new mixes contained 30% and 45% RAP by weight of total mixture and were designated as 30-BMD and 45-BMD, respectively. The optimum binder content of the two mixes was determined based on balancing cracking and rutting resistance. The changes in the mix composition and binder content were determined with the goal of improving the mixes' performance, even if the final properties of the mix did not satisfy existing volumetric requirements.

4.6 REVIEW OF HIGH RAP MIX DESIGN

The inclusion of RAP in hot mix asphalt designed using the Superpave procedure was formalized through NCHRP Project 9-12, in which a three-tier system was developed. The system was based on the properties of the hardened RAP binder and blending charts were developed for high RAP contents

(McDaniel et al., 2000; West et al., 2013). Through the following NCHRP Project 9-46, which was aimed at improving the recycling practice, the design of high RAP mixes was guided by the RAP binder ratio (West et al., 2013). In both cases, recovery and grading of the aged binder were key steps in the design process.

To include higher RAP quantities, recycling agents such as rejuvenators and softening agents have also been introduced. Rejuvenators are chemical agents capable of restoring the physical and chemical properties of the old binder. They differ from softening agents, which lower the viscosity of the aged binder (Roberts et al., 1991). It is important to select the right rejuvenating agent, based on its compatibility with the aged binder (Al-Qadi et al., 2007). A lot of uncertainties still remain over the properties of the rejuvenated RAP binder; therefore, RAP has often been used in lower-level applications without fully exploiting the value of the asphalt binder available in the RAP (Zaumanis & Mallick, 2015).

The inherent variability of RAP, the difficulty of defining the interaction level of the virgin binder, recycled binder, and additives, and the open questions concerning rejuvenators have led to the definition of new mix design methods (West et al., 2018). The BMD concept was introduced to design mixes as the best compromise between rutting resistance and cracking resistance (Zhou et al., 2006). While increasing the confidence in the final properties of the mix, BMD is not necessarily linked with the evaluation of the RAP binder grade and the interaction level between the recycled binder and virgin binder.

Laboratory performance tests must be associated with appropriate management of the RAP. It is fundamental that the material used during the design stage maintains consistent properties throughout the production process. To support the construction of highly recycled mixes, RAP needs to be appropriately processed, crushed, and screened. RAP stockpiles need to be regularly sampled for quality control to ensure that consistent gradation and binder content are maintained (Nady, 1997).

4.7 MIXES PROPERTIES

All the mixes evaluated had a nominal maximum aggregate size (NMAS) equal to 9.5-mm and the selected binder performance grade (PG) was 64-22. The gradation curves of the mixes are reported in Figure 56, while the volumetric parameters of the control mixes are reported in Table 16. It is possible to observe how both control mixes were designed accordingly with VDOT’s volumetric requirements. The 30-BMD and 45-BMD mixes were instead designed with the goal of obtaining a coarser mix allowing less passing at the sieve sizes of 4.75-mm and 2.36-mm.

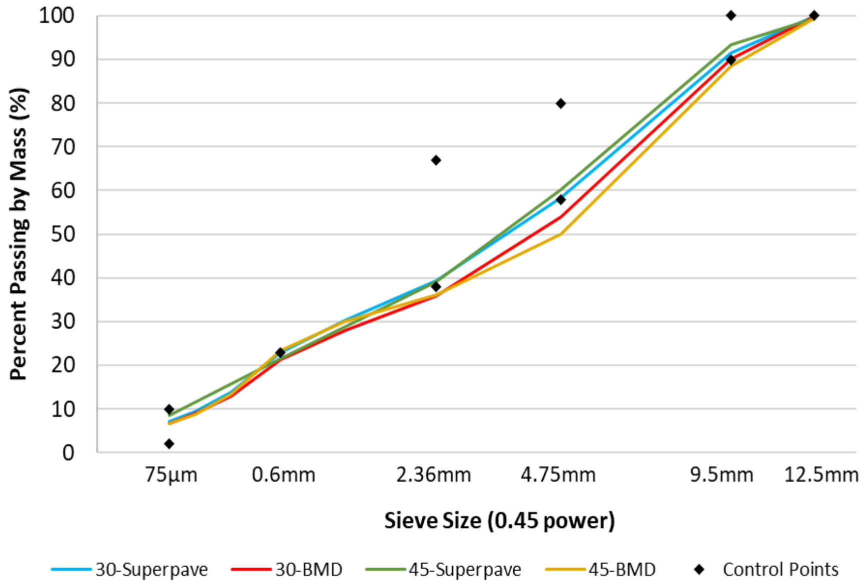


Figure 56 Gradation curves

Table 16 Control mixes properties

	30-Superpave	45-Superpave	Criteria (VDOT, 2016)
G_{mm}	2.492	2.518	-
VFA (%)	77.2	77.2	75-80
VMA (%)	16.5	16.6	16 (min.)
Fines/Asphalt Ratio	1.2	1.3	0.7-1.3

4.8 LABORATORY AGING OF THE SPECIMENS

To analyze mix performance at the mix design stage, appropriate aging needs to be applied to the specimens produced in the laboratory. The American Association of State Highway Transportation Officials (AASHTO) recommends 4 hours of aging at 135°C (AASHTO, 2015).VDOT modified this by requiring 4 hours of aging at the mix design compaction temperature. However, there is still considerable discussion in the community regarding the most appropriate aging protocol to simulate plant-produced material. To determine the most appropriate way of aging the samples during the study, plant-produced samples of the control mix were collected as a reference to compare the cracking resistance results due to the different aging techniques. Both 2 hours and 4 hours of aging were evaluated. The results are shown in Figure 57.

Based on the comparison with specimens taken directly from the plant, the 2-hour aging produced more similar results, while the 4-hour aging significantly reduced the laboratory performances of the mixes. For this reason, the aging applied to specimens during the study was 2 hours at compaction temperature.

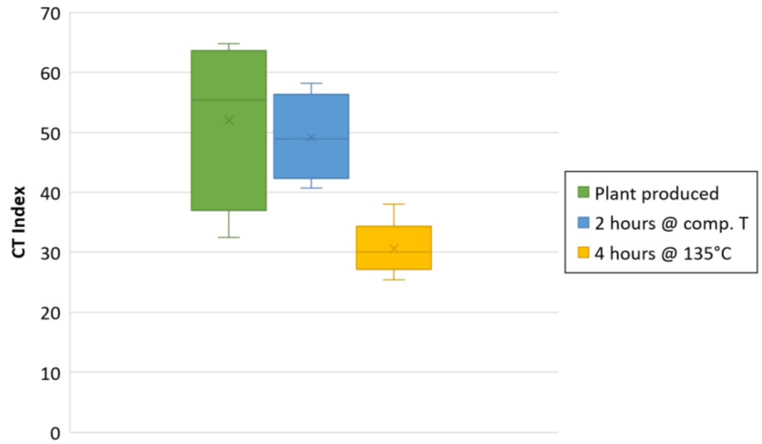
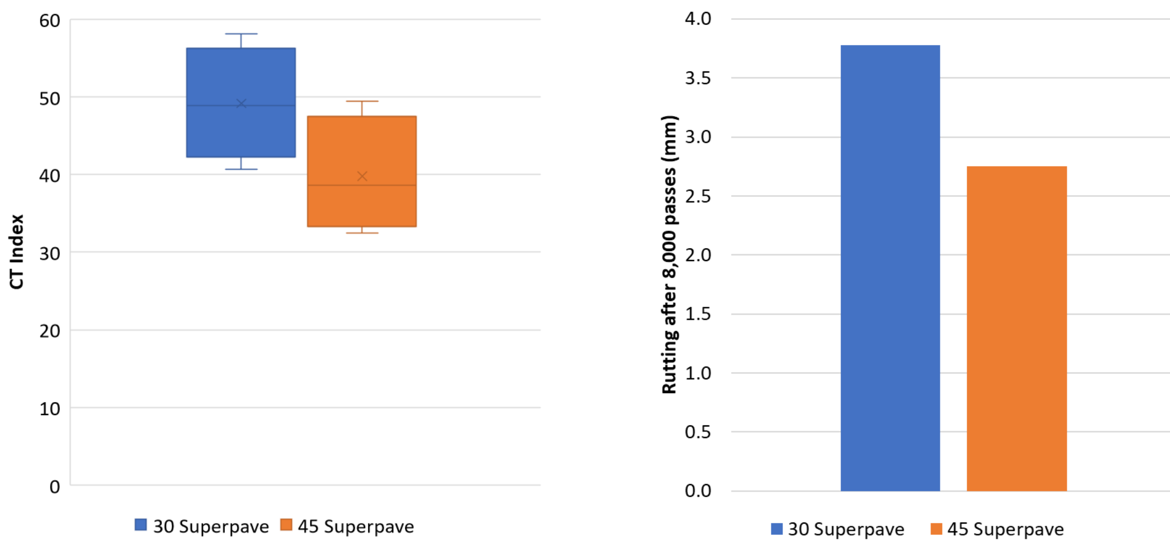


Figure 57 Aging impact on recorded cracking resistance of the control mix (30-Superpave). The middle line of the box represents the median, the x in the box represents the mean (4 specimens per aging type)

4.9 PERFORMANCE OPTIMIZATION PROCESS

The performance evaluation was first conducted on the control mixes, with respect to both cracking and rutting resistance, evaluated through the indirect tensile test and APA, respectively. The results are shown in Figure 58. While there are still no common guidelines on the definition of performance requirements, the VDOT is looking to establish thresholds to identify satisfactory levels of performance in the field. In particular, for the CT Index, the proposed minimum value is 70, while the measured rut depth after 8,000 passes in the APA should be less than 8-mm.



(a) **(b)**

Figure 58 CT Index (a) and APA (b) results for control mixes

Both mixes met the volumetric specifications; however, Figure 58 shows that the cracking resistance minimum was not met while the rutting resistance test result was well below the proposed maximum limit. This behavior has been associated with Superpave mixes (Watson, 2003). Even with a limited impact, the inclusion of 45% RAP in the mix corresponded to lower cracking resistance and higher rutting resistance. These trends have been traditionally associated with high RAP contents and have also been confirmed by various studies presented in the literature (Al-Qadi et al., 2012; Kim et al., 2007).

To improve the mixes cracking resistance and obtain better overall performance while increasing the RAP content, a different aggregate structure was studied and tested at different asphalt contents. Figure 59 and Figure 60 show the performance test results for the mixes containing 30% and 45% RAP, respectively, which do not meet VDOT gradation requirements. The shaded areas represent the zones in which the mix fails to meet performance criteria: the orange area highlights rutting depths greater than 8 mm while the blue area shows CT Index values lower than 70.

The definition of optimum binder content was based on balancing both performance requirements. As expected, the impact of increasing the asphalt content was noticeable with regard to both cracking and rutting. As the mixture binder content increased, the mix became less stiff allowing it to have higher cracking resistance. The mix containing 30% RAP showed the potential of reaching very high values of cracking resistance, while the 45% RAP mix appeared to be more limited. This limitation can be explained by the partial contribution of the RAP binder: when comparing the two mixes with the same binder content design value, the partial contribution of the RAP binder is going to appear with greater impact on the 45% RAP mix. The 45-BMD mix showed an increase in cracking resistance with respect to increasing binder content while the rutting susceptibility is not consistent with increasing binder content. Also for 30-BMD, higher binder contents corresponded to higher cracking resistance, with a measured maximum CT Index value at 7% AC more than ten times larger the maximum measured at 6% AC. The rutting resistance for 30-BMD showed a slight deteriorating trend however, the rutting resistance was well below the 8-mm limit in all cases.

The test results indicated that to achieve satisfying performances, the minimum design AC for 30-BMD was approximately 6.2%, while for 45-BMD it was 6.5%. The design asphalt contents were selected based on the compliance with the minimum CT Index criteria plus a safety factor. Both cases represented a meaningful improvement in terms of performance when compared to the control mixes.

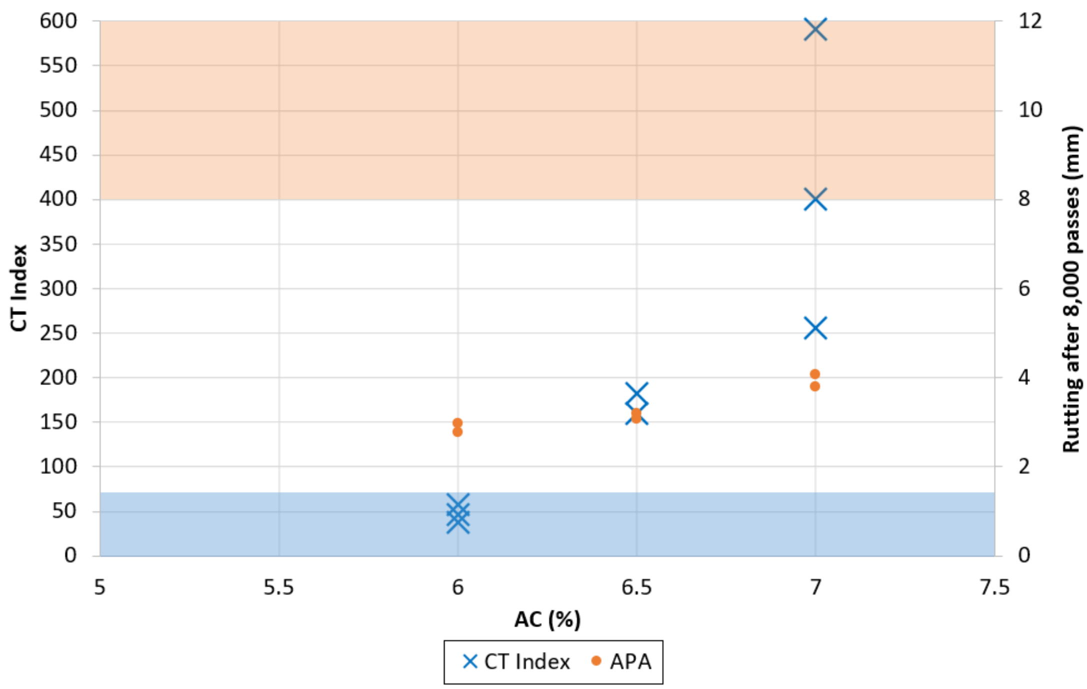


Figure 59 Performance tests on mix 30-BMD

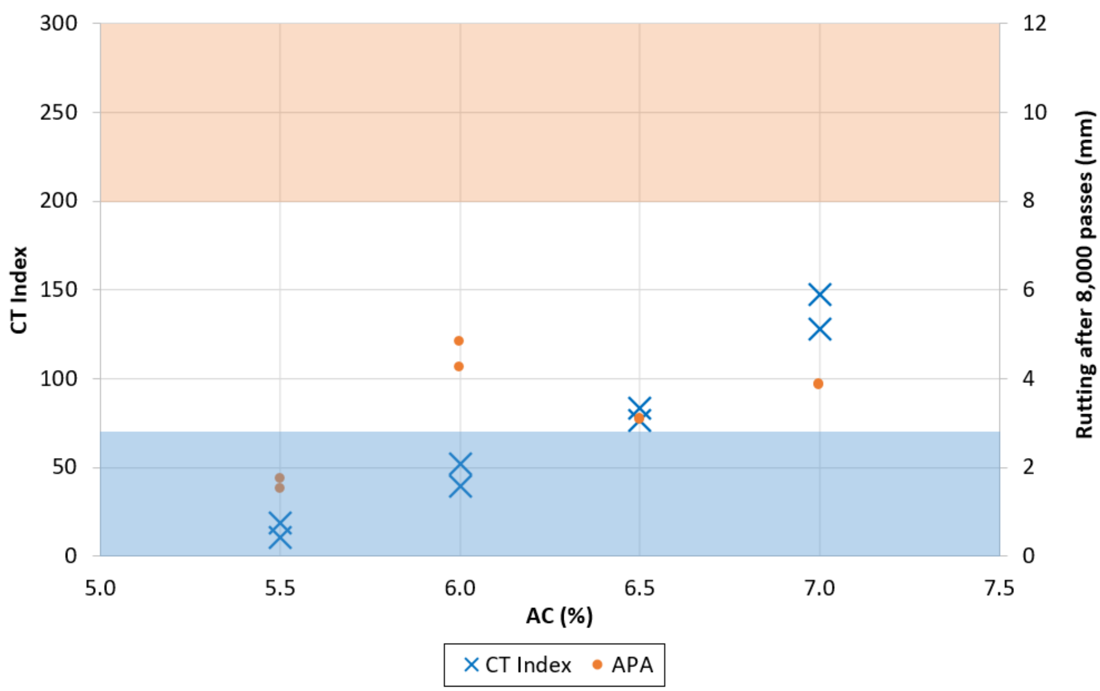


Figure 60 Performance tests on mix 45-BMD

4.10 COST COMPARISON

To further examine the feasibility of implementing the BMD mix design process rather than the volumetric requirements, a cost analysis was conducted to evaluate the impact on the mixes' production cost. For both mixes 30-BMD and 45-BMD, the design AC was selected so that it was possible for the BMD mixes to outperform the control mixes and achieve acceptable levels of cracking and rutting resistance with respect to VDOT proposed requirements. A summary of the mixes composition is shown in Table 17.

The material costs and final estimates are summarized in Table 18. To calculate the costs of the various mix components, representative Virginia statewide averages were used and assumptions on mixture constituents costs were used. In particular, the aggregate costs were discounted by 50% to account for their use in asphalt mixture production. For the RAP, while the purchase cost is considered equal to zero, processing-related costs were considered. For this analysis, the quantity of required virgin binder was calculated as the design AC minus the quantity of binder provided by the RAP.

In general, as shown by the comparison between the two control mixes in Figure 61, the increased use of RAP resulted in a reduction of virgin aggregates and binder. However, to pass the requirements of the laboratory optimization process, the mix 30-BMD required more virgin binder content than the control mix, resulting in a higher production cost (+6%) when compared to the control mixture with the same RAP content. With respect to the 45% RAP optimized mix, it was possible to achieve significant savings (7%) compared to the 30-Superpave mix even if the design binder content of the BMD mix was higher.

Table 17 Breakdown of mixes composition

Mix Component	**Mix**			
	30-Superpave	**45-Superpave**	**30-Optimized**	**45-Optimized**
Aggregate No. 8	43	39	50	40
Aggregate No. 10	14	13	10	0
Natural sand	13	3	10	15
RAP	30	45	30	45
Asphalt Content [a]	5.9	5.9	6.2	6.5

[a] Sum of virgin binder and binder included in the RAP.

Table 18 Mixes cost comparison

Mix Component	Cost ($/ton)	Mix			
		30-Superpave	45-Superpave	30-Optimized	45-Optimized
Aggregate No. 8 [(a)]	22.50	9.23	8.44	10.68	8.60
Aggregate No. 10 [(b)]	15.00	2.06	1.96	1.49	0.12
Natural sand [(b)]	15.00	1.92	0.55	1.49	2.23
RAP [(c)]	5.00	1.41	2.12	1.41	2.10
Binder cost [(d)]	528.55	22.38	17.98	23.99	21.23
Cost per US ton ($)		37.00	31.05	39.06	34.29
Cost per lane ($/mile) [(e)]		895.38	751.40	945.22	829.86

[(a)] Adjusted Virginia statewide averages (Jan 1, 2018 through Feb 1, 2020).

[(b)] Adjusted Virginia statewide averages (Nov 1, 2016 through Dec 1, 2018).

[(c)] The RAP purchase cost is assumed equal to zero, the cost listed is related to the RAP processing phase.

[(d)] VDOT asphalt price (PG 64S-22, April 2020). The calculated cost is relative only to the virgin binder.

[(e)] Assumption of 165 lb. of mix per yd^3, with a layer thickness of 1.5 in.

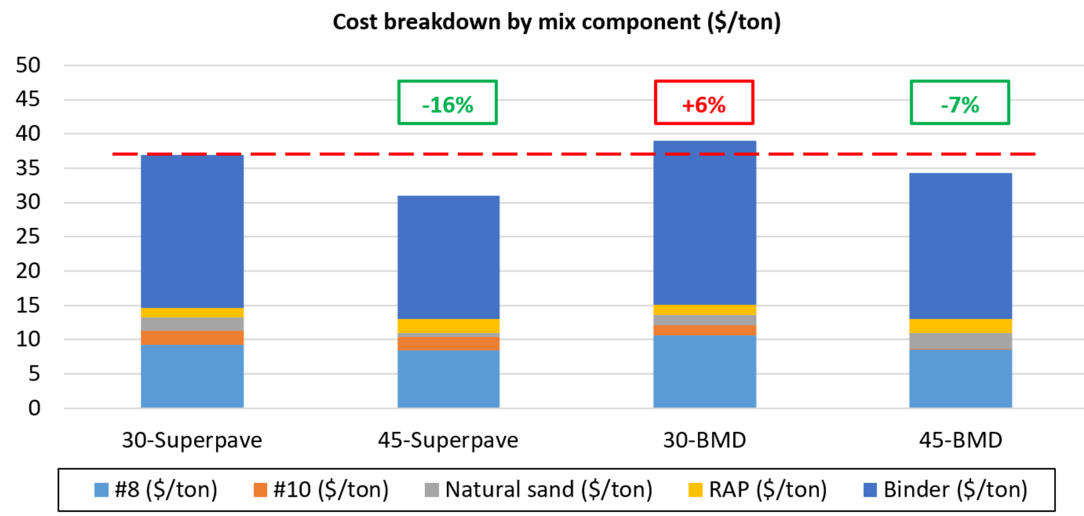


Figure 61 Mix cost comparison

To further examine the economic feasibility of the BMD process, a simplified life-cycle cost analysis (LCCA) was conducted. The LCCA examined the impact of production and operating (maintenance) costs over a period of 30 years. The control mix 30-Superpave was used as a reference with a service life varying between 8 and 12 years. At the end of the service life, mill and overlay operations were planned for all mixes. The scheduled operations over the course of the 30-years analysis period were discounted at a rate of 4%. Different scenarios were evaluated for both the optimized mixes, which supposedly would achieve extended service lives, thus requiring fewer maintenance operations. Four different scenarios of service life variation were evaluated, in which the cost change of each optimized mix was analyzed from a decrease of 1-year to an increase of 2-years. The results are showed in Table 19 (30-BMD) and Table 20 (45-BMD).

Table 19 Cost increase/reduction of 30-BMD with respect to 30-Superpave (%)

Service Life 30 - Superpave (years)	Service Life Change 30 - BMD (years)			
	-1	+0	+1	+2
8	16.6%	5.6%	-3.2%	-10.7%
9	15.1%	5.6%	-2.6%	-8.4%
10	14.5%	5.6%	-0.7%	-6.2%
11	12.2%	5.6%	-0.3%	-5.5%
12	11.7%	5.6%	0.1%	-4.8%

Table 20 Cost increase/reduction of 45-BMD with respect to 30-Superpave (%)

Service Life 30 - Superpave (years)	Service Life Change 45 - BMD (years)			
	-1	+0	+1	+2
8	2.3%	-7.3%	-15.0%	-21.6%
9	1.1%	-7.3%	-14.5%	-19.6%
10	0.5%	-7.3%	-12.8%	-17.6%
11	-1.5%	-7.3%	-12.4%	-17.0%
12	-1.9%	-7.3%	-12.1%	-16.5%

It is possible to observe how for 30-BMD a life extension of 1-year is enough to present a more convenient solution, with the exception of the scenario in which the reference mix, 30-Superpave, provides a service life equal or greater than 12-years. Instead, 45-BMD allows achieving cost savings even if the service life is 1-year shorter than the control mix (when compared to 30-Superpave's service lives equal or greater than 11-years).

