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Improvement Of Pavement Monitoring For More Sustainable Management Of Road Pavements

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Engineering Structures, Transportation"
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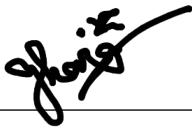
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Declaration

I hereby declare that this thesis, submitted in fulfillment of the requirements for the academic degree of Doctor of Philosophy (Ph.D.) in Advances in Modeling, Health Monitoring, Infrastructures, Geomatics, Geo techniques, Hazards, Engineering Structures, and Transportation (AIM HIGHEST), with a focus on CEAR-03/A – Roads, Railways, and Airports, is my own unaided work. I confirm that I have used only the sources explicitly indicated, and all direct and indirect references have been properly acknowledged.

Parts of this dissertation have been published in international journals and/or presented at conferences (see the list of the author's publications in the relevant section).

Palermo, July 18, 2025



USMAN GHANI

Dedication

To my father (late), mother, wife (Summaya Khalil), Jerry and Mini Qurban. . .

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First of all, I would like to express my deepest gratitude to my family—especially my wife and children—for their unwavering support and patience throughout the completion of this research work. After the passing of my beloved father, it was extremely difficult to return and continue this research. At times, I felt on the verge of giving up. But giving up was never an option. As the saying goes, you are never a loser until you quit trying.

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Usman Ghani

Palermo July, 2025



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(AIM HIGHEST)

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CEAR-03/A – Roads, Railways and Airports

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Abstract

The sustainable management of road pavements is a global priority, given their essential role in supporting economic growth and societal connectivity. However, deterioration caused by heavy traffic loads, environmental factors, and inadequate maintenance underscores the critical need for effective pavement monitoring. This research investigates and enhances existing monitoring approaches, with a focus on laboratory testing methodologies used by Italian road authorities.

It identifies limitations in the current practices—specifically, the inability of existing methodologies to distinguish between conventional bituminous and Alternative Paving Materials (APMs). To address these gaps, this study introduces both basic and advanced testing techniques aimed at improving pavement monitoring, enhancing performance prediction, and promoting more sustainable road infrastructure.

A pragmatic approach, incorporating both basic and advanced damage-related characterization methods, was employed to evaluate and enhance pavement monitoring. Basic-level tests include the Indirect Tensile Asphalt Cracking Test (Ideal-CT) for cracking resistance, the Indirect Tensile Strength Ratio (ITSR) for moisture sensitivity, and High Temperature Indirect Tensile Testing (HT-IDT) for rutting evaluation. Advanced-level methods based on Simplified Visco-Elastic Continuum Damage Model (SVECD) involve the Asphalt Mixture Performance Tester (AMPT), used to assess dynamic modulus, cyclic fatigue, and stress sweep rutting performance.

In addition, damage models such as the model for stiffness evaluation Model for Stiffness Evaluation (2s2p1D), SVECD, Model to Evaluate the Viscoplastic Behavior of the Asphalt Mixtures (shift), and the Coefficient of Thermal Contraction (CTC) model were applied to evaluate material behavior under realistic service conditions.

The findings demonstrate significant improvements in the ability to monitor and predict the performance of alternative paving materials, including rubberized and reclaimed asphalt mixtures. A comprehensive database of the tested materials was also developed to serve as a reference for quality assurance, future pavement management systems, and optimized maintenance strategies.

Ultimately, this study contributes to the advancement of reliable and sustainable road infrastructure.

Keywords:

Pavement monitoring, alternative paving materials (APMs), damage-related characterization, basic laboratory testing, advanced level approach

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List of Abbreviations

AMPT	Asphalt Mixture Performance Tester
ANAS	Italian Road Specifications
APMs	Alternative Paving Materials
AASHTO	American Association of State highway and Transportation officials
CTC	Coefficient of Thermal Contraction
CTI	Indirect Tensile Coefficient
COS	Strain Control Mode
CS	Stress Control Mode
CX	Actuator Displacement Control Mode
DMR	Dynamic Modulus Ratio
DQI's	Data Quality Indicators
DPSE	Dissipated Pseudo strain Energy
ECR	Engineered Crumb Rubber
FHWA	Federal Highway Administration
FlexMATTM cracking	damage modeling tool for fatigue cracking
FlexMATTM rutting	damage modeling tool for rutting
FlexPAVETM	Pavement Performance and Analysis Simulation Based Program
FWD	Falling Weight Deflectometer
HMA	Hot Mix Asphalt
HT-IDT	High Temperature Indirect Tensile Testing
IDT	Indirect Tensile Strength Testing
Ideal-CT	Indirect Tensile Asphalt Cracking Test
IRI	International Roughness Index

ISV	Internal State Variable
ITS	Indirect Tensile Strength
ITSd	Average Indirect Tensile Strength of Dry Specimens
ITSR	Indirect Tensile Strength Ratio
ITSw	Average Indirect Tensile Strength of Wet Specimens
M&R	Maintenance and Rehabilitation
MEPDG	Mechanistic Empirical Pavement Design Guide
Nf	Number of Cycles Till Failure
PG	Performance Grade
PMS	Pavement Management System
PQI	Pavement Quality Index
PRS	Performance Related Specifications
PSI	Present Serviceability Index
PSR	Present Serviceability Rating
QA/QC	Quality Assurance and Quality Control
RAP	Reclaimed Asphalt Pavement
RO-I	Research Objective-I
RO-II	Research Objective-II
RO-III	Research Objective-III
RO-IV	Research Objective-IV
RSI	Rutting Strain Index
SVECD	Simplified Visco-Elastic Continuum Damage Model
Sapp	Apparent Surface Damage Capacity Index
shift	Model to Evaluate the Viscoplastic Behavior of the Asphalt Mixtures
SSR	Stress Sweep Rutting Test
TRLPD	Triaxial Repeated Load Permanent Deformation
TRS	Thermorheologically Simple

USURA B	Italian Gradation Specifications for Wearing Surfaces
VECD	Viscoelastic Continuum Damage
VEPCD	Visco Elastoplastic Continuum Damage
WR_{max}	Highest Amount of Stored Pseudostrain Energy
WRC	Total Dissipated Pseudostrain Energy
t-SS	Time to Stress Superposition Principle
t-TS	Time to Temperature Superposition Principle
2s2p1D	Model for Stiffness Evaluation

Chapter 1

Introduction

1.1 Problem statement

Road infrastructure plays an important role in the growth of the economy and society across the globe. However, over time, the combined effects of traffic loading, environmental conditions, and inadequate maintenance contribute to the deterioration of road pavements. As a result, road pavements increasingly experience premature failures. Therefore, to ensure the sustainable and long-term performance of road infrastructure, it is essential to focus on effective and appropriate Maintenance and Rehabilitation (M&R) processes. The scheduled M&R process requires efficient monitoring of road infrastructure. A key aspect of pavement monitoring involves the laboratory testing of pavement materials before, during, and after the construction of road pavements. Especially for the monitoring of road pavements on small and rural highways, the current approach