4.11 FINDINGS AND CONCLUSIONS

In this study, it was possible to investigate how the implementation of a BMD system could represent a significant upgrade of the current design practice. Through this performance-based procedure, it was possible to obtain a high recycled content mix, such as 45-BMD, which would provide better overall performance while providing a reduction in production cost when compared to traditional mixes. In addition to the economic savings, the higher RAP content improves the environmental impact of the mix as it uses less virgin materials. Overall, the following conclusions were drawn:

- The use of gradation and volumetric requirements did not guarantee satisfactory performance (in terms of laboratory cracking and rutting resistance) for the 30-Superpave and 45-Superpave mixes, which were designed in accordance with Superpave requirements.
- The mixtures designed using the Superpave gradation and volumetric requirements were outperformed by the selected BMD mixes.
- As expected, for both the control and optimized mixes, the inclusion of higher RAP contents corresponded to lower cracking resistance and higher rutting resistance in the laboratory.

- Even if high RAP contents may require higher asphalt contents to achieve satisfactory cracking resistance, the impact on rutting performance were very limited for the mixtures evaluated in this study.
- Compared to the control mixes, the optimized mixes showed potential economic savings. The 45-BMD mix resulted in lower production costs and presented better laboratory performance than the control mixes. A simplified LCCA showed how a 1-year service life extension would be enough to justify the higher production cost of 30-BMD. Also, the LCCA showed that, when compared to the control mix 30-Superpave, 45-BMD would allow achieving significant savings even if it wouldn't extend the service life.

4.12 RECOMMENDATIONS

Based on the findings of this research, the following recommendations are made:

- The study used two simple performance tests to measure the cracking and rutting resistance of the mixes. However, the selection of appropriate laboratory tests is fundamental for the effectiveness of a BMD approach. In particular, field performance is needed to verify the conclusions that are based on laboratory testing. Techniques like pavement recycling are generally promoted because of their economic savings and environmental benefits they entail. However, only if the field performances are adequate, it would be possible to fully take advantage of the recycling process. If the RAP inclusion results in shorter pavement lifespans or inappropriate performance, the initial benefits would become irrelevant due to the necessity of additional maintenance.
- The RAP binder content affects the mixture's final asphalt content, which may result in the mix being higher or lower than the design AC. If the asphalt content of the RAP changes, the mixes could become under-asphalted or over-asphalted, resulting in poor pavement performance. Therefore, because of its inherent variability, the properties (e.g. gradation and AC) of the RAP taken from the plant stockpiles need to be checked throughout the process. The need to track the sources of the RAP and maintain separate stockpiles should be investigated. Changes in the binder content of the RAP source stockpile and the effects of using multiple RAP stockpiles should be investigated to determine the influence on producing a consistent mixture having high recycled contents.
- Even though part of the aged RAP binder contributes to providing the mix properties , the 45% RAP mix required a higher overall AC percentage than the 30% RAP to achieve the same level

of performance. This is because at higher RAP contents the aged binder contributes to a larger proportion of the mixture properties. For this reason, the use of rejuvenators should be investigated to further optimize the necessary quantity of virgin binder especially at high RAP contents.

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CHAPTER 5 – THREE-LEVEL PERFORMANCE EVALUATION OF HIGH RAP ASPHALT SURFACE MIXES

Fabrizio Meroni

Graduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Gerardo W. Flintsch, Ph.D., P.E.

Professor, Charles Via, Jr. Department of Civil and Environmental Engineering, Virginia Tech
Director of the Center for Sustainable Transportation Infrastructure,
Virginia Tech Transportation Institute

Jhony Habbouche, Ph.D., P.E.

Research Scientist
Virginia Transportation Research Council

Brian K. Diefenderfer, Ph.D., P.E.

Principal Research Scientist
Virginia Transportation Research Council

Filippo Giustozzi, Ph.D., PEng

Senior Lecturer, Civil and Infrastructure Engineering
Royal Melbourne Institute of Technology

5.1 ABSTRACT

To support an increased use of reclaimed asphalt pavement (RAP) and innovative materials, appropriate laboratory performance tests are required. There exist a wide selection of performance tests with varying complexity, reliability, and cost. However, there is no consensus on which tests are best suited to support the design and quality assessment of asphalt mixtures. The main purpose of this effort was to assess the impact of testing complexity on the estimated performance of asphalt pavements through a comprehensive evaluation of four different highly recycled surface mixes produced and placed in the Commonwealth of Virginia. For that purpose, the research team defined a three-level testing framework herein referred to as basic, intermediate, and advanced. Each level was characterized by an increasing degree of complexity and cost and included tests to characterize both the cracking resistance and the rutting resistance of the evaluated mixtures. Under the performance evaluation process, it was possible to investigate the features of the various laboratory tests. Through the review of the theoretical background, the evaluation of the test procedures, and statistical analysis of the results, it was possible to identify the strengths and weaknesses of each test. Based on the test results, recommendations were provided on the design of highly recycled surface mixes. Also, it was possible to provide guidelines to develop appropriate quality assessment criteria and mix design methodology.

Keywords

Pavement Recycling, RAP, Surface Mix, Balanced Mix Design, Laboratory Performance, AMPT

5.2 INTRODUCTION

Currently, a reclaimed asphalt pavement (RAP) material content of 15–20% is becoming a standard practice for the production of asphalt mixtures worldwide (Tarsi et al., 2020). In some countries, such as the Netherlands and Japan, this RAP content percentage has been largely exceeded, while in others, like the United States, the numbers have been relatively stagnant.

A 2019 survey on the use of recycled materials in the U.S. pavement industry reported that the average RAP content in asphalt mixtures was 21.1%, which was exactly equal to the previous year's value (Williams et al., 2020). Compared to 2009, when the National Asphalt Pavement Association conducted the first national survey on pavement recycling, asphalt mixtures have shown significant growth in average RAP content; however, the year-over-year rate of growth plateaued at around 20% in 2014. The common perception that recycled materials have lower quality compared to virgin products was reflected by the definition of strict roadway agency specifications: in 2011 only five agencies allowed 30% or greater RAP content in their mixes (Copeland, 2011). In 2017, the state asphalt pavement associations (SAPAs) confirmed that the limits on RAP content induced by agencies' specifications are still considered the main roadblocks to greater RAP use (Williams et al., 2018).

Other countries instead pushed the recycling boundaries. In the Netherlands, it is common practice to include 50% RAP in most asphalt mixtures, for both base and surface layers (Mohajeri, 2015). Most significantly, in 2008, a new specification system for asphalt mixtures was implemented, in line with the European standards (EN13108 series). In this new scenario, Dutch contractors have the freedom to select their own mix composition and design procedure as long as the final performance requirements are fulfilled. In 2013, Japan reported an average of 47% RAP content (West & Copeland, 2015). The main measures taken by the Japanese industry have included RAP fractionation, moisture control, and use of rejuvenators. In addition, specifications were developed with respect to testing directly on the RAP and the mix at the design stage.

It is clear how one key to increasing recycling levels with confidence has been the use of performance tests on mixes. In fact, in 2018, the introduction of mixture performance testing measures was also identified by the SAPAs as having the most potential to increase recycled materials use (Williams et al., 2019). The use of performance tests was included in the original Superpave framework (Cominsky et al., 1994); however, due to excessive costs and complexity, this part of the Superpave system was never implemented (Button, 2004). The Federal Highway Administration (FHWA) developed, through National Cooperative Highway Research Program (NCHRP) Project 9-19 (Witczak et al., 2001) and Project 9-29 (Bonaquist, 2008; Bonaquist et al., 2003), the Asphalt Mixture

Performance Tester (AMPT). The objective of these projects was to develop a testing machine that was able to perform simpler tests on the mixes to replace the equipment that should have complemented the Superpave mix design system as it was originally conceived (FHWA, 2013). The AMPT is capable of providing engineering properties for mixture evaluation and pavement structural design using specific computer software. In parallel, multiple other simplified test procedures have been developed to evaluate both cracking (Seitllari et al., 2020; Zhou et al., 2016) and rutting resistance (Kandhall & Mallick, 1999; Walubita et al., 2019; Walubita et al., 2012).

Currently, to support the mix design and quality acceptance processes, transportation agencies have a wide selection of tests that vary in complexity, cost, testing time, and required operator training. Many states, such as California (Harvey et al., 2014), Illinois, Iowa, Louisiana, New Jersey (Bennert et al., 2014), and Texas (Zhou et al., 2014) have already developed performance-based mix design systems. At the federal level, NCHRP Project 20-07/Task 406 developed a framework that evaluated various approaches to incorporate performance testing and criteria at the design stage and implement balanced mix design procedures (R. West, 2018).

5.3 OBJECTIVE

The objective of this study was to assess the impact of the level of testing complexity on the predicted performance of asphalt pavements through a comprehensive laboratory evaluation of four different highly recycled surface mixes produced and placed in the Commonwealth of Virginia. The mixes featured either 30% or 45% RAP, different asphalt binder contents, the use of a warm mix asphalt (WMA) additive, and the use of a rejuvenator. To analyze the mixes’ performance in great depth, the research team defined a three-level testing framework herein referred to as basic, intermediate, and advanced. Each level was characterized by an increasing degree of complexity and included tests to characterize both the cracking resistance and the rutting resistance of the evaluated mixes. Through the review of the theoretical background, the evaluation of the test procedures, and a statistical analysis of the results, it was possible to identify the strengths and weaknesses of each test. Based on these considerations, it was possible to provide guidelines to develop appropriate quality assessment criteria and mix design methodology.

5.4 SIGNIFICANCE OF THE STUDY

In order to support the use of greater RAP contents and innovative materials (e.g. polymers, fibers, and rejuvenators) into asphalt mixes, the balanced mix design approach has been established. However, there are many performance tests of varying complexity, reliability, and cost from which to choose.

This research aims at comparing different tests that can characterize asphalt mixes: some are more simple tests, promoted because of their relatively faster execution and lower cost; some tests involve machines like the AMPT, which is promoted because of its relatively higher reliability and closer association with pavement design software. It is fundamental to define the suitability of each level of laboratory tests in order for them to be adequately used as tools to support the mix design and the quality assessment of asphalt mixes.

5.5 EXPERIMENTAL PLAN

5.5.1 Materials

The experimental plan consisted of performing multiple laboratory tests to characterize the mixes' resistance to primary distresses such as cracking and rutting. Four high-RAP asphalt mixes were evaluated and are listed in Table 21. The evaluated mixes were dense-graded surface mixtures with a nominal maximum aggregate size (NMAS) of 9.5 mm. Figure 62 shows that the design gradations were in agreement with the Virginia Department of Transportation (VDOT) requirements (VDOT, 2016). Note that, even though mixtures 45_NR and 45_R had the same gradation, they differed in terms of asphalt content, with mix 45_R featuring the use of a bio-oil based rejuvenator.

Table 21 General Design Overview of Evaluated Mixtures

Mix ID	RAP (% by weight)	Mix Design Information
30_LB	30	5.6%[a] PG 64S-22, 0.5% WMA Cecabase RT® 945 with AD-here® LOF 65-00[b] (control mix)
30_HB	30	6.0%[a] PG 64S-22, 0.5% WMA Cecabase RT® 945 with AD-here® LOF 65-00[b]
45_NR	45	6.8%[a] PG 64S-22, 0.3% WMA Evotherm® J1
45_R	45	6.2%[a] PG 64S-22, 0.3% WMA Evotherm® J1, 3.5% rejuvenator Evotherm® CA7

[a]Percentage by weight of the mix
[b]Bitumen performance-improving additive

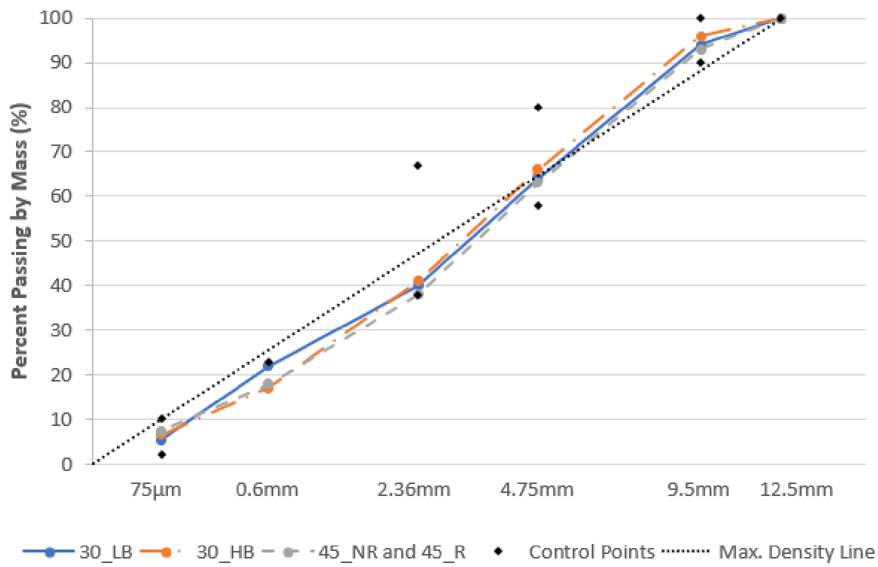

Figure 62 Aggregate Gradation Curves of Evaluated Asphalt Mixtures

The materials were obtained during the construction of the experiment testing lanes at the Virginia Accelerated Pavement Testing (APT) facility located at the Virginia Tech Transportation Institute (VTTI). Loose mixtures were collected from the plant and brought back to the laboratory for further processing. All tests were conducted on reheated laboratory-compacted specimens. Reheating was accomplished by heating the cooled mixture to the laboratory compaction temperature prescribed from the mix design and then compacting it. The effects of aging were not evaluated as part of this study; therefore, no additional laboratory aging was applied to the evaluated mixtures. All specimens were compacted to an air void content of 7.0 ± 0.5%. A minimum of three replicates were prepared and evaluated for each selected performance test.

5.5.2 Volumetric Properties

The primary volumetric properties of the four mixtures are reported in Table 22. In order to better characterize these mixtures, the corresponding asphalt binder film thickness was calculated. The process considered the estimated surface area of the aggregates, the total asphalt content (AC), and the effective asphalt content. It should be noted that a PG 64S-22 asphalt binder from the same supplier was used to produce all asphalt mixtures.

Table 22 Volumetric Properties of the Evaluated Mixes

Mixture ID	30_LB	30_HB	45_NR	45_R
Job Mix Formula ID	**443**	**444**	**445**	**446**
Composition				
RAP (% by weight)	30	30	45	45
PG of Virgin Binder	64-22S[(a)]	64-22S[(a)]	64-22S[(a)]	64-22S[(a)]
Property				
AC (%)	5.6	6.0	6.8	6.2
Gmm	2.553	2.531	2.508	2.545
VTM (%)	4.0[(b)]	2.1[(c)]	1.2[(c)]	2.5[(c)]
VMA (%)	16.8	15.9	16.9	16.7
VFA (%)	76.2	87.0	92.7	84.8
Dust proportion	1.0	1.1	1.1	1.2
Gsb	2.756	2.756	2.765	2.765
Gse	2.770	2.772	2.786	2.785
Pbe	5.42	5.80	6.54	5.95
Surface Area (m^2/kg)	6.26	6.38	6.85	6.85
Asphalt film thickness (microns)	9.0	9.5	10.0	9.1

[(a)]The ‘S’ designation stands for low to medium traffic loading levels (VDOT, 2016).
[(b)]The mix was designed based on volumetric properties: the design AC content was selected at 4% air voids.
[(c)]The mix was designed to solely meet pre-selected performance criteria (and not volumetric properties).

5.6 TESTS’ SELECTION

The process of test selection considered the most available and accepted testing procedures for cracking and rutting evaluation (West et al., 2018). With respect to cracking, only the tests that characterize the mix at intermediate temperature were considered, with the objective of characterizing the overall cracking performance concerning mechanisms such as top-down and bottom-up fatigue due to traffic loading.

Extensive reviews of the various available tests developed to characterize asphalt mixtures are already present in the literature, such as NCHRP Project No. 9-57 (Zhou et al., 2016) for cracking and Project No. 9-17 (Kandhal & Cooley, 2003) for rutting. In the present study, the research team aimed at identifying different levels of testing based on their complexity, testing time, and equipment cost. This goal was in consideration of the fact that, in addition to the scientific validity of a test, it is the practicality and accessibility by contractors and transportation agencies that determine its potential to become a widespread tool for asphalt mixtures’ evaluation.

It was possible to identify three levels of testing for both cracking and rutting: basic, intermediate, and advanced. The basic level was characterized by reasonably fast and inexpensive tests whereas the advanced level included more extensive and demanding procedures using higher cost equipment. The cracking and rutting tests taken into account are listed in Table 23 and Table 24, respectively.

Table 23 Testing Levels of Considered Cracking Tests

Level	Test
Basic	Indirect Tensile Cracking Test at Intermediate Temperature (Zhou et al., 2017)
Intermediate	Semicircular Bending Test – Louisiana Transportation Research Center (SCB-LTRC) (Wu et al., 2005) Semicircular Bending Test – Illinois Flexibility Index Test (I-FIT) (Ozer et al., 2016) Fenix Test (Pérez-Jiménez et al., 2010) Texas Overlay Test (OT) (Zhou & Scullion, 2005)
Advanced	Flexural Bending Beam Test (AASHTO, 2019a; Tayebali et al., 1994) Simplified Viscoelastic Continuum Damage (S-VECD) Fatigue Test (AASHTO, 2019b)

Table 24 Testing Levels of Considered Rutting Tests

Level	Test
Basic	High Temperature Indirect Tensile Test (HT-IDT) (Christensen et al., 2004)
Intermediate	Hamburg Wheel Tracking Device (HWTD) (AASHTO, 2019b) Asphalt Pavement Analyzer (APA) (AASHTO, 2019c)
Advanced	Confined Flow Number (CFN) (AASHTO, 2019b) Stress Sweep Rutting (SSR) (AASHTO, 2019d)

5.6.1 Basic Level

As mentioned, the basic level included only tests characterized as having a short specimen preparation and testing time. In particular, these tests do not require any specific cutting, coring, or gluing.

The Indirect Tensile (IDT) test is notably easy to perform: the cylindrical specimens require compaction to a height of 62 mm and, after checking the air void content, conditioning to the target temperature (typically 25°C) (ASTM, 2019). Then, each specimen is centered in the loading fixture and a load is applied to a constant load-line displacement (LLD) rate of 50 mm/min. The test is completed in less than 30 seconds and the cracking tolerance index (CT_{Index}) can be easily computed with an electronic spreadsheet. In addition, the equipment cost is limited, as it is commercially available for a starting price of approximately $6,000 (Humboldt, 2020) for a screw-drive type machine and up to $23,000 for a servo-hydraulic type machine.

Similarly, the high-temperature indirect tensile test (HT-IDT) for rutting evaluation uses 95-mm tall specimens which, after the air voids verification, need to be conditioned to the high PG temperature of the mix (Christensen et al., 2004; Jenks et al., 2011). The test setup is the same as the CT_{Index} test and the HT-IDT strength can be immediately calculated using the maximum load measured and the specimen geometrical properties (ALDOT, 2020).

5.6.2 Intermediate Level

The intermediate level included tests that needed longer times for specimens' preparation, including cutting, notching, and gluing, and/or testing operations.

Two different procedures are available for the Semicircular Bending (SCB) test: one developed by the Louisiana Transportation Research Center (LTRC) and the other by the Illinois Center for Transportation, which is known as Illinois Flexibility Index Test (I-FIT). The SCB-LTRC features the use of three different notch depths (25 mm, 32 mm, and 38 mm) so that it is possible to determine the critical strain energy release rate (Jc). The LLD in this case is 0.5 mm/min (ASTM, 2016). The necessary equipment is the same as that used by the IDT tests; in addition, the cost of tools, such as a water-cooled masonry saw, needs to be considered. The I-FIT features the compaction of one gyratory pill compacted to a height of 160 mm, which needs to be cut in order to obtain two discs 50-mm thick each. The air void contents are then checked on both discs and, after being cut in half, a notch of 15-mm is sawed in each specimen. A constant LLD rate of 50 mm/min is applied until specimen failure. Fracture energy, post-peak slope, displacement at peak load, and critical displacement are determined and calculated from the load-displacement curve to calculate a cracking index (known as FI) (AASHTO, 2018).

The Fenix test features similar specimens as the SCB test. The specimen's halves, divided by the notch, are glued to steel plates and a loading rate of 1 mm/min is applied. The dissipated energy and tensile stiffness index are calculated based on the load-displacement curve (Pérez-Jiménez et al., 2010; Pérez Madrigal et al., 2017).

The Texas Overlay test (OT) consists of gluing a specimen to two steel plates. While one plate is fixed, the other is moved to simulate the opening and closing mechanism of joints/cracks below an asphalt overlay. The specimens' preparation includes four cuts, and a gluing jig is necessary. The Texas OT is a cyclic displacement-controlled test and the number of load cycles to failure is reported as are the fracture properties (AASHTO, 2019b). The Texas OT can be performed both on specific equipment, which costs approximately $46,000 (Zhou et al., 2016), and machines like a Universal Testing Machine (UTM) and AMPT.

For rutting, tests involving the use of the APA and the Hamburg Wheel Tracking Device (HWTD) were included in the intermediate level. For the APA test, the specimen preparation requires compacting gyratory cylindrical specimens to a height of 75 mm (originally, beam-shaped samples were tested). The specimens are placed into specific molds and then inserted into the APA machine. The machine applies 8,000 loading cycles through the use of loaded wheels on top of pressurized linear hoses that are moved back and forth over a test specimen (AASHTO, 2019c). The HWTD can either test slab specimens or gyratory cylindrical specimens. The specimens are submerged in a temperature-controlled water bath and are repetitively loaded (20,000 cycles) using a steel wheel (AASHTO, 2019b). In addition, the HWTD can evaluate the moisture susceptibility of a mixture through the definition of the stripping point. For both the APA and the HWTD, the rut depth is plotted against the number of passes and the main test parameters are obtained. The equipment for both tests can be obtained for approximately $60,000 (Gilson, 2020).

5.6.3 Advanced Level

The advanced level featured tests that require operations of cutting and/or coring to prepare the specimens as well as multiple days for test completion.

The Flexural Bending Beam Test features the use of a beam specimen with a section of 50 mm by 63 mm, and 380 mm long. During the test, the beam is held by four clamps and subjected to a repeated haversine (or sinusoidal) loading through the two inner clamps, while the outer clamps provide a reaction load. This configuration produces a constant bending moment over the center part of the beam between the two inside clamps. The test is usually run in strain-controlled mode with a frequency of 10 Hz and is performed at an intermediate temperature, usually 20°C (68°F) for the Commonwealth of Virginia. The beam preparation is quite time-consuming and the test machine is usually a universal test machine that may exceed $150,000 in terms of procurement costs and operation (Zhou et al., 2016); alternatively, a stand-alone servo-pneumatic four point bending system (4PB) can be found commercially available at around $25,000.

The Simplified Viscoelastic Continuum Damage (S-VECD) Fatigue, Confined Flow Number (CFN), and Stress Sweep Rutting (SSR) tests can all be performed using the AMPT machine. The AMPT setup was developed as part of NCHRP Project 09-29 "Simple Performance Tester for Superpave Mix Design," in which the FHWA assumed responsibility for the development of the Simple Performance Test machine (FHWA, 2013). Today, the machine is reportedly available for $120,000. The preparation of AMPT specimens requires coring and cutting operations. According to the American Association of State Highway and Transportation Officials (AASHTO) R 83, the typical AMPT specimens are cut and cored from laboratory compacted specimens with a diameter of 100 mm and a height of either 150 mm or 130 mm. The testing and operator training times are significantly demanding. The test results constitute the primary inputs for pavement design software such as FlexPAVE™ and AASHTOWare® Pavement Mechanistic-Empirical Design.

5.6.4 Selected Tests and Methodology

To characterize the asphalt mixtures at different levels of testing complexity, at least one cracking test and one rutting test was selected from each of the aforementioned groups. The selected tests are listed in Table 25:

- Due to its ease and testing speed, the IDT test setup was selected to represent the basic level for evaluating both cracking (IDT test) and rutting (HT-IDT).
- At the intermediate level, many alternatives were evaluated. For cracking resistance, the I-FIT was selected primarily due to survey responses from State DOTs and asphalt contractors, published as part of NCHRP Project 20-07 (West et al., 2018), where the I-FIT was listed as the test with most potential to address top-down and bottom-up fatigue cracking. For rutting, the APA was selected. Compared to the HWTD, the APA constitutes a similar but less demanding test procedure. The APA's higher practicality also motivated its selection by VDOT as part of the proposed balanced mix design procedure (VDOT, 2019).
- For the advanced level, the AMPT-based testing was considered. The AMPT can characterize cracking resistance (through the S-VECD test), rutting resistance (through the CFN and SSR tests), and the dynamic modulus (E*) of the mixtures. Moreover, the AMPT has been promoted for its ability to use the same tests for both mixture evaluation and structural design and the FHWA has adopted the AMPT in the development of performance-related specifications.

Table 25 Selected Performance Tests at Various Testing Levels

Level	Cracking	Rutting	
Basic	CT_{Index}	HT-IDT	
Intermediate	I-FIT	APA	
Advanced	S-VECD	CFN	SSR

5.6.5 Cracking Tests Attributes

The formation of cracks in asphalt pavements is due to a combination of multiple factors, such as materials, structure, climate, and loading conditions. In all cases, three stages are involved in crack formation: initiation, with the formation of microcracks due to load applications; propagation, with the formation of macrocracks due to the joining of microcracks; and eventual fracture or complete failure of material (Dowling, 2012). To better understand this pavement distress, the use of fracture mechanics and fracture tests were developed. Many tests that evaluate the fracture energy under a monotonic loading (e.g. IDT, SCB) were studied and their correlation with the field cracking has been positively evaluated (Al-Qadi et al., 2019; Zhou et al., 2016). However, this laboratory cracking mechanism is very different from what actually happens in the field, and therefore proper fatigue tests were introduced as well (Underwood et al., 2012a).

Despite these potential differences, with respect to the materials properties, agencies and contractors have started using cracking tests and associated cracking indices to indicate the cracking fatigue resistance of asphalt mixtures. The use of such indices aims at supporting the mix design phase as well as quality control and assurance. It is possible to observe how such indices can be derived both from fracture tests (CT_{Index} and FI) and cyclic loading tests (S_{app}).

- CT_{Index} – The CT_{Index} is derived from the performance of a typical IDT test, run at a constant LLD rate, and the analysis of the measured load-displacement curve. The IDT cracking test has its roots in the crack growth theory, in which the crack propagation rate is a function of the range of stress intensity seen in a loading cycle (Bazant & Prat, 1988; Paris & Erdogan, 1963). Through a series of assumptions and simplification, it is possible to see that cracking growth rate is highly related to the fracture energy (G_f), the post-peak slope at 75% maximum load (m_{75}), the displacement corresponding to 75% of the maximum measured load (l_{75}), and the specimen diameter (D) (Zhou et al., 2017). The CT_{Index} is calculated using the following equation:

$$CT_{Index} = \frac{G_f}{|m_{75}|} \cdot \frac{l_{75}}{D} \cdot \frac{t}{62}$$

- I-FIT – Similarly to the IDT test, the I-FIT derives from the crack growth theory (Bazant & Prat, 1988). The main output of the I-FIT is the Flexibility Index (FI). The FI is calculated using the slope of the post-peak curve at the inflection point (m) and the fracture energy (G_f) (Al-Qadi et al., 2019). The FI equation is shown below:

$$FI = 0.01 \cdot \frac{G_f}{|m|}$$

  The calculated FI is an index that shows an asphalt mixture's overall capacity to resist damage related to cracking (Al-Qadi et al., 2015). The higher the FI, the better a mixture can resist crack propagation under tensile stress. (AASHTO, 2018).

- S-VECD – The S-VECD test procedure uses the AMPT to first conduct a dynamic modulus test to determine the linear viscoelastic properties of the mix. Afterwards, a direct tension cyclic test is performed to develop the damage characteristic curve. Together, the viscoelastic material properties and damage characteristic curve can be used to obtain the fatigue behavior of an asphalt mixture. A few AMPT cyclic fatigue tests at a single temperature are sufficient to obtain the S-VECD model (Underwood et al., 2012b). To define the asphalt mixtures' fatigue performance, a fatigue cracking index parameter, referred to as apparent damage capacity (S_{app}), has been introduced (Wang et al., 2020). The calculation of S_{app} can be conducted with "FlexMAT™ for Cracking," an Excel-based tool provided by the FHWA. The goal was to allow pavement engineers to make sound decisions, based on the S-VECD test results, about mix design and mix quality control. Also, with FlexPAVE™, a pavement performance prediction program, the model allows the prediction of an asphalt pavement fatigue performance under realistic moving loads and climatic conditions (Wang et al., 2018). In this study, an analysis of the fatigue test output was followed by the determination of the S_{app} index.

5.6.6 Rutting Tests Attributes

Similarly to cracking, the occurrence of permanent deformation in asphalt pavements can be due to a combination of factors: hot mix asphalt (HMA) quality, properties of base and subgrade, structural design, climate, and traffic loading. Many laboratory tests aim at replicating the conditions of a pavement under traffic loading. Such accelerated procedures consist of the application of repeated loads in a short period of time at high temperatures (e.g., APA and CFN). Other research aims instead at applying the time-temperature superposition principle to use lower test temperatures and slower loading rates; in such a way, the test conditions would be rheologically equivalent to the high-frequency traffic loading at the critical pavement temperatures for rutting (Christensen et al., 2004). Based on this assumption, tests like the HT-IDT were developed and were found to be effective as simple tests for evaluating the resistance of asphalt mixes to rutting.

- HT-IDT – The HT-IDT is performed similarly to the tensile strength ratio (TSR) test described in AASHTO T 283. In the HT-IDT's case, the specimens are conditioned at the test temperature, which is defined as the temperature 10°C below the average, 7-day maximum pavement temperature, 20 mm below the pavement surface at 50% reliability, as determined using LTPPBind, Version 3.1 (Advanced Asphalt Technologies, 2011). For the selected Virginia location, the test temperature was equal to 49°C. The final test parameter is the HT-IDT strength:

$$HT - IDT\ strenght = \frac{2 \cdot P}{\pi \cdot D \cdot H}$$

Where P is the maximum load, D is the average diameter, and H is the average height. A proposed set of recommended minimum HT-IDT strength requirements is reported in Table 26 (Christensen et al., 2004).

Table 26 Preliminary HT-IDT requirements (Christensen et al., 2004)

Traffic Level (million ESALs)	Rut Resistance	Min. HT-IDT Strength (kPa)
Less than 3	Poor	N/A
Between 3 and 10	Fair	200
Between 10 and 30	Good	320
Greater than 30	Excellent	440

- APA – The APA is a device designed to test the rutting resistance of asphalt mixtures through cyclic wheel load applications in an environmentally controlled chamber. The test temperature has to be set to the high temperature of the standard Superpave PG mixture's binder, which, for all the mixes examined in this study, was equal to 64°C. The main output of the test is the rutting development over 8,000 load cycles.
- CFN – The CFN test features the application of a repeated haversine axial compressive load pulse of 0.1 seconds every 1.0 seconds at a constant temperature. Throughout the test, which can be conducted with the AMPT, the permanent and resilient axial strains (ε_p and ε_r) are measured as a function of the load cycles. Flow number (FN) is defined as the number of load cycles corresponding to the minimum rate of change of permanent axial strain, which coincides with the start of the tertiary region of the cumulative permanent strain curve. In other words, FN indicates the onset of shear deformation in asphalt mixtures, which constitutes a meaningful indicator of the rutting resistance. The FN test was run in the confined mode at four different temperatures: 30°C, 40°C, 50°C, and 60°C. The confining pressure was 69 kPa, the deviator stress was 483 kPa, and the maximum number of loading cycles was set to 10,000.
- SSR – The SSR test follows a similar procedure to the FN test but takes into account the effects of temperature, loading time, and deviatoric stress on the rutting resistance. In contrast to the FN test, which can represent the resistance to permanent deformation only in terms of ranking at a certain testing condition, SSR generates a rutting master curve which describes the distress potential at various temperature and traffic combinations (Kim & Kim, 2017). SSR tests were performed on four test specimens at two temperatures: high (T_H) and low (T_L). The selection of appropriate temperatures is defined in AASHTO TP 134: T_H is calculated from the degree-days parameter obtained using LTPPBind Version 3.1, while T_L is selected based on the PG grade. In

both cases, the specimens are subjected to three 200-cycle loading blocks of three deviatoric stress levels, as shown in Table 27, and constant confining pressure of 69 kPa.

Table 27 SSR Test Parameters

Test Parameter	T_H	T_L
Loading Block 1 (kPa)	689	483
Loading Block 2 (kPa)	483	689
Loading Block 3 (kPa)	896	896
Load pulse (s)	0.4	0.4
Rest period (s)	3.6	1.6

The main SSR output is the shift model for rutting, which is composed of one permanent strain master curve and two shift functions. The shift model has been incorporated into the pavement performance prediction program, FlexPAVE™ (AASHTO, 2019d). Also, based on the SSR test outcome, the Rutting Strain Index (RSI) was computed. Since the field performance is a consequence of rutting across all layers of the pavement system, three different reference structures have been defined to study all kinds of asphalt mixtures: surface, intermediate, and base layers (Ghanbari et al., 2020). For surface mixes, the following three-layer system is considered: subgrade (E = 69 MPa), 20-cm thick aggregate base (E = 206 MPa), and 10-cm thickness of the evaluated surface asphalt mix. To determine the RSI, the average permanent strain due to a predefined traffic level at the end of a 20-year pavement service life (240 months) is calculated. The calculation of RSI for each mix was conducted with “FlexMAT™ for Rutting,” an Excel-based software tool provided by the FHWA.

5.7 TEST RESULTS AND DISCUSSIONS

5.7.1 Cracking Performance

5.7.1.1 CT_{Index}

Six replicates were tested for each mix and the results are shown in Figure 63; the middle line of the box represents the median, the x represents the mean, the bottom line of the box represents the median of the 1st quartile, the top line of the box represents the median of the 3rd quartile, and the vertical lines extend from the ends of the box to the minimum and maximum values. The two worst performing mixes were those containing 30% RAP: mixes 30_LB and 30_HB reported averages of 36 and 70 respectively. Overall, the best performing mixes were the ones that contained 45% RAP: mixes 45_NR and 45_R averaged 435 and 148, respectively. In these mixes, the stiffening effect that is usually associated with high RAP content (Al-Qadi et al., 2007) was well mitigated by the use of a high AC and the rejuvenator, respectively. Note that all mixes were tested in short-term aging conditions.

With respect to the volumetric properties, it was possible to observe how the best performing mix (45_NR) had a thicker estimated asphalt film thickness, while the worst (30_LB) had a thinner film. Figure 63 also shows how the only mix that was well below the VDOT proposed threshold of 70 (VDOT, 2019) was control mix 30_LB, which is a typical mix used in Virginia.

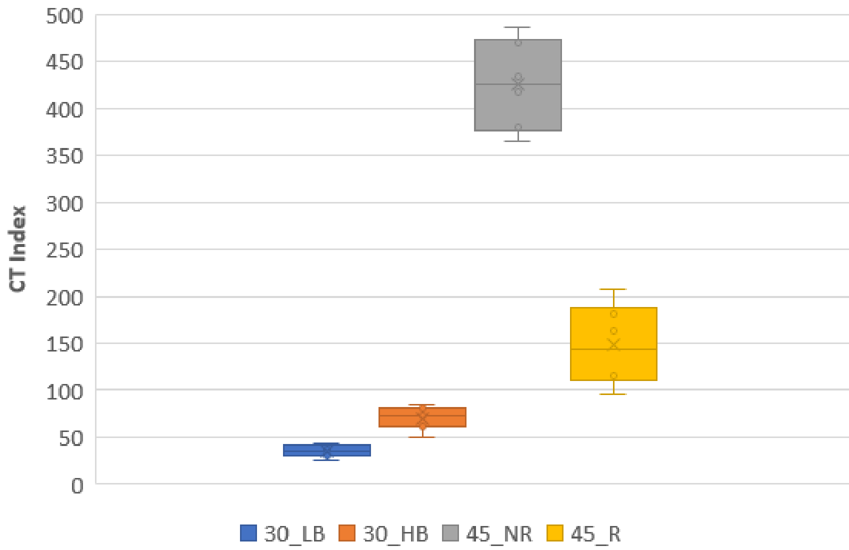

Figure 63 IDT Cracking Test Results

5.7.1.2 I-FIT

Figure 64 shows the results of the I-FIT, in which six specimens were tested for each mix. The highest average FI was achieved by the 45_NR mix (22.8). In contrast to the IDT test, mixes 30_HB and 45_R achieved very similar FI values (6.9 and 7.3 respectively). The worst-performing mix was control mix 30_LB, with an average FI equal to 2.0. It is worth highlighting again the difference in asphalt film thickness between the mixes.

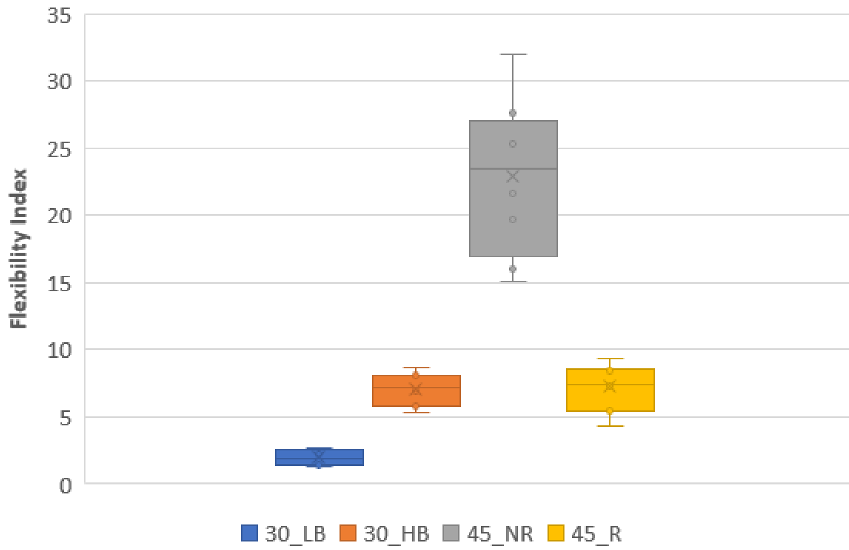

Figure 64 I-FIT Results

5.7.1.3 S-VECD and Damage Capacity Index (S_{app})

The dynamic modulus master curves at a reference temperature of 20°C are shown in Figure 65. The tests were conducted for all mixtures at 4.4°C, 21.1°C, 37.8°C, and 54.4°C. They were conducted at 25, 10, 5, 1, 0.5, and 0.1 Hz loading rate as specified in AASHTO T 378. Then, each mix was tested at least at three different strain levels for cyclic fatigue in accordance with AASHTO TP 107.

A traditional way of evaluating the fatigue life of an asphalt mixture specimen is to consider the number of cycles corresponding to a 50% reduction of the initial stiffness $|E^*|$. The results are shown in Figure 66. It is possible to observe how, since the higher the curve, the better the fatigue life of the mix, 45_NR was the best performing mix (i.e. to obtain similar numbers of cycles to failure, higher strain levels were needed for 45_NR). The second best mix was 45_R, while 30_LB and 30_HB achieved similar outputs characterized by a shorter fatigue life. These results were expected since mixture 45_NR had the highest AC and because the short term aging was not likely to greatly affect the overall mixtures' flexibility.

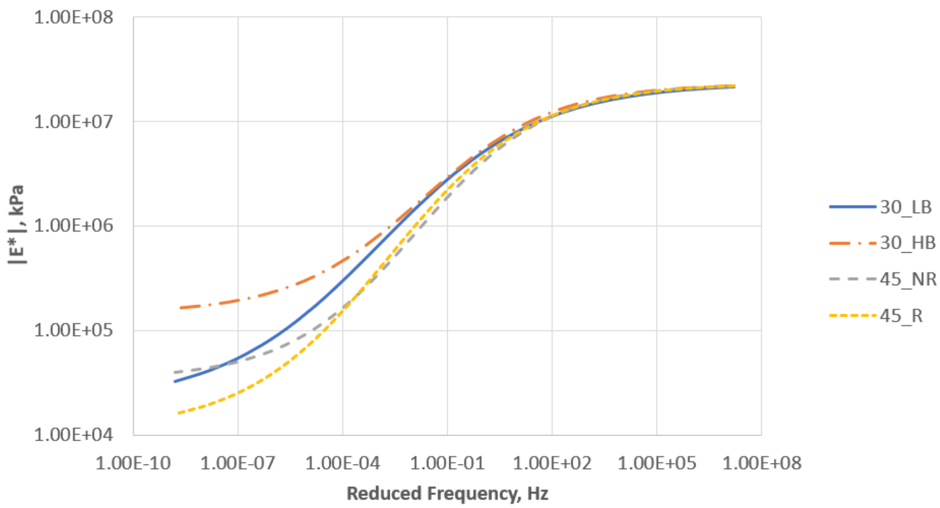

Figure 65 Dynamic Modulus Test Results

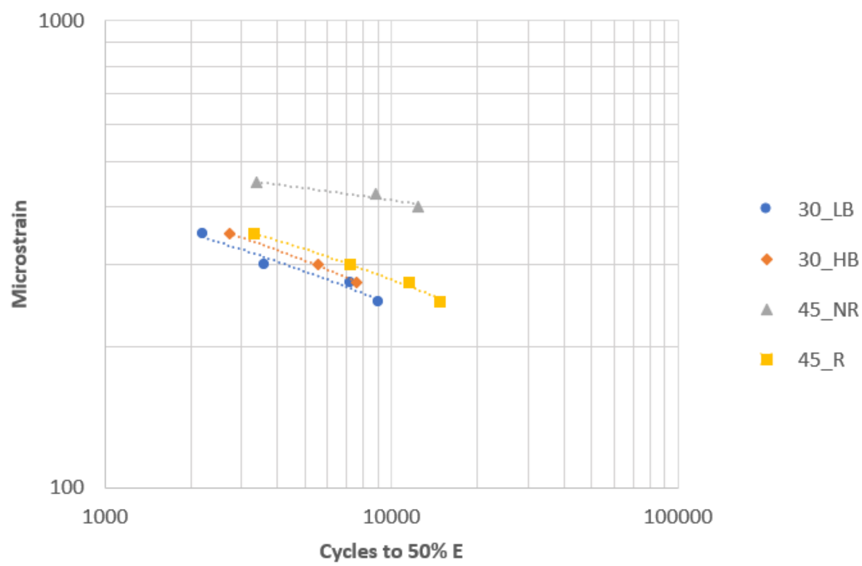

Figure 66 S-VECD Fatigue Test Results

For the calculation of the S_{app} index, the average temperature of the high and low PGs of the virgin binder (i.e., 21°C) was selected for all evaluated mixes. The results are shown in Figure 67. Interestingly, with reference to the recommended S_{app} threshold value, shown in Table 28, all mixes fell into the same category. This can be attributed to the similar PG of the resultant binder blends for the evaluated mixes. The mixes achieved similar results, between a minimum of 15.77 (mix 30_HB) and a maximum of 21.92 (mix 45_NR). These results can be explained by the fact that the S_{app} parameter considers the effects of both the material's modulus and toughness on fatigue failure (i.e., ability to absorb energy without fracturing). For instance, mix 30_LB was characterized by high modulus (approximately 6,500 MPa at a test temperature and frequency of 24°C and 10 Hz respectively) and low toughness. On the contrary, mix 45_NR featured a low modulus (around 3,000 MPa) but, for a set strain level, many more cycles were needed to reach failure, as shown in Figure 66. Therefore, for the mixes under study, the effects of modulus and toughness were believed to balance each other when computing the S_{app} index.

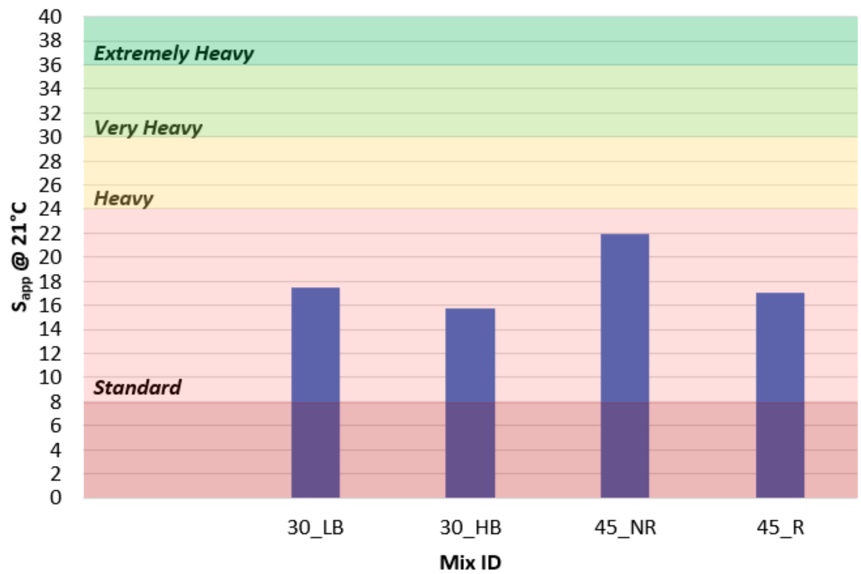

Figure 67 S_{app} Results and Recommended Traffic Designation Tiers Highlighted

Table 28 Recommended Threshold Values for S_{app} Parameter (Wang et al., 2020)

Traffic Level (million ESALs)	Tier	S_{app} Limits
Less than 10	Standard	$S_{app} > 8$
Between 10 and 30	Heavy	$S_{app} > 24$
Greater than 30	Very heavy	$S_{app} > 30$
Greater than 30 and slow traffic	Extremely heavy	$S_{app} > 36$

5.7.2 Rutting Performance

5.7.2.1 HT-IDT

Six specimens were tested per mix and the test results in Figure 68 shows that mix 30_LB had the highest strength values with an average of 277 kPa. Mixes 30_HB and 45_R had similar performance (217 kPa and 198 kPa respectively), while the worst mix in terms of rutting resistance was 45_NR, probably due to the high AC, with a resultant average of 110 kPa. Mix 45_NR was also the only mix that was clearly located in the “Poor” rutting resistance tier (see Table 26), while the other three mixes all fell into the “Fair” category (only part of mix 45_R replicates).

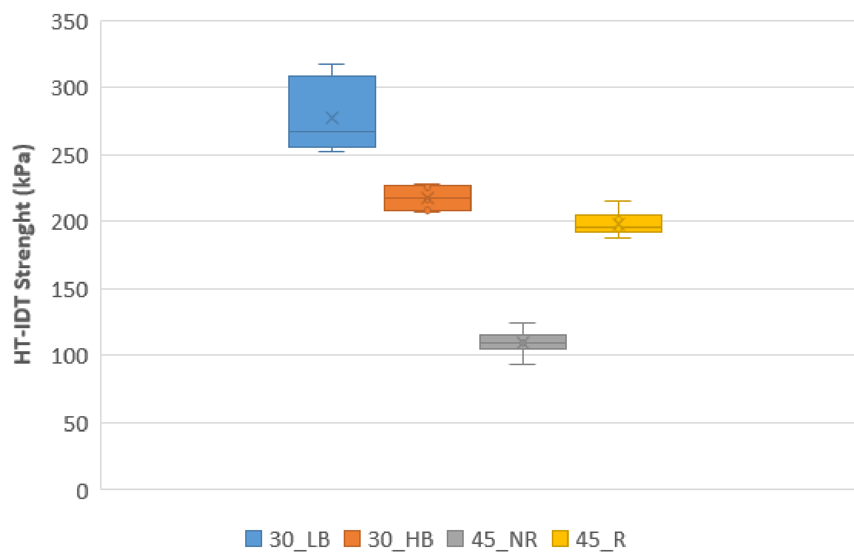

Figure 68 HT-IDT Results

5.7.2.2 APA Test

The averages of the final rut depths measured are shown in Figure 69. The 30% RAP mixes (30_LB and 30_HB) recorded the lowest, and almost identical, average rut values (2.38 mm and 2.41 mm respectively). The 45% RAP mix with rejuvenator 45_R also demonstrated a good rutting resistance (2.76 mm). Interestingly, as shown in Figure 65, mix 45_R exhibited the lowest stiffness at high temperature. The 45% RAP mix with high AC (45_NR) showed instead a behavior that suggests potential higher rutting since the average final rut was 5.78 mm. However, for the proposed VDOT performance acceptance criteria of an 8-mm maximum final rut (VDOT, 2019), all mixes had a satisfactory performance.

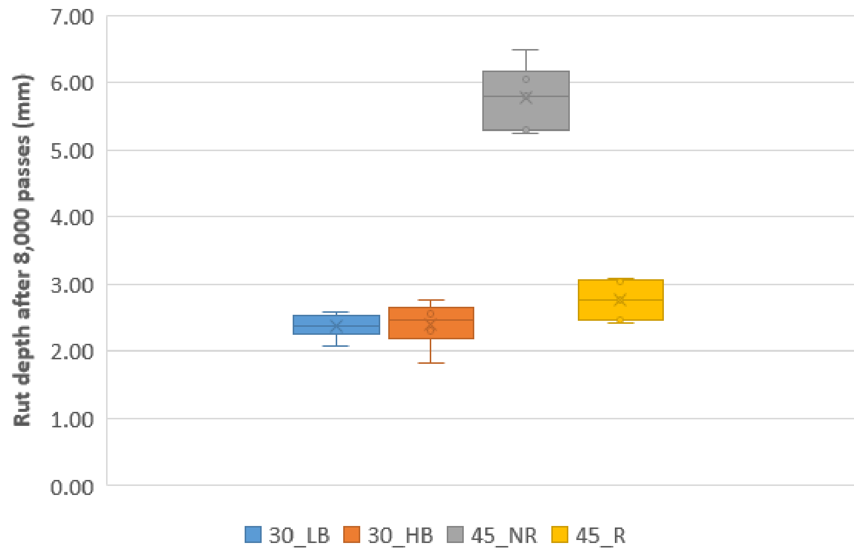

Figure 69 APA test results

5.7.2.3 CFN Test

The CFN test results in Figure 70 indicate how mix 45_NR, when tested at 30°C, and mixes 30_LB, 30_HB, and 45_R, when tested at 40°C, did not reach the flow point.

The only temperatures at which all the mixes were tested were 40°C and 50°C. Mixture 30_LB was evaluated at a relatively lower range of temperatures in order to be able to accumulate a minimum amount of measured data prior to specimen failure. The results are shown in Figure 71. The most rut-resistant mixes were 30_LB and 30_HB, while the worst mix was 45_NR. Mix 45_R achieved an intermediate result.

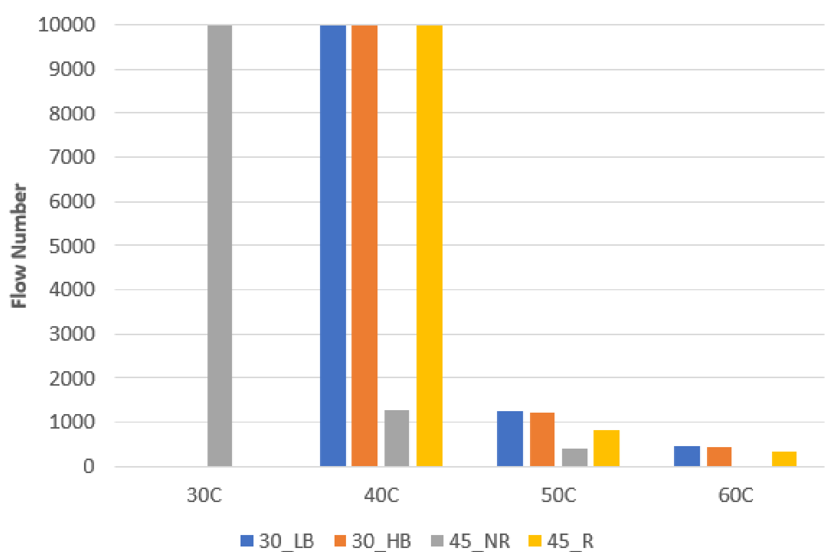

Figure 70 Confined FN Test Results

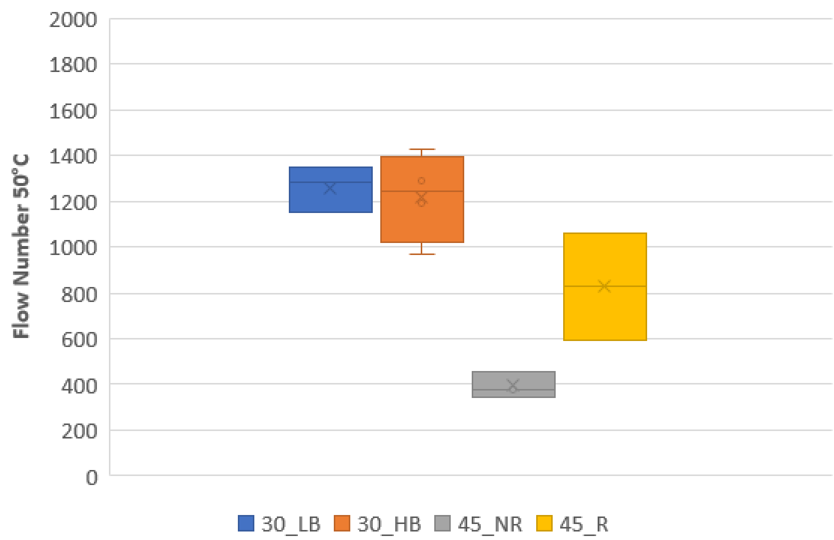

Figure 71 Confined FN Test Results at 50°C

5.7.2.4 SSR and Rutting Strain Index (RSI)

The SSR test results are shown in Figure 72, which also highlights the recommended threshold values of RSI (listed in Table 29). The 30% RAP mixes, 30_LB and 30_HB, were the best performers, with an RSI of 1.48 and 1.20 respectively. Together with mix 45_R (RSI of 2.09) these mixes fell into the "Very Heavy" traffic category. Mix 45_NR instead fell in the "Standard" category, which is two levels below, with an RSI of 4.05.

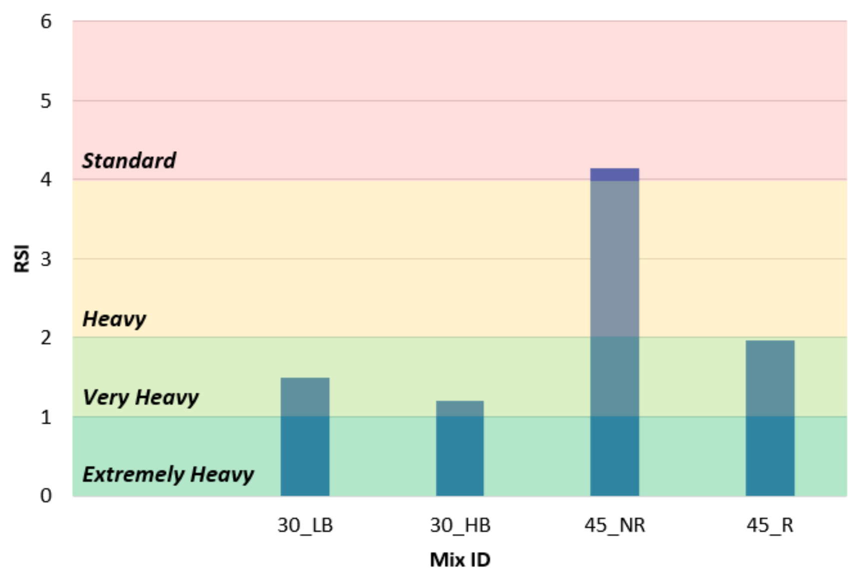

Figure 72 RSI Results and Recommended Traffic Designation Tiers

Table 29 Recommended threshold values of RSI (Ghanbari et al., 2020)

Traffic Level (million ESALs)	Tier	RSI Limits
Less than 10	Standard	4<RSI < 12
Between 10 and 30	Heavy	2<RSI < 4
Greater than 30	Very heavy	1<RSI < 2
Greater than 30 and slow traffic	Extremely heavy	RSI < 1

5.8 STATISTICAL ANALYSIS

An analysis was carried out to evaluate the coefficient of variation (COV) of each test. For the cracking tests, the CT_{Index} test had the lowest COV (19%), which meets the precision estimates of 20.7% recently developed by researchers at Virginia Transportation Research Council as part of their CT_{Index} Round Robin effort for untrimmed data single-operator (Diefenderfer S.D. et al., 2020). Table 30 and Table 31 show the average test result and respective COV for each test and mix combination. On each row, the colored bars are adjusted in relation to the highest value present in that row. With respect to the results, for most of the tests, the higher the value, the better the performance (cells shaded in green). The only exceptions are the APA and RSI results, for which the higher the value, the poorer the rutting performance (cells shaded in red). In both tables, the last column indicates the COV of every test. Generally, the rutting tests showed a lower dispersion of the results, with the exception of the CFN test performed at 60°C. The HT-IDT had the lowest overall COV (7%). For the cracking tests, the CT_{Index} test had the lowest COV (19%).

Table 30 Cracking Test Results and Corresponding Coefficients of Variation (COV)

Level	Test	30_LB		30_HB		45_NR		45_R		Test Avg. COV
		Average	COV	Average	COV	Average	COV	Average	COV	
Basic	CT	35.52	17%	69.96	17%	425.39	11%	147.54	29%	19%
Intermediate	I-FIT	1.96	31%	6.85	17%	22.85	25%	7.27	25%	25%
Advanced	S_{app}	17.45	N/A	15.77	N/A	21.92	N/A	17.03	N/A	N/A

Table 31 Rutting Test Results and Corresponding Coefficients of Variation (COV)

Level	Test	30_LB		30_HB		45_NR		45_R		Test Avg. COV
		Average	COV	Average	COV	Average	COV	Average	COV	
Basic	HT - IDT, kPa	277.52	10%	217.43	4%	109.75	9%	198.17	5%	7%
Intermediate	APA, mm	2.38	8%	2.41	14%	5.78	8%	2.76	10%	10%
Advanced	CFN 30°C	N/A	N/A	N/A	N/A	10000.00	0%	N/A	N/A	N/A
	CFN 40°C	10000.00	0%	10000.00	0%	1276.67	18%	10000.00	0%	N/A
	CFN 50°C	1259.00	8%	1218.00	16%	393.67	15%	826.50	N/A	13%
	CFN 60°C	418.67	31%	438.00	39%	N/A	N/A	253.00	N/A	35%
	RSI (SSR), %	1.48	N/A	1.20	N/A	4.05	N/A	2.09	N/A	N/A

In order to analyze the discerning capability of each test, Tukey's method was used to find means that were significantly different from each other. Table 32 and Table 33 summarize the results, indicating with different letters the mixes that were significantly different in terms of performance. For the CFN results, only the tests conducted at 50°C were considered; all the mixes were tested at this temperature, which was also close to the HT-IDT test temperature.

Table 32 Cracking Evaluation: Tukey's Honestly Significant Difference (HSD) Test Results

Test	Mix			
	30_LB	30_HB	45_NR	45_R
CT_{Index}	C	C	A	B
I-FIT	C	B	A	B

Table 33 Rutting Evaluation: Tukey's Honestly Significant Difference (HSD) Test Results

Test	Mix			
	30_LB	30_HB	45_NR	45_R
HT-IDT	A	B	C	B
APA	A	A	B	A
CFN 50°C	A	A	C	B

For the cracking tests, results show that both the CT_{Index} and the I-FIT tests separated the performance of mix 45_NR from the others. For the other mixes, the distinction was different between the two tests; however, in the worst-performing group, mix 30_LB was always present.

As far as the rutting tests, the worst-performing mix, according to all of the tests was mix 45_NR, which was always associated with a letter on its own. However, each test described the rutting performance of the other three mixes differently: according to the HT-IDT, mix 30_LB was clearly the best performing; the APA instead grouped all three together; while the CFN indicated mixes 30_LB and 30_HB as best.

The results also show that neither the COV analysis nor Tukey's test were applicable to fatigue (both the fatigue life curves and S_{app}) and SSR test results since the index values were obtained from a population of samples and without any replicates. It was instead possible to compare all of the tests in terms of ranking capabilities. Table 34 shows the ranking of all evaluated mixes according to every test performed, with numbering from 1, which is the mix with the relatively best performance, to 4, which is the mix with the relatively worst performance.

Table 34 Ranking of Cracking and Rutting Performance for all Evaluated Mixes

Distress of Interest	Cracking				Rutting			
Index ID	**CT_{Index}**	**I-FIT**	**S-VECD**	**S_{app}**	**HT-IDT**	**APA**	**CFN 50°C**	**RSI**
30_LB	4	4	4	2	1	1	1	2
30_HB	3	3	3	4	2	2	2	1
45_NR	1	1	1	1	4	4	4	4
45_R	2	2	2	3	3	3	3	3

For cracking, CT_{Index}, I-FIT, and S-VECD tests ranked the mixes in the same order. Only the mixes' calculated damage capacity index (S_{app}) gave a different ranking, with the only constant being that 45_NR was the best performing mix.

Similarly, for rutting, HT-IDT, APA, and FN resulted in the same exact raking for the mixes. In comparison, the calculated RSI only inverted the order of the two best mixes: 30_LB and 30_HB.

5.9 PROPOSED MIX DESIGN METHODOLOGY

Based on the results obtained, it was possible to consider different ways of implementing the use of performance tests during routine mix design operations. It was possible to observe how:

- With the exception of S_{app} and RSI, the ranking of the mixes was the same at all levels.
- The basic level tests were characterized by low COVs and, based on Tukey's test results, had good discrimination capabilities among evaluated mixtures.
- The intermediate level tests had higher COVs and, for the APA, fewer differences were found between the mixes' performances.
- The advanced level tests, with respect to the analysis of the S-VECD output and the CFN, reported similar results to the basic and intermediate levels. However, when processing the S-

VECD and SSR data with FlexMAT™, the indexes obtained (S_{app} and RSI) provided different information on the predicted performance results.

For these reasons, and to support a more simplified approach to designing mixtures that incorporate new materials or are based solely on performance characteristics, a reduced two-level design system was proposed based on the results of this study. The required performance tests can be selected in relation to the project importance, which could be based on the expected traffic level.

- Low-medium importance (e.g., less than 10 million ESALs) – The mix design process can be supported by the performance of basic tests, such as IDT, for both cracking and rutting. The use of these tests needs to be accompanied by the set of thresholds (e.g. minimum values of 70 and 150 kPa for CT_{Index} and HT-IDT, respectively) that would rule out mixtures prone to cracking and/or rutting.
- Medium-high importance (e.g., more than 10 million ESALs) – The selection of the mixes is complemented by the performance of advanced level tests, such as S-VECD and SSR. However, instead of using the simplified indexes obtained with FlexMAT™, the output of these tests is then used in the pavement performance prediction program FlexPAVE™, which allows simulating the behavior of the whole pavement structure over the course of a number of years. Noted that this method requires previous investigation of the other pavement layers' properties.

5.10 FINDINGS AND CONCLUSIONS

In this study, through the performance of multiple laboratory tests, it was possible to analyze and clearly identify the strengths and weaknesses of four mixes with a high recycled content. Also, through the most advanced level of testing, it was possible to obtain new perspectives from which to gain a deeper understanding of the mixes' behavior. Overall, the following conclusions were drawn:

- The best performing mix in terms of cracking was mix 45_NR, which was characterized by a 45% RAP content as well as a 6.8% AC. The high AC proved to be very effective in counteracting the increased stiffness introduced by the high RAP content. However, this also significantly affected the rutting resistance, which was consistently the worst considering all the mixes.
- The best mix in terms of rutting was 30_LB (30% RAP and 5.6% AC). At the same time, this mix performed poorly in terms of cracking.
- Mix 30_HB, which, had the same 30% RAP content as mix 30_LB, but also included small changes in gradation and a higher AC (6%), achieved much better cracking resistance while

maintaining a great rut resistance. It was possible to observe how small design changes can greatly impact the mix's final performance.

- Mix 45_R, which shared the same gradation as mix 45_NR, featured the use of a rejuvenator that made it able to achieve good overall performance while requiring a lesser amount of AC (6.2%). In fact, while the cracking resistance was lower than mix 45_NR's, it was still much higher than the resistance of the 30% RAP mixes. At the same time, the rutting resistance was excellent and the fatigue performance was among the best as well.
- Through all the testing levels it was possible to rank the mixes similarly, with the only exception being part of the advanced level of testing. In fact, with respect to cracking, this was only true when the analysis was conducted looking at the strain-cycles to failure curves (Figure 66). The further calculation of the damage capacity index (S_{app}) indicated a different classification of the mixes. Also, for the rutting evaluation, with the calculation of the RSI, the best performing mix was different from what the other tests showed. These observations could be attributed to the recovery phenomenon of asphalt mixtures occurring during the rest period of the tests conducted using the AMPT.
- The use of the damage capacity index (S_{app}) suggested that fatigue cracking is a complex phenomenon in which different parameters, such as modulus and toughness, always need to be considered.
- In light of Tukey's HSD test, it was possible to observe how every test resulted in different performance groups. However, the best and worst mixes were consistently defined in the same way throughout all tests.
- The calculation of each test COV showed great repeatability for the HT-IDT and APA rutting tests. The CFN test showed increased variability as the test temperature increased. For the cracking tests, the COV values were generally higher, varying from 17% to 31%; on average, the COVs were equal to 19% and 25% for the CT_{Index} and I-FIT respectively.

5.11 RECOMMENDATIONS

Based on the findings of this research, the following recommendations were made:

- Based on all the tests' results, the use of 45% RAP together with a rejuvenator showed great promise for cracking and rutting resistance.
- The use of high RAP content, if combined with high AC to counteract the stiffness introduced by the recycled material, could result in poor performances in terms of rutting. Particular

attention should be paid to the selection of the right strategy to support high recycling volumes, considering the effects on both cracking and rutting. Furthermore, partial requirements on volumetric and aggregate gradation properties (e.g. restriction on minimum design air void and VMA) should be considered when designing high-RAP mixes with relatively high AC contents to avoid any potential flushing and bleeding.

- While the changes in the mixes' composition were clearly and consistently reflected in improvements and degradations of the performance at the base and intermediate testing levels, the advanced tests showed how the impacts of these changes could be different in different testing environments. The experimental plan presented in this study should be extended to more mixtures to further support the selection of performance tests that would better support the operations of mix design and quality assurance.
- The information presented in this study related to only the laboratory performance of the mixtures. Additional insight would be gained by comparing the laboratory performance with performance of constructed test sections, trafficked under controlled conditions, in the field.

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CHAPTER 6 – SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY

This dissertation had two main goals: (1) the production of optimized high-RAP content mixtures capable of outperforming traditional mixtures, and (2) the evaluation of how performance-based mix design procedures can support the use of high amounts of RAP.

Through the review of the state-of-the-art in pavement recycling, it was possible to take into consideration many different aspects of the process. Since the first versions of mix design procedures, the goal was to provide a tool that would allow HMA producers to obtain mixtures capable of resisting various types of distresses. Over the years, other factors came into play such as economic crisis and increased environmental awareness, and RAP became an essential component of all asphalt mixtures. Multiple studies have investigated the properties of the recycled material with the objective of deriving the maximum benefit from it. At first, asphalt mixtures were heavily regulated by requirements on the final volumetric properties of the mixes as well as the quality of aggregates and binder. As recycling became more popular, adjustments were made and new techniques (e.g. the use of softer binders and rejuvenators) were developed to include RAP. Nonetheless, lack of confidence in the RAP properties and strict specifications have prevented a more appropriate use of the recycled material and still today represent a limitation.

On the other hand, the development of many laboratory tests has allowed the asphalt industry to consider the use of performance tests at the design stage and it has been observed how it can support a larger employment of recycled mixes. The concept of balanced mix design was introduced and, with it, the idea that asphalt mixtures could be designed in accordance with performance-based criteria and not necessarily with volumetric ones. With this approach, it has been possible to obtain highly recycled mixes that also performed at a high level.

Over time, it became clear how the use of RAP does not affect a mixture's performances in a predefined and certain way. To a certain extent, recycling pavements at high rates still represent a risk for agencies. To reduce this risk, there is a very wide availability of tests that could be selected to support the design and characterization of recycled mixes. Therefore, one of the main ways for agencies and asphalt producers to utilize the full potential of RAP is through the employment of performance tests at the design stage.

In this framework, the dissertation focused on the investigation of high-RAP surface mixes, through the conduction of three different studies:

- The first study used both laboratory tests and an APT experiment to evaluate the rutting potential of an optimized surface mix with 26% RAP content. The optimization was made with reference to a traditional mix, to which the rutting performance was compared. The optimized mix featured a lower compaction energy and a different aggregate structure.
- The second study produced and compared the performances of highly recycled mixes (30% and 45% RAP contents) through a balanced mix design procedure. Two mixes were designed with the traditional volumetric design system while two more were designed following a performance-based design approach. The study established the potential of the balanced mixes for outperforming the traditional mixtures and evaluated their economic feasibility.
- The third study aimed at comparing both the rutting and cracking performances of four highly recycled surface mixes (30% and 45% RAP contents) through a variety of tests. The study allowed to characterize the mixtures and assess their performance, and to investigate various tests to establish their possible role in a performance-based design and quality control procedure.

6.2 FINDINGS

The research has led to the following findings:

- With reference to a traditional mixture, an optimization process that includes a lower number of design gyrations and a different aggregate structure can result in a mixture that presents better rutting performance even with a higher content of asphalt binder.
- Meeting the Superpave mix design requirements alone, with particular reference to gradation limits and volumetric properties, does not guarantee satisfactory performance in terms of both cracking and rutting resistance.
- Performance-based mix design methods can produce surface mixes with high recycled contents capable of achieving better overall performances (with respect to cracking and rutting resistance) than mixes that contain the same amount of RAP but are designed only considering volumetric mix design criteria.
- The typical behavior associated with RAP inclusion, namely higher RAP contents correspond to lower cracking resistance and higher rutting resistance was observed. However, it was also possible to observe that, with specific solutions (e.g. higher final contents of asphalt binder and the use of rejuvenators), this behavior can be changed.

- Even if high RAP contents may require high contents of virgin asphalt binder to achieve satisfactory cracking resistance, the impact on rutting performance can be very limited.
- The optimization process through a balanced mix design procedure showed the potential to achieve significant economic savings. High-RAP content (45% by weight of the final mixture) surface mixes designed controlling the cracking resistance, through the CT_{Index} test, and the rutting resistance, through the APA, resulted in lower production costs and presented better laboratory performance than the traditional mixes.
- Simple laboratory tests, such as CT_{Index} and HT-IDT, showed the same ranking capabilities, for both cracking and rutting performance, as fairly more complex tests such as I-FIT and APA. An agreement was also found with respect to the tests conducted with the AMPT; more specifically, S-VECD test for cracking and CFN and SSR tests for rutting.
- Considering Tukey's HSD test, it was possible to observe how different tests produced different performance groups. The test considered were CT_{Index} and I-FIT for cracking and HT-IDT, APA, and CFN for rutting.
- CT_{Index} recorded a lower COV than I-FIT (19% and 25% respectively); also, HT-IDT recorded a lower COV than APA and CFN at 50°C (7%, 10%, and 13% respectively).
- The use of higher RAP contents (45% by weight of the final mixture) reached a better overall compromise between the impacts on rutting and cracking performances when associated with the use of a rejuvenator rather than a higher AC.

6.3 CONCLUSIONS

Based on the work presented in this dissertation, it can be concluded that current mix design techniques could greatly benefit from the addition of simple performance tests, such as the CT_{Index} test for cracking resistance and the HT-IDT or the APA for rutting resistance. Both these factors have shown the potential to support the improvement of current HMA surface mixtures' performances and facilitate the increase of the current recycling levels.

Throughout the three studies presented, it became clear that the mixtures designed in accordance with the current volumetric requirements could be significantly improved.

- Both full-scale (APT) and laboratory testing showed no indication that optimized mixtures, designed with a lower compaction energy, would have rutting problems, supporting the implementation of a reduction of the current design gyration levels.

- The transition to a performance-based methodology can allow the use of more RAP and improve the performances at the same time. Through laboratory performance tests, it would be possible to identify the best strategies to increase the RAP contents without jeopardizing the final performance. Furthermore, performance-based mix design methods showed the potential of achieving significant economic savings (by both the reduction of the production costs and the extension of a mixture's life span).
- There is a wide selection of performance tests that could be included in a mix design system, however, in light of the results of extensive laboratory testing, a very good agreement was reported between tests of varying complexity, testing time, and equipment cost.

6.4 SIGNIFICANCE

The design of sustainable road infrastructures is constantly evolving. While practices such as recycling have been introduced a long time ago, their inherent variability and project-specific impacts prevented their transition to standardized routine procedure. Over the years, pavement engineers developed multiple tools to support their designs, such as a very wide selection of laboratory tests and machines like the HVS capable of verifying the field performances of diverse pavement structures. It became clear how these instruments could be used both to enhance the performances of traditional pavement materials and to constitute a support for innovative asphalt mixtures. With particular reference to recycling, laboratory tests emerged as one of the best tools to reduce the uncertainties related to the inclusion of high quantities of RAP as well as the use of rejuvenators and other mixture's modifiers.

While design methods such as Superpave and Hveem paved the way to the implementation of reliable and predictable asphalt mixtures, today the transition to a performance-based mix design represents the necessary step to extend the same level of confidence and knowledge towards pavement recycling.

Throughout the whole dissertation, it was possible to observe how pavement recycling, especially at high rates, cannot be linked to a unique set of rules and design methods. It is instead necessary to support the recycling process with a flexible design tool, such as a performance-based procedure, which can provide reliable project-specific information. The experiments conducted confirmed the potential of using performance tests, even simple ones, to improve current practices and improve the final field and economic performances.

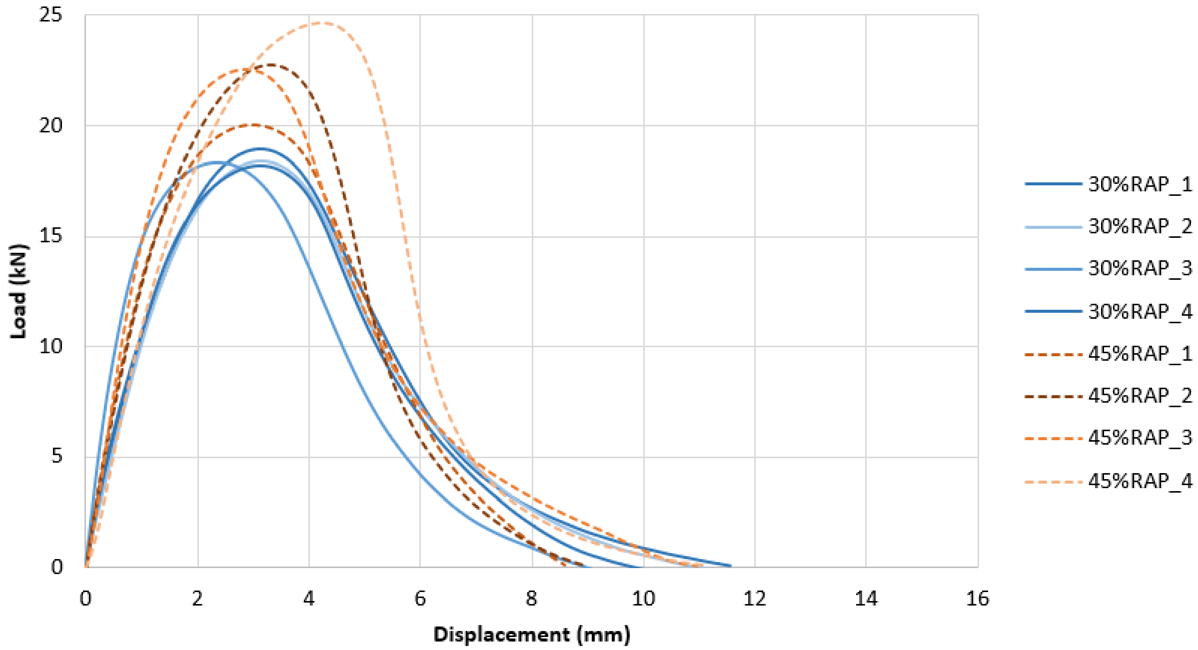
6.5 RECOMMENDATIONS FOR FUTURE RESEARCH

- The selection of appropriate laboratory tests is fundamental for the effectiveness of a performance-based mix design methodology that supports the operations of mix design and quality control. In particular, the three-level experimental plan presented in Chapter 5 could be extended to more mixtures, in order to further check the features and compare the discerning capabilities of each test.
- The degree of blending between RAP and virgin aggregates greatly affects the final performances of a given mixture. The effect of the RAP quality and, more specifically, of the RAP binder blending capabilities on the performances of highly recycled mixes should be further investigated. In particular, mixtures that share the same job-mix formula but include RAP from different sources could be analyzed and compared.
- The evaluation of field performances and APT programs, such as the one underway at the Virginia Tech Transportation Institute, can complement the laboratory characterization of the surface mixtures and constitute a benchmark to check the agreement with the laboratory ranking and grouping of the mixes.
- Based on the laboratory and field performances, a Life Cycle Assessment should be conducted to estimate the environmental impacts related to the use of the recycled mixtures evaluated in this dissertation.

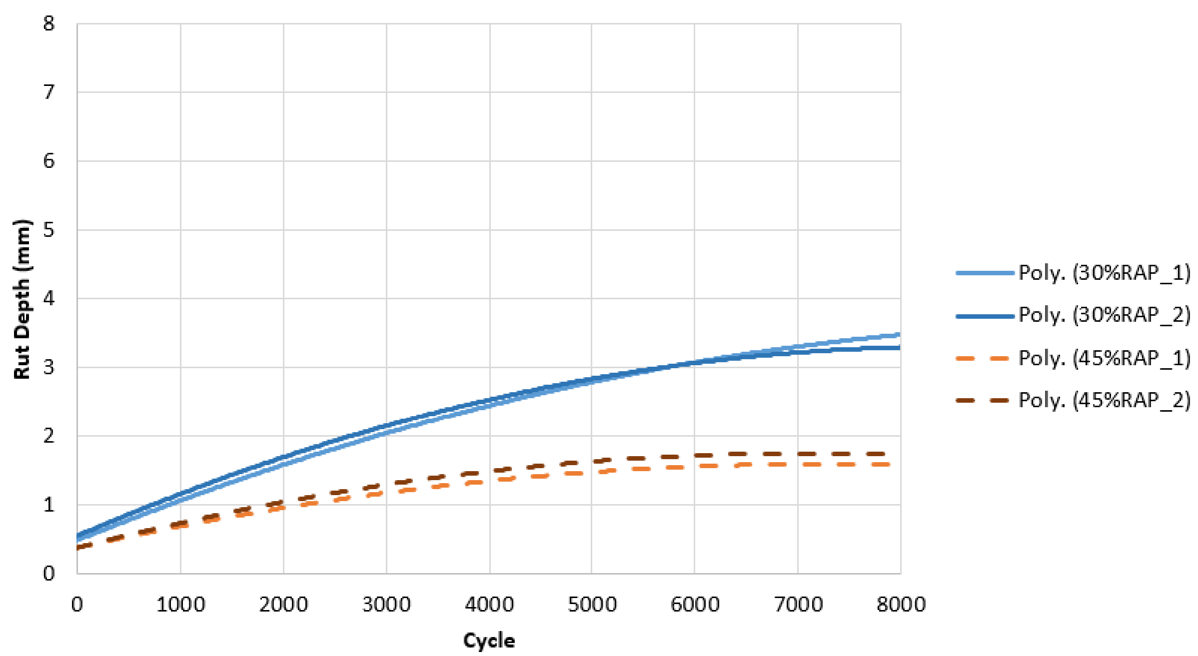
APPENDIX A: CHAPTER 4 – LABORATORY TEST RESULTS

A.1 CONTROL MIXES: 30-SUPERPAVE, 45-SUPERPAVE

CT Index

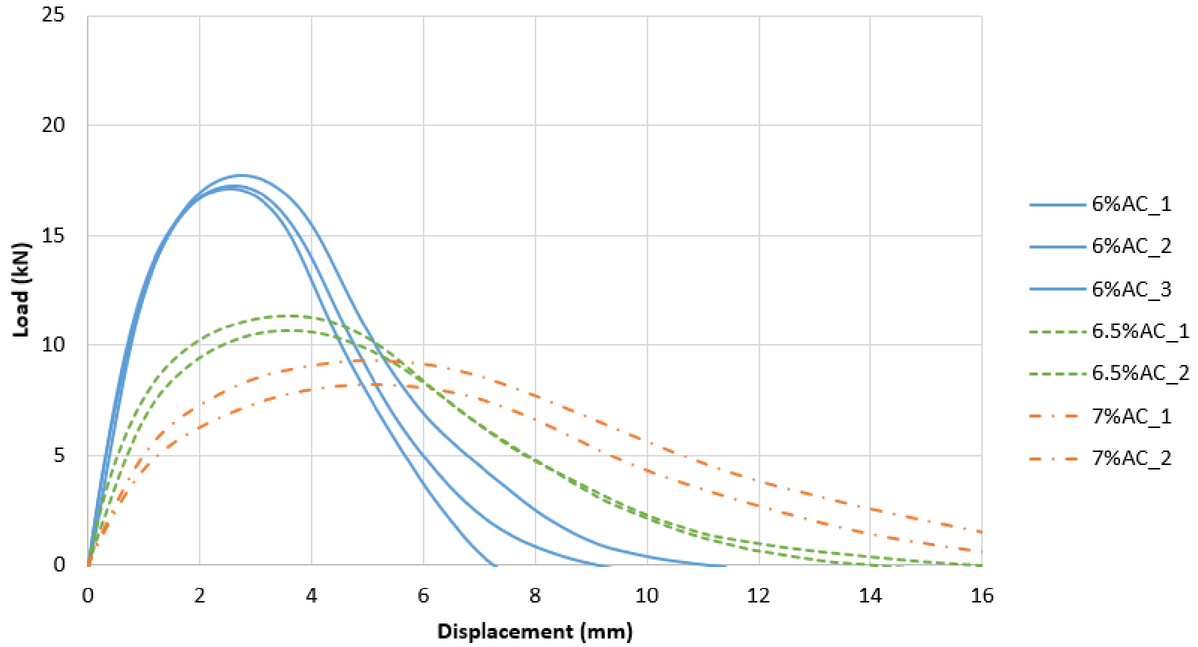


APA (2nd order polynomial trend lines)

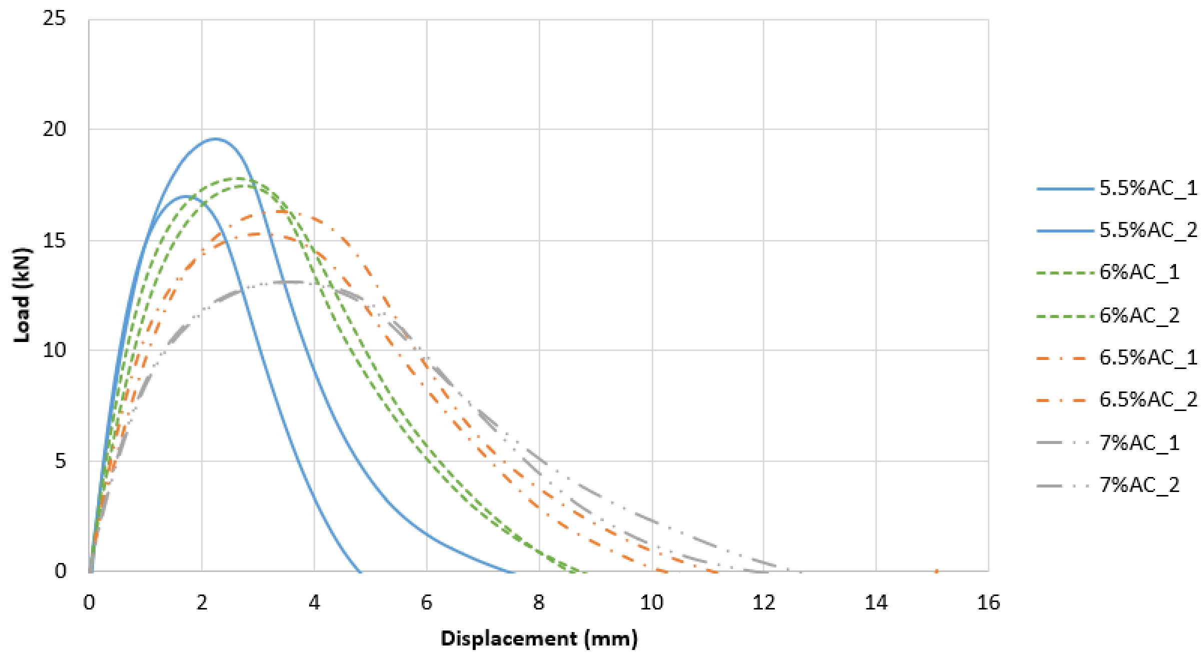


A.2 OPTIMIZED MIXES: 30-BMD, 45-BMD

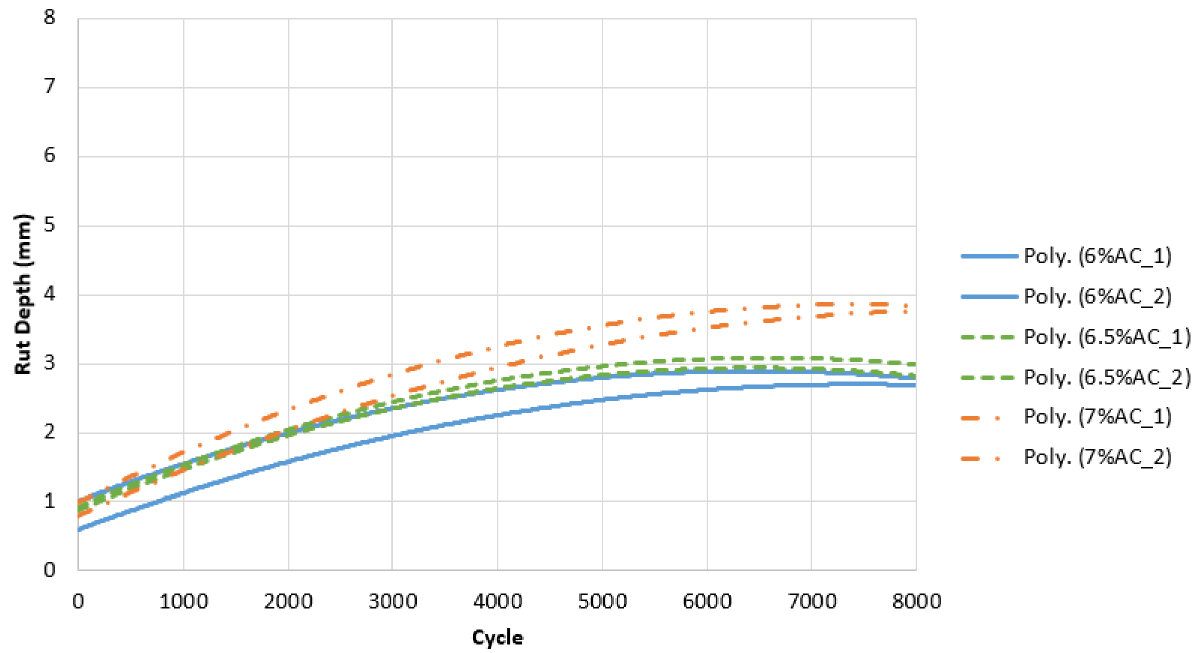
CT Index 30-BMD



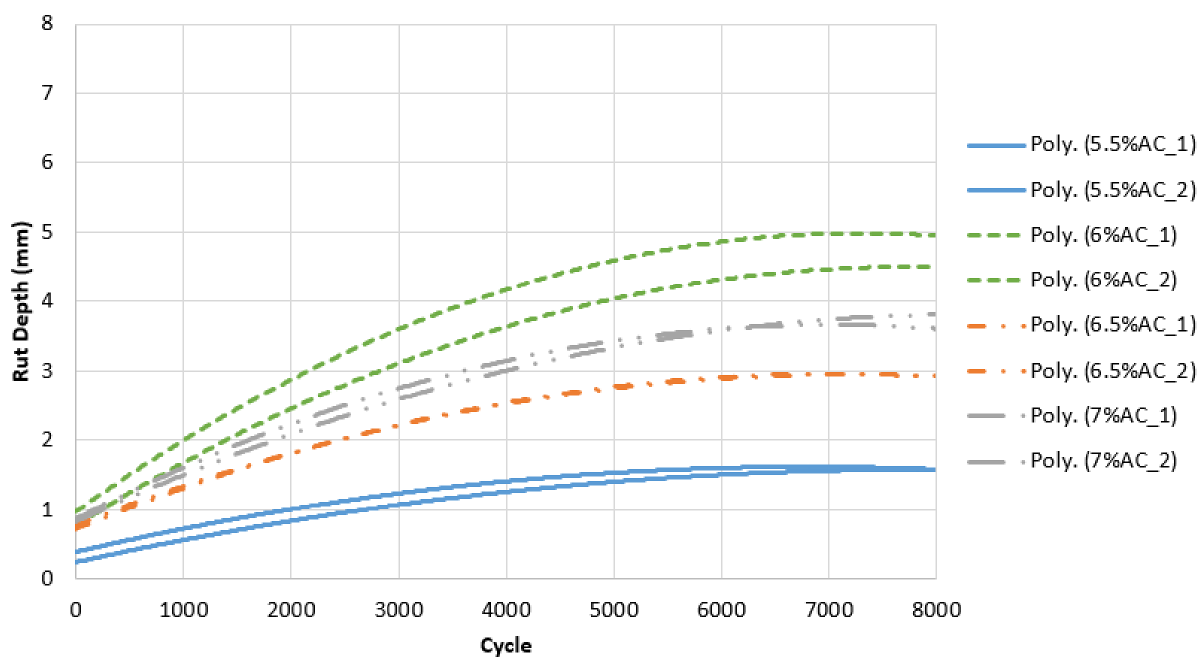
CT Index 45-BMD



APA 30-BMD (2nd order polynomial trend lines)



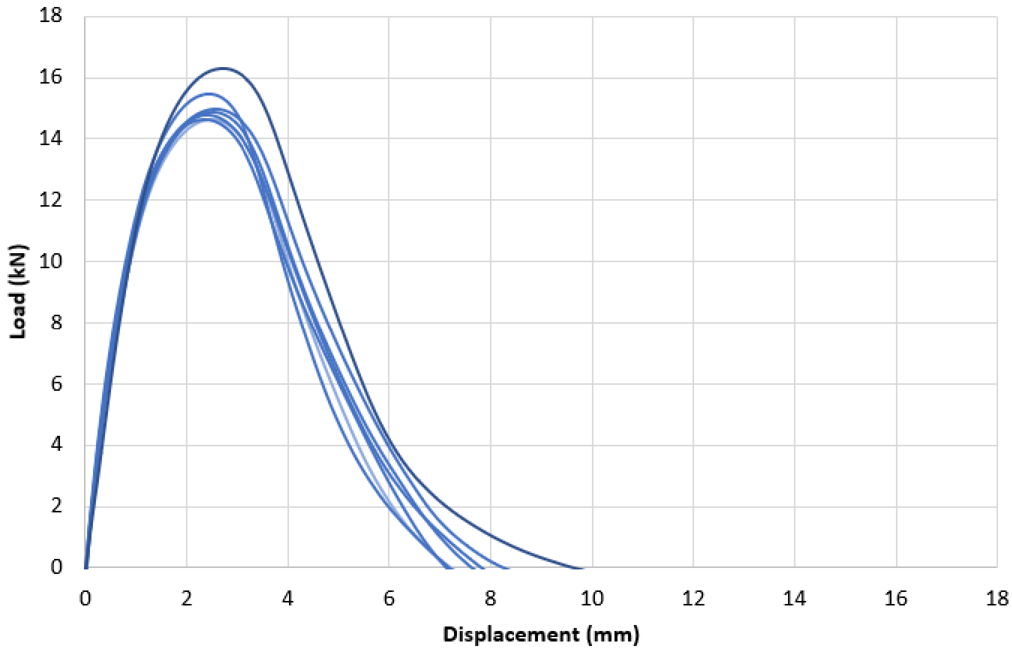
APA 45-BMD (2nd order polynomial trend lines)



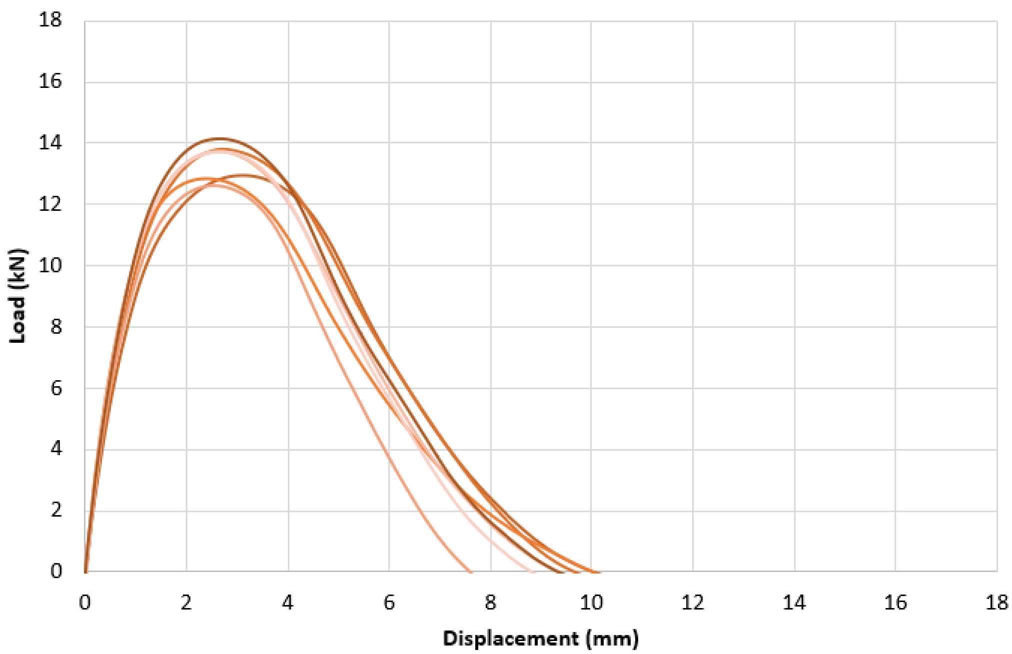
APPENDIX B: CHAPTER 5 – LABORATORY TEST RESULTS

B.1 CT INDEX

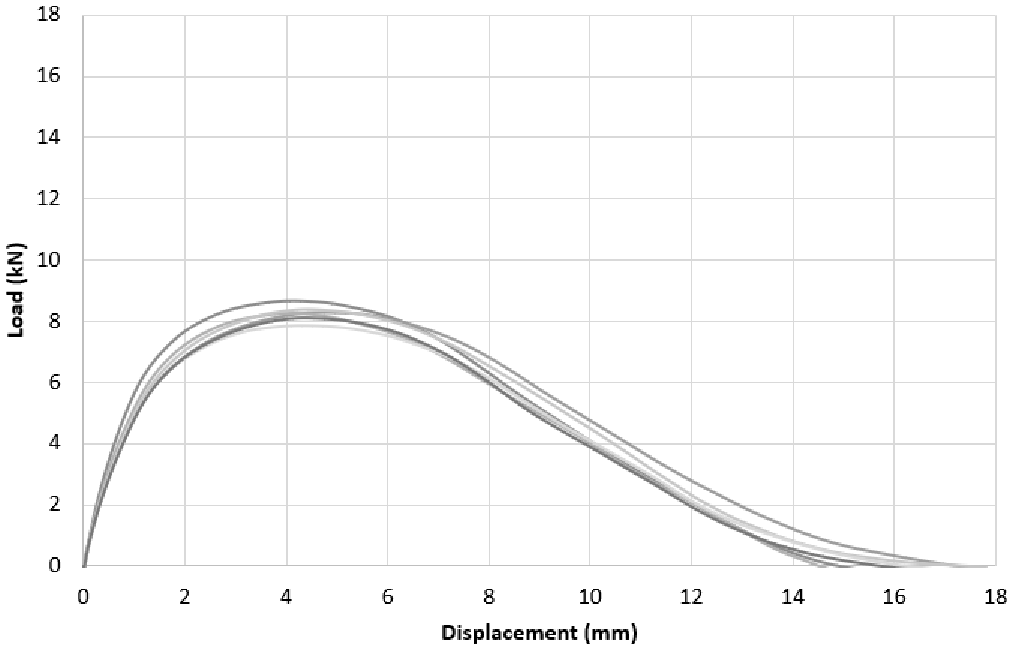
Mix 30_LB



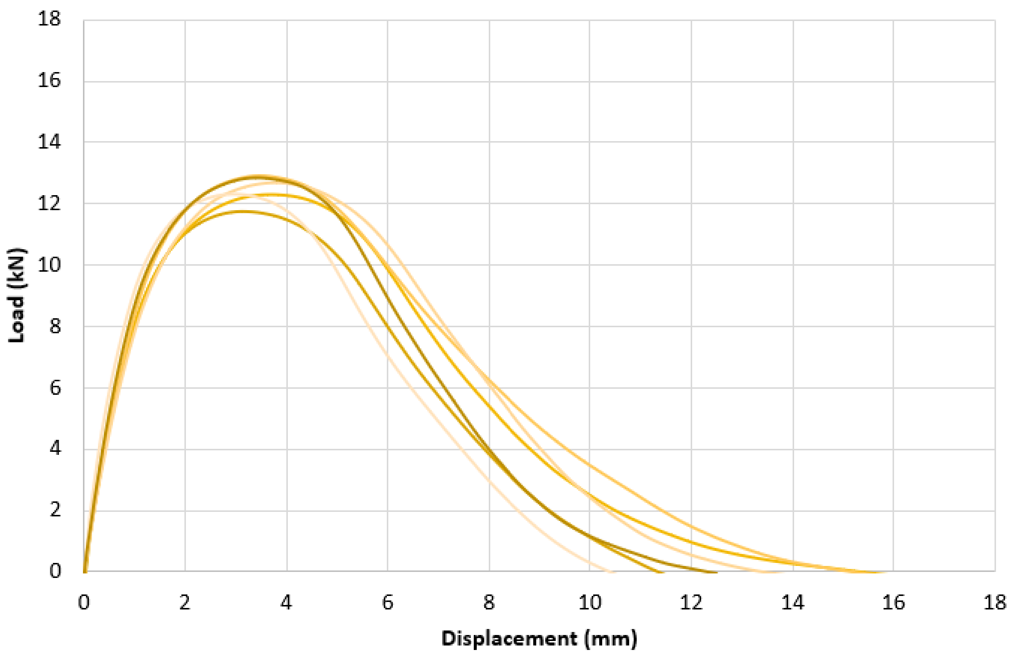
Mix 30_HB



Mix 45_NR

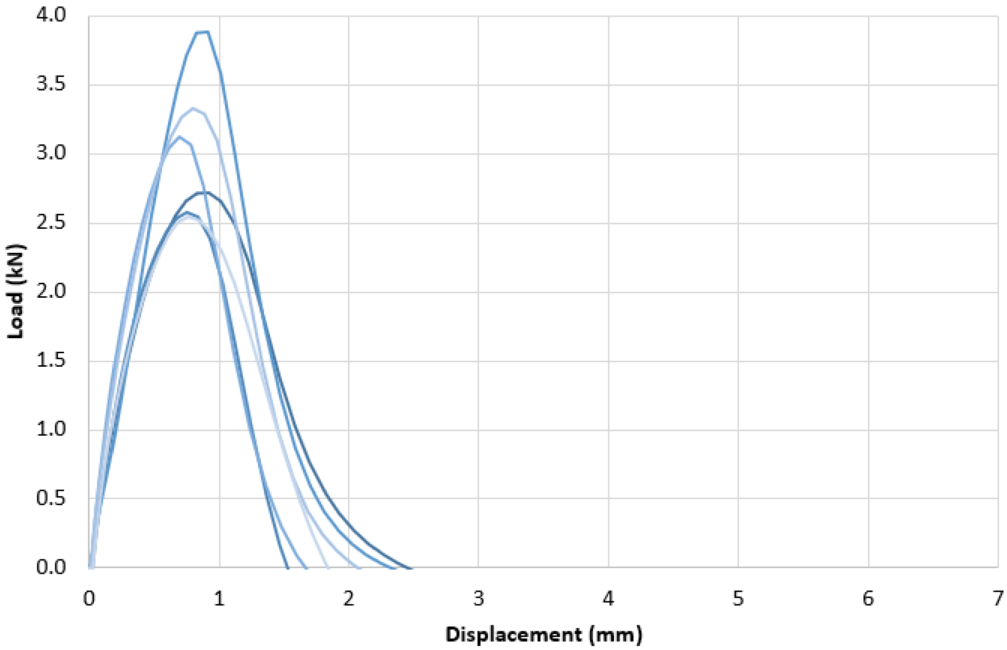


Mix 45_R

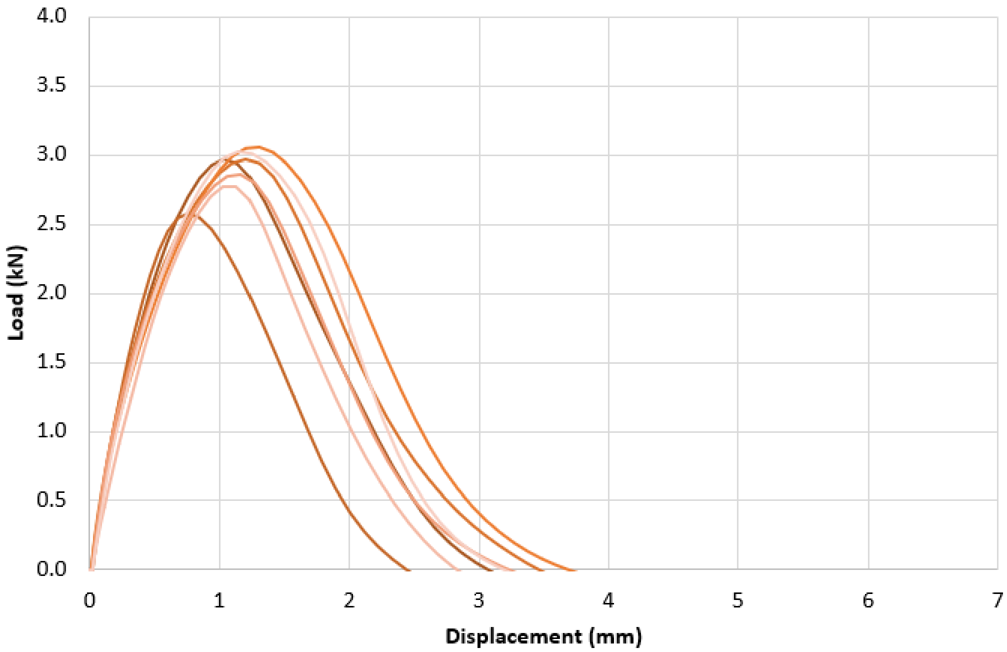


B.2 I-IFIT

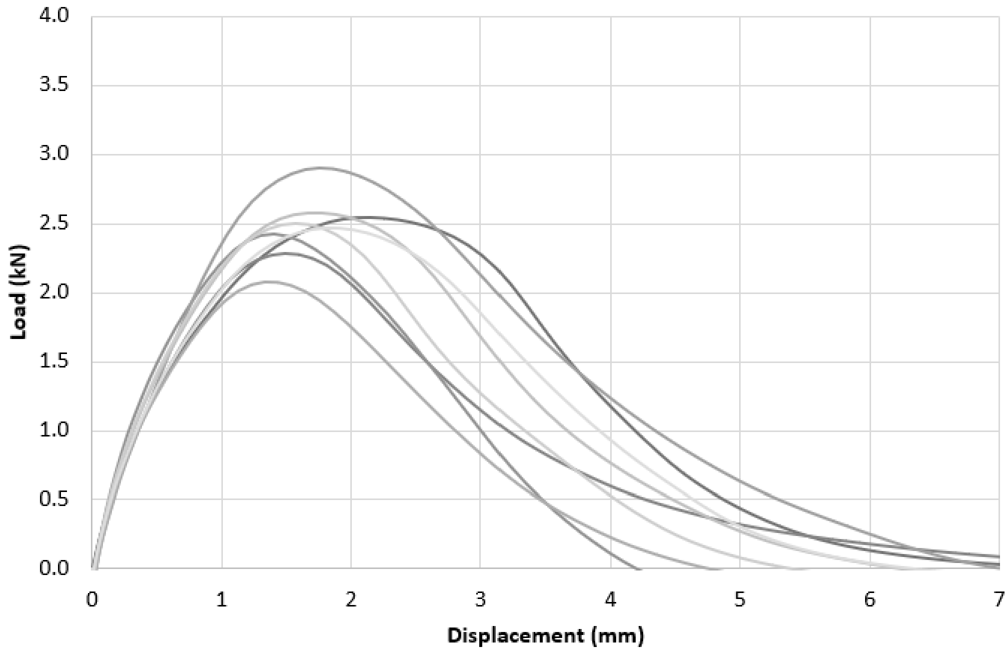
Mix 30_LB



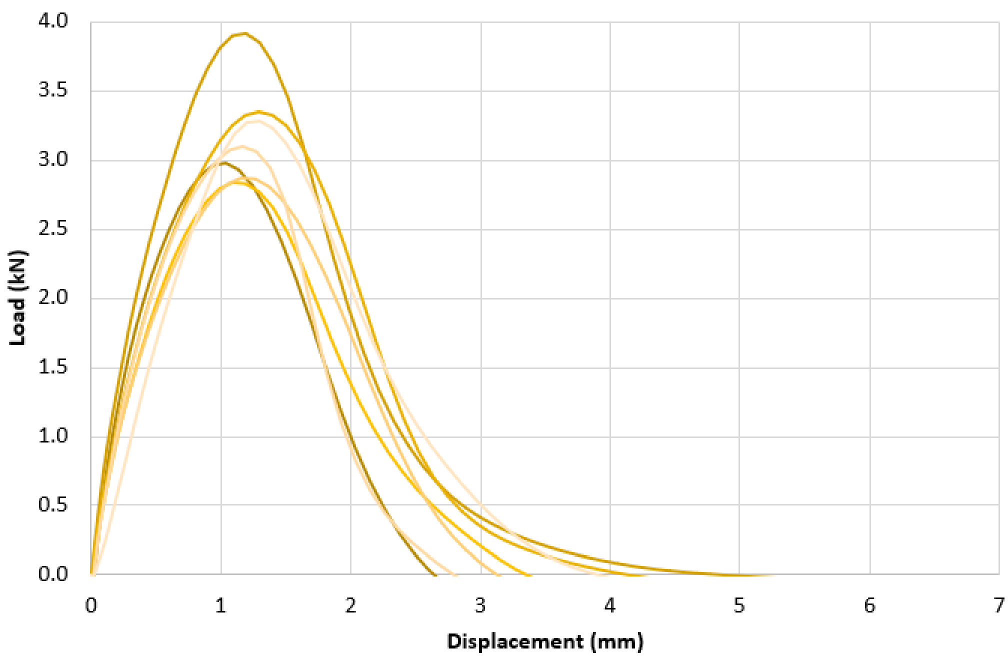
Mix 30_HB



Mix 45_NR

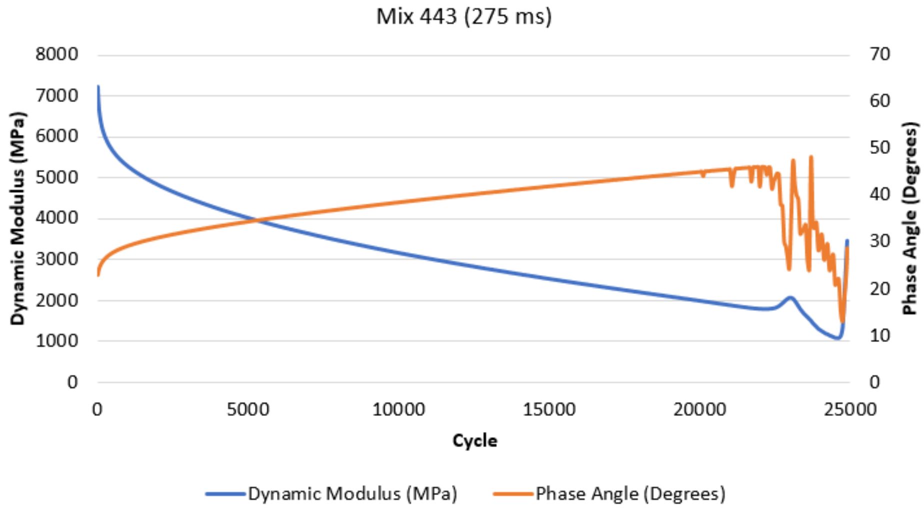


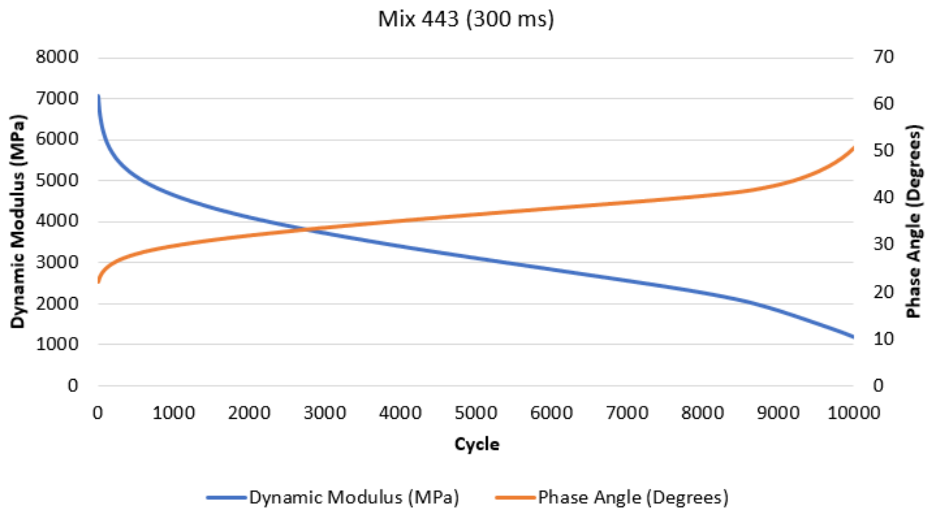
Mix 45_R

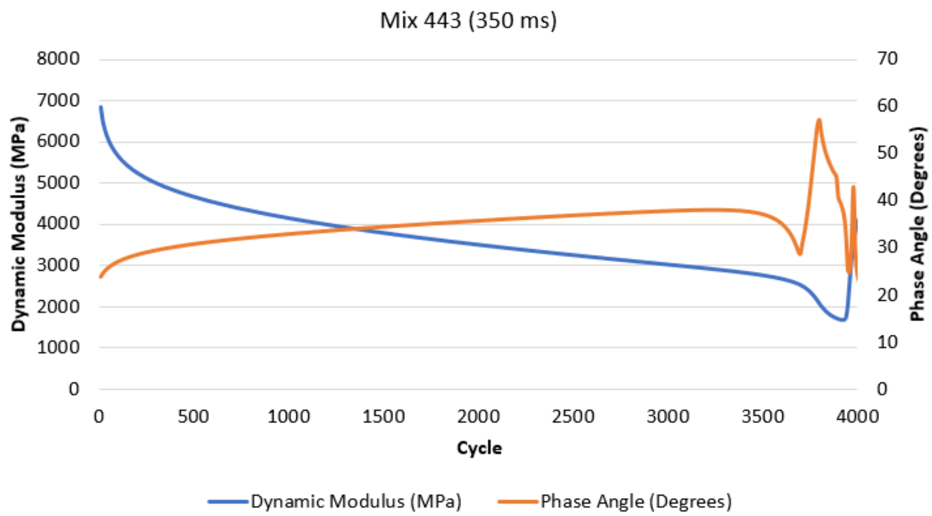


B.3 S-VECD

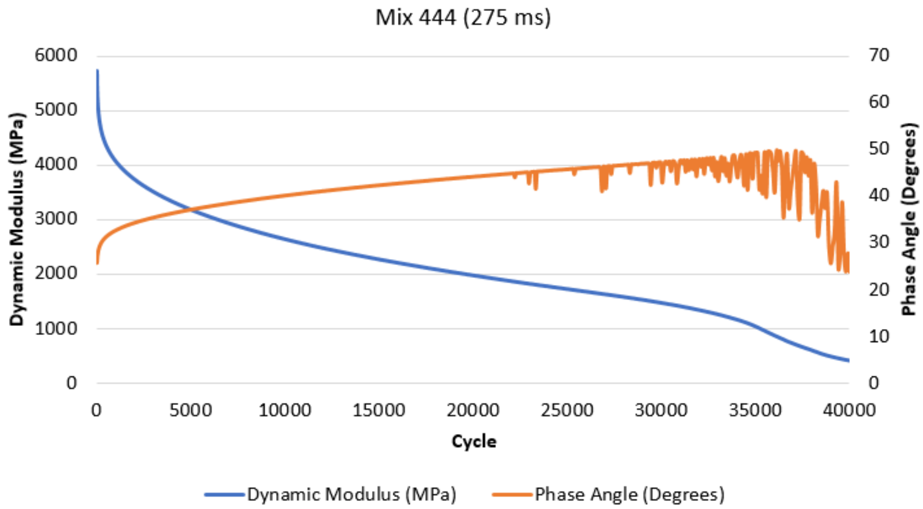
Mix 30_LB (275 ms, 300 ms 350 ms)

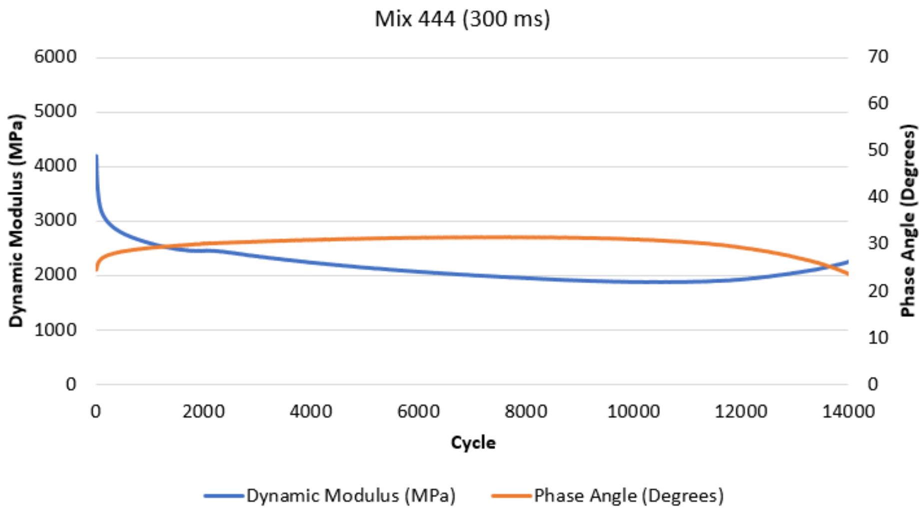


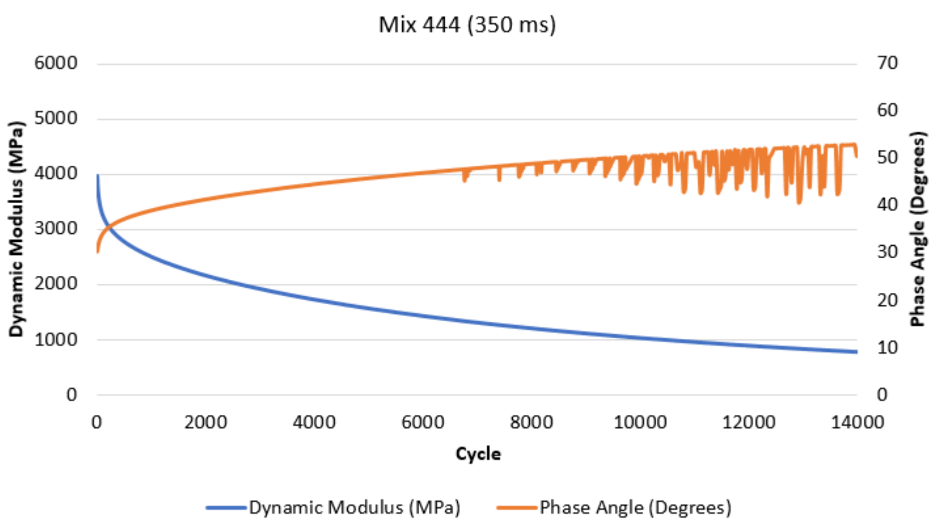




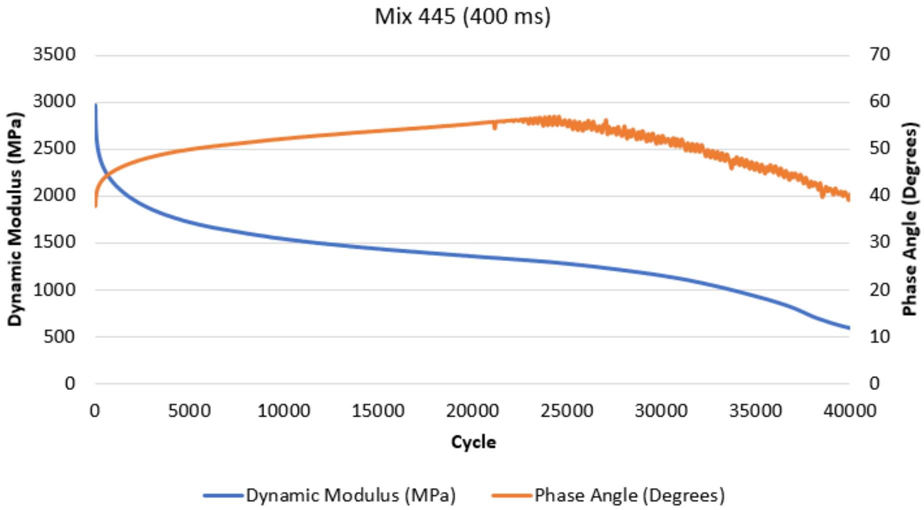
Mix 30_HB (275 ms, 300 ms, 350 ms)

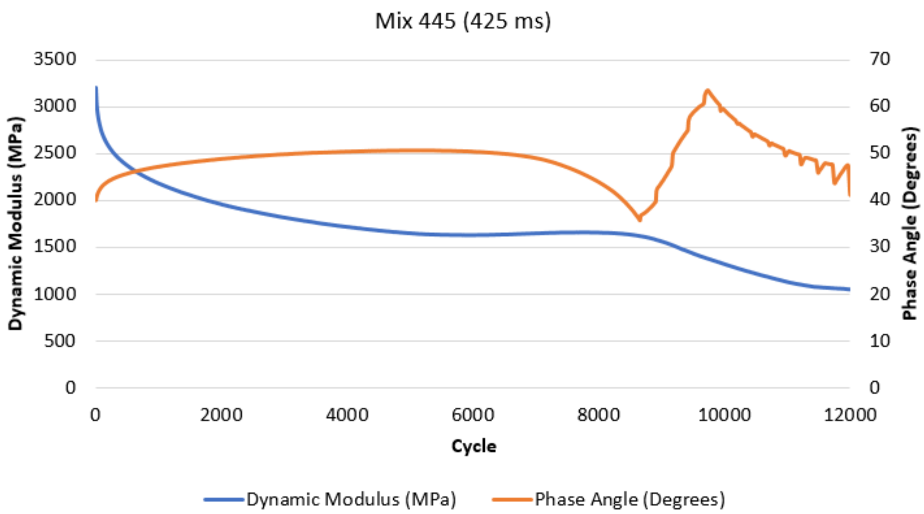


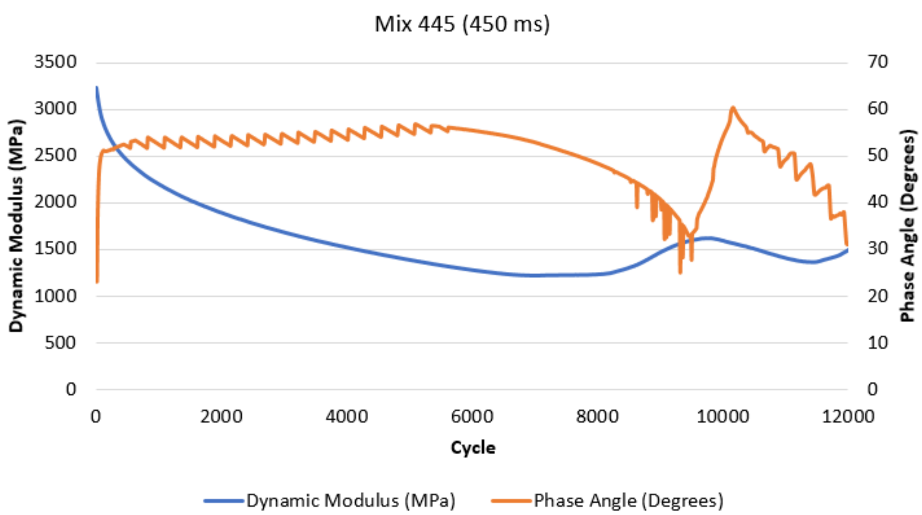




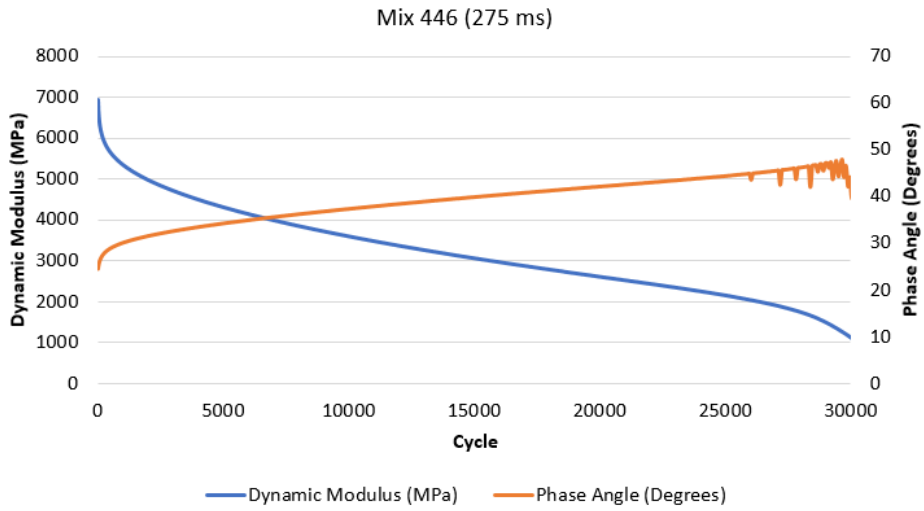
Mix 45_NR (400 ms, 425 ms, 450 ms)

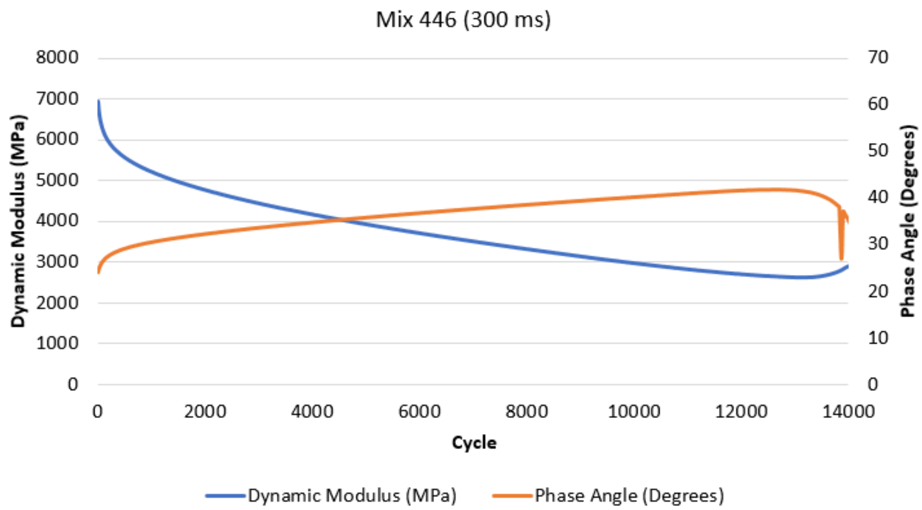


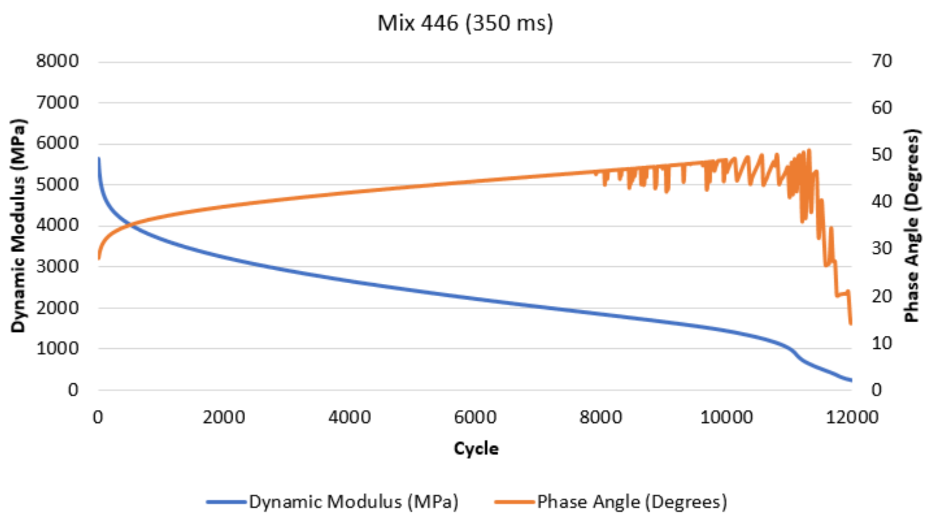




Mix 45_R (275 ms, 300 ms, 350 ms)

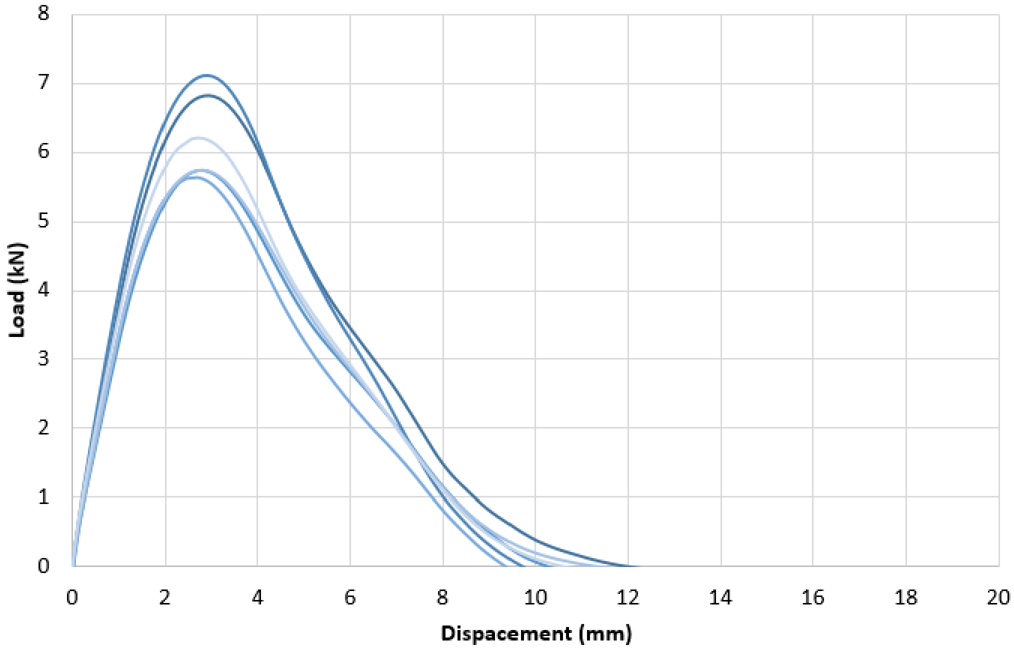




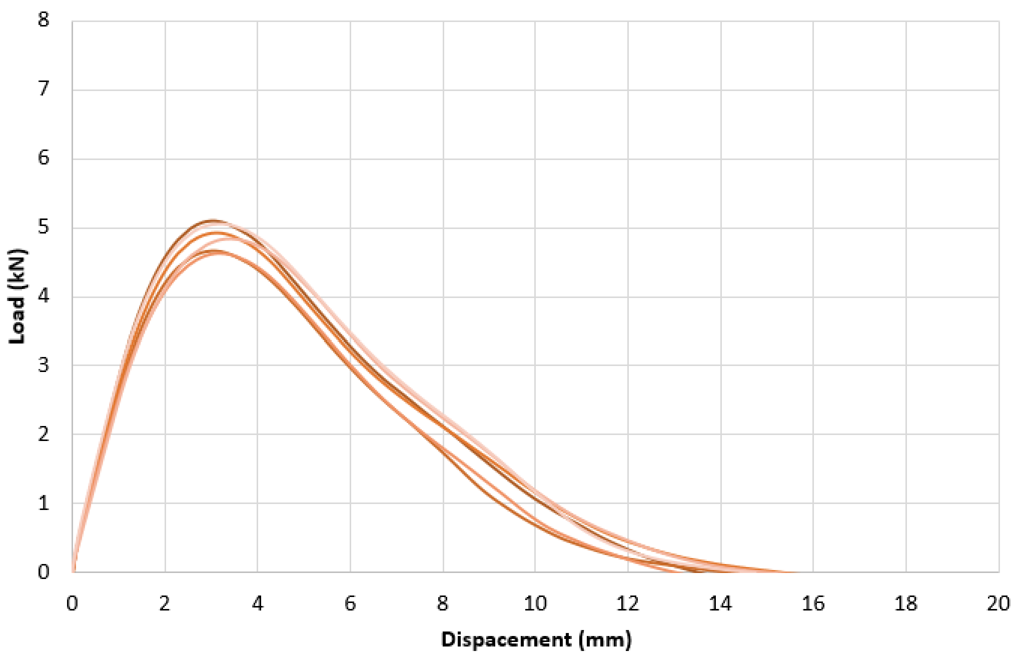


B.4 HT-IDT

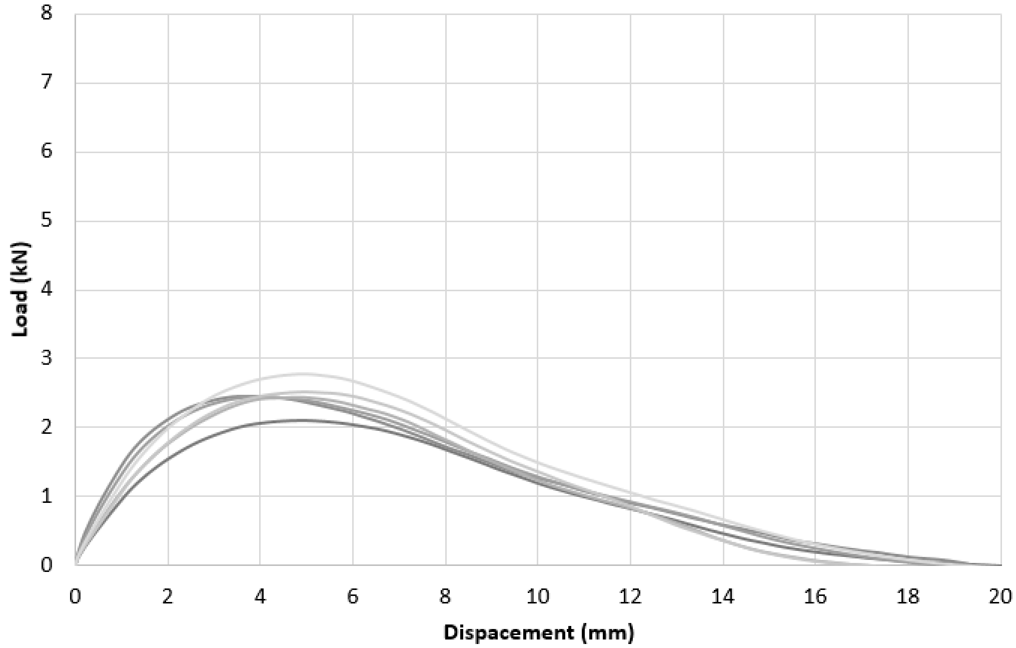
Mix 30_LB



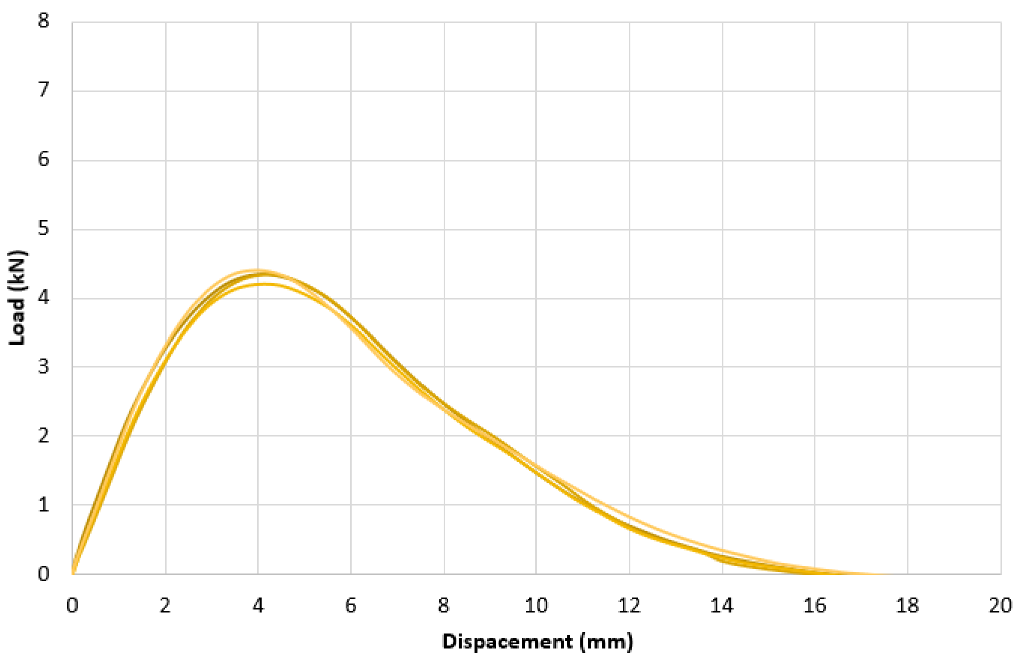
Mix 30_HB



Mix 45_NR

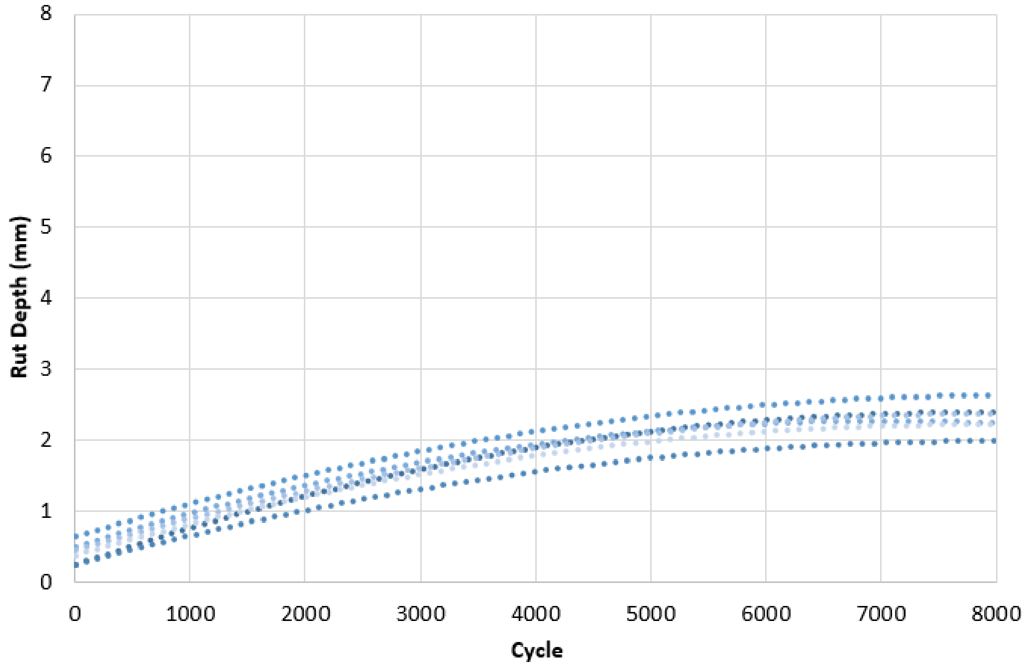


Mix 45_R

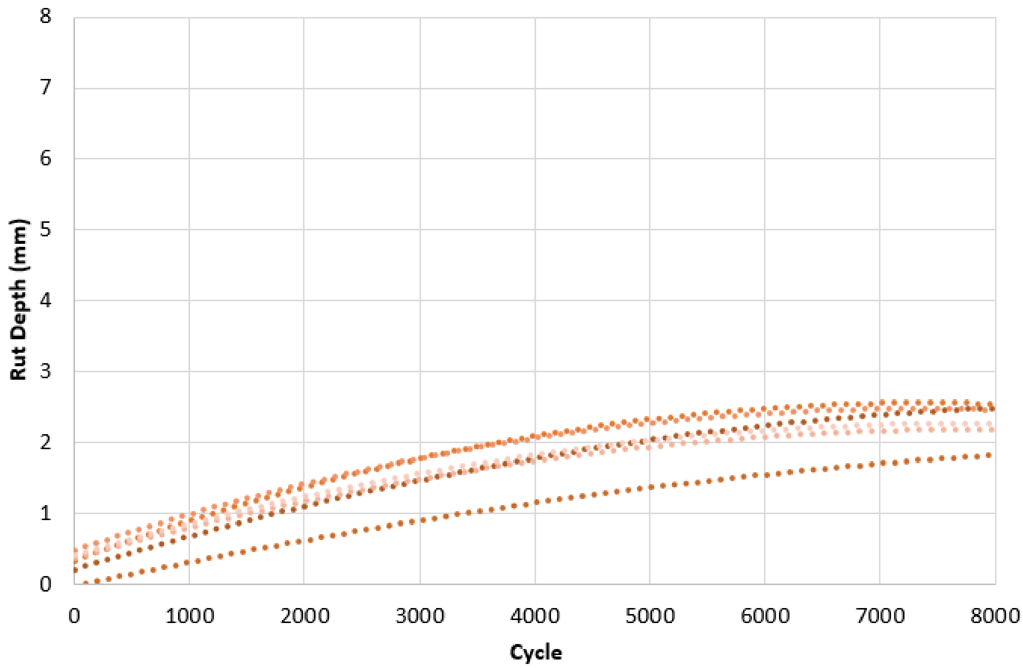


B.5 APA

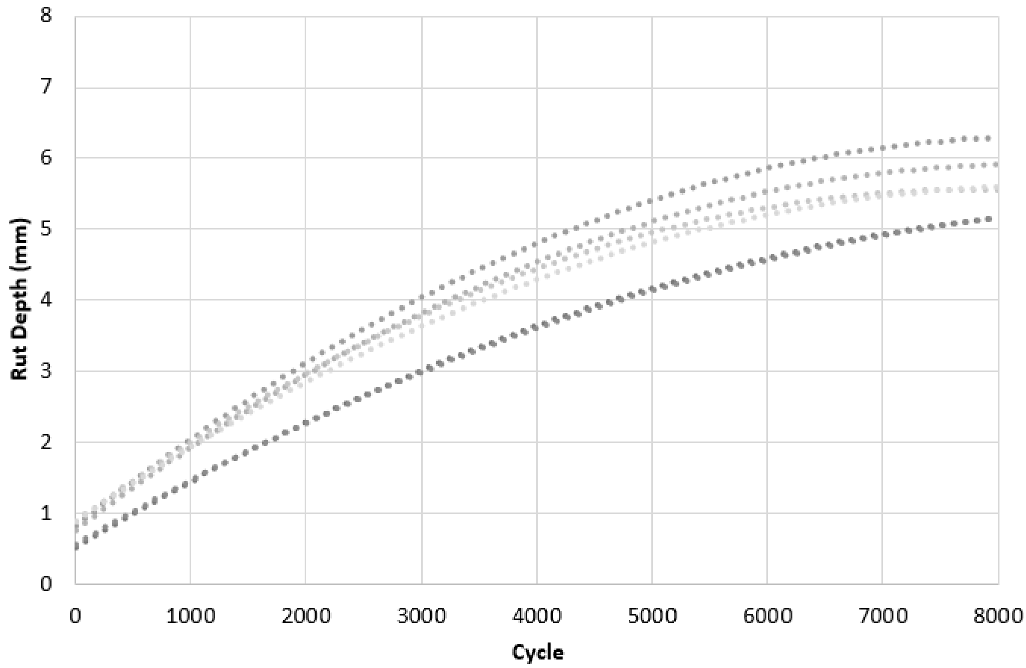
Mix 30_LB (2nd order polynomial trend lines)



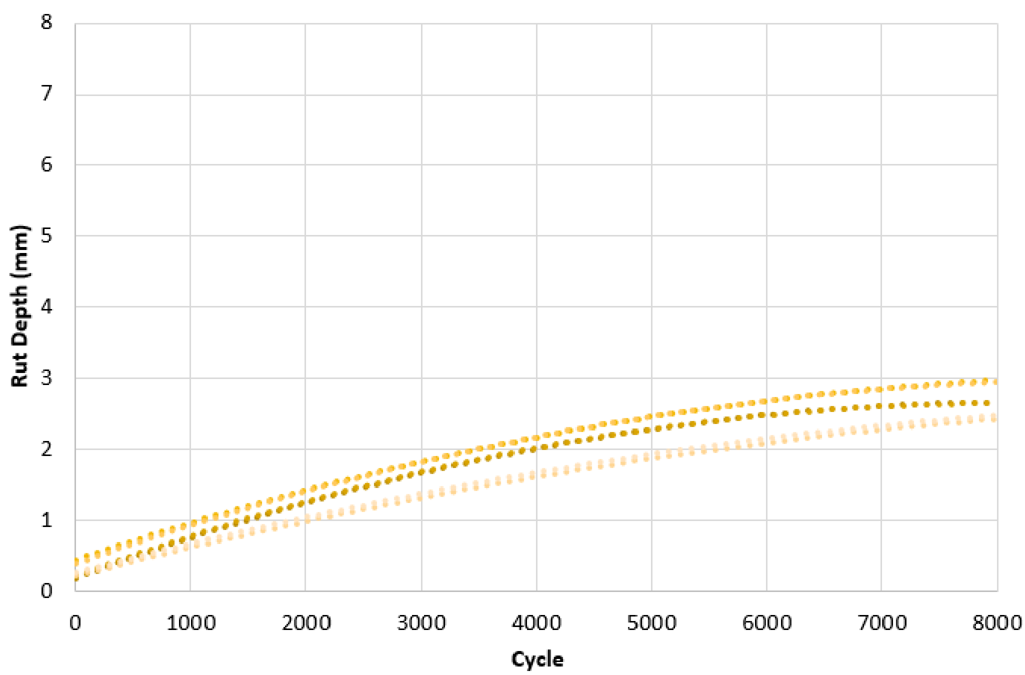
Mix 30_HB (2nd order polynomial trend lines)



Mix 45_NR (2nd order polynomial trend lines)

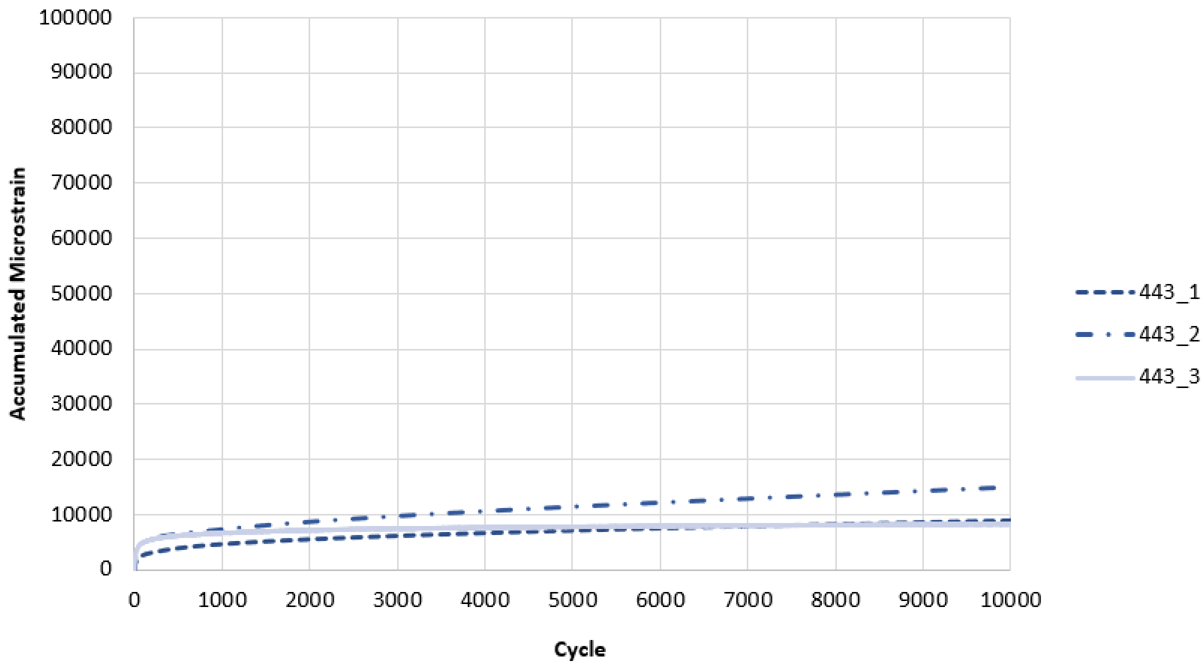


Mix 45_R (2nd order polynomial trend lines)

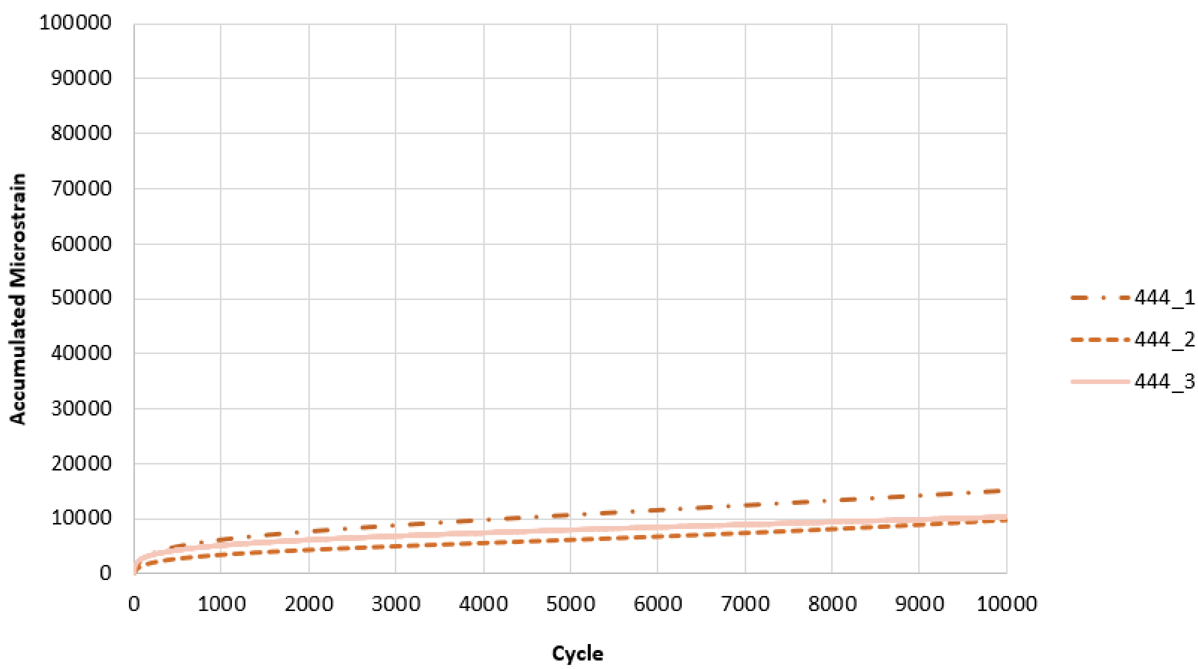


B.6 CFN 40°C

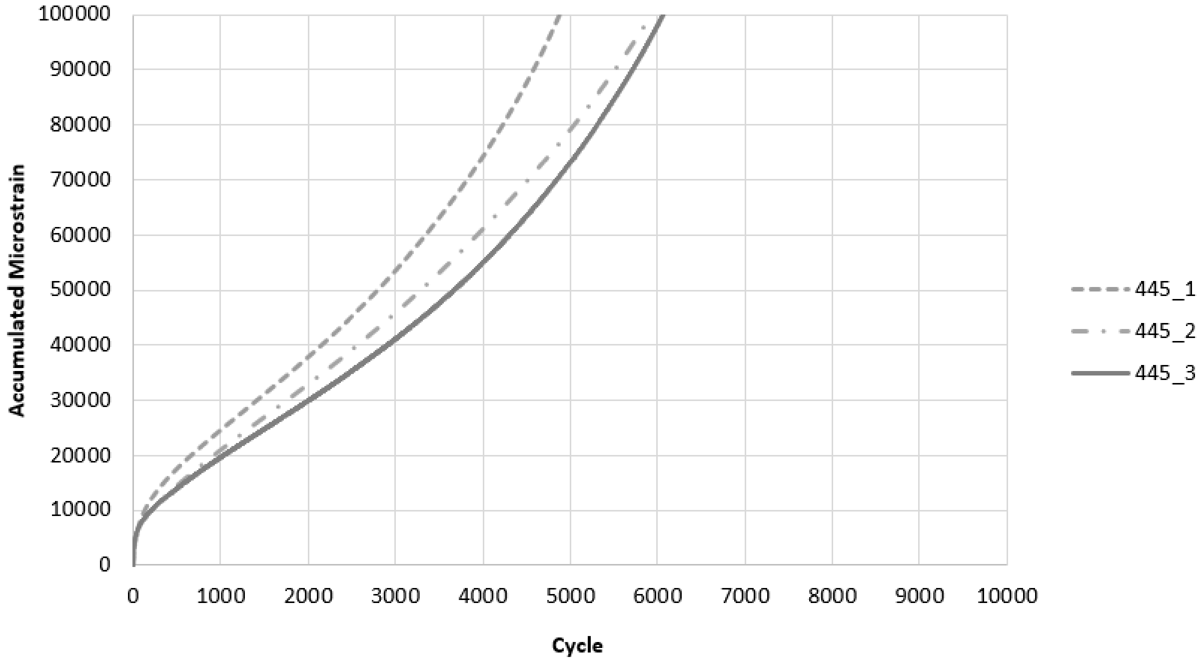
Mix 30_LB



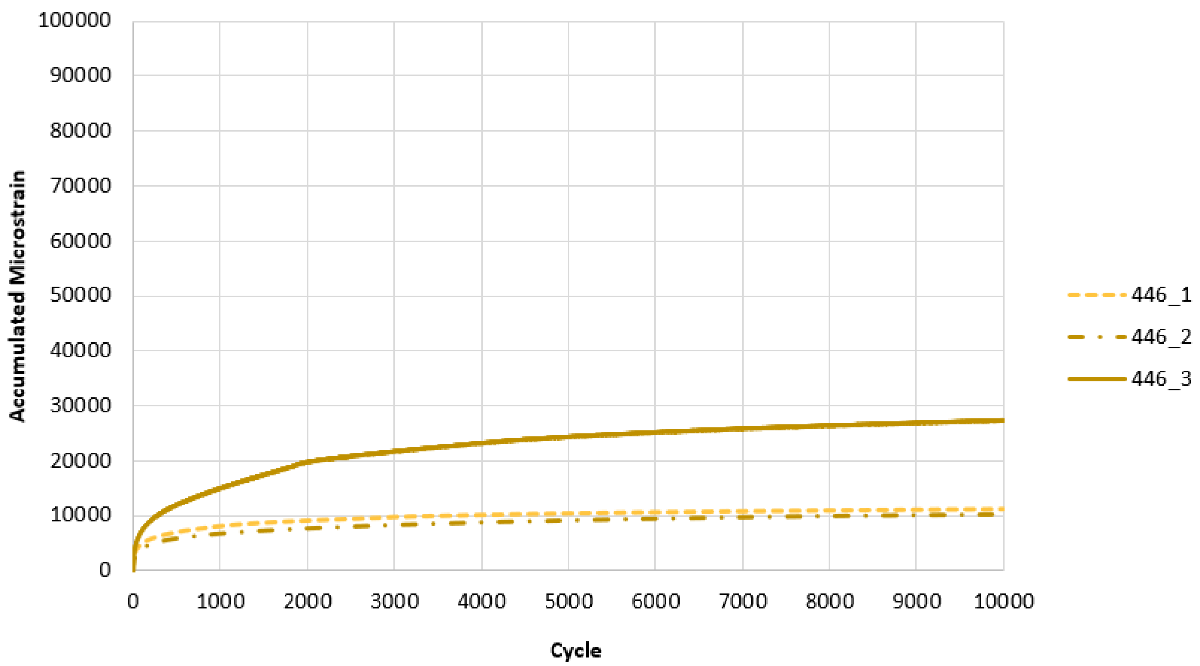
Mix 30_HB



Mix 45_NR

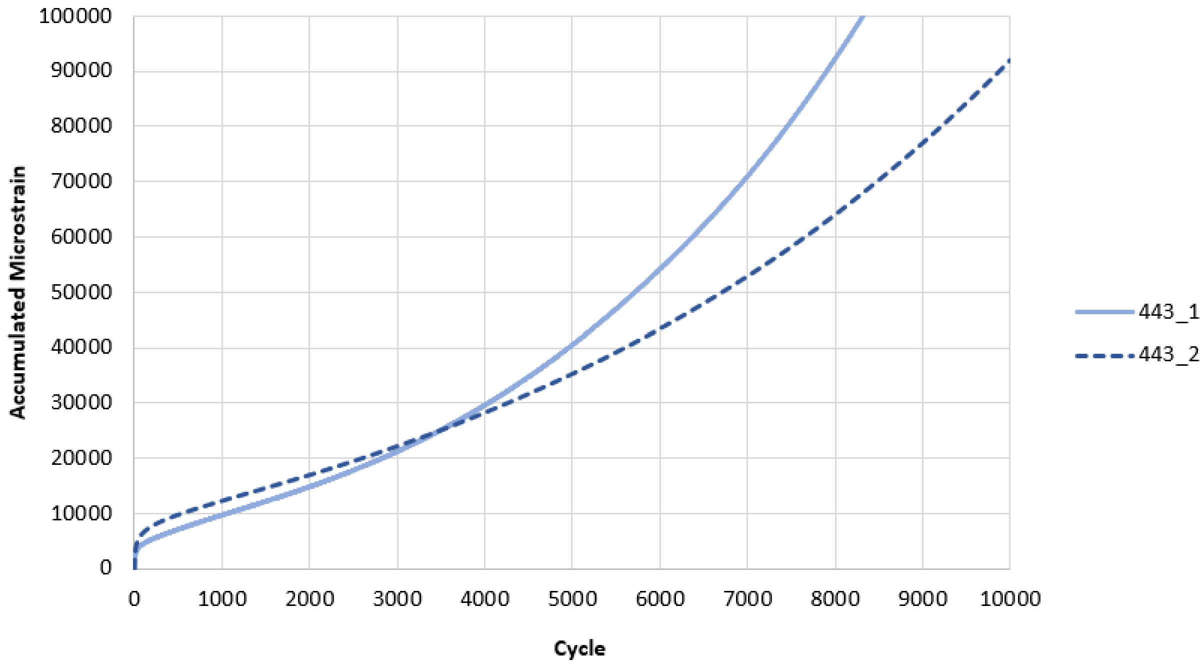


Mix 45_R

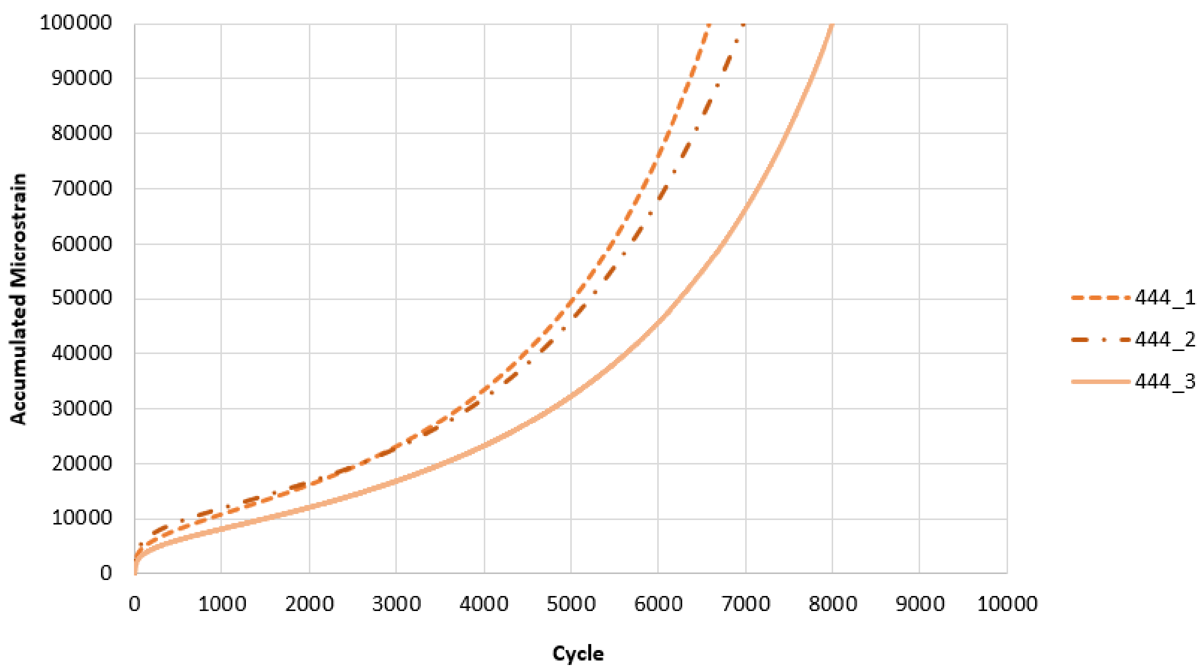


B.7 CFN 50°C

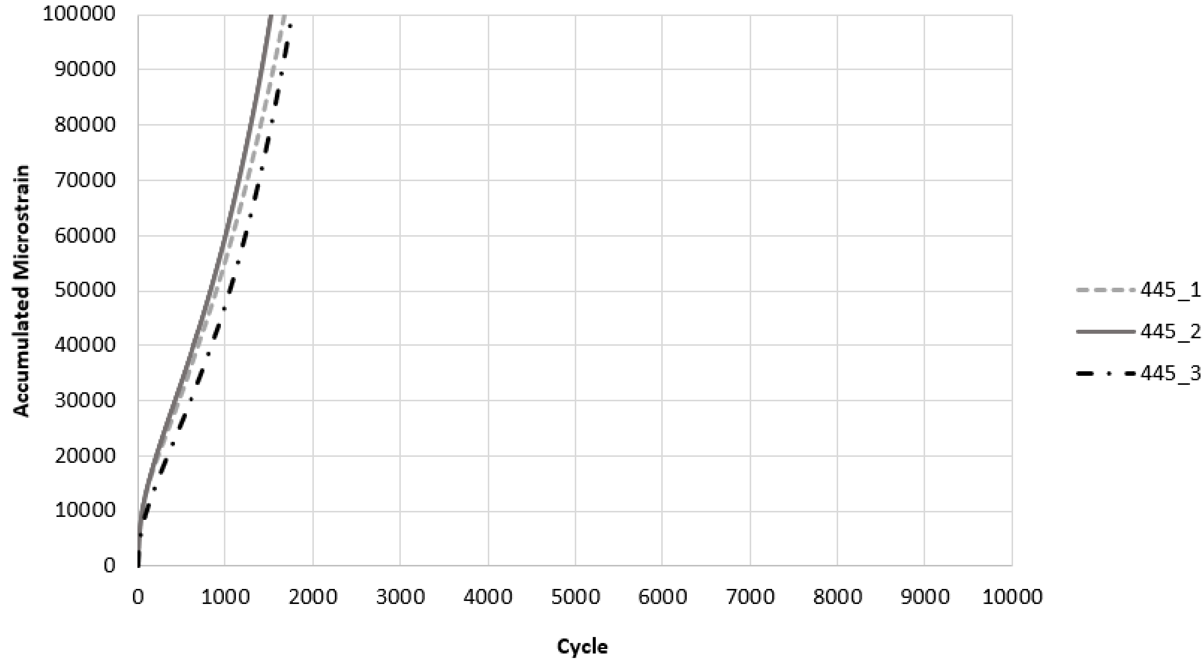
Mix 30_LB



Mix 30_HB



Mix 45_NR



Mix 45_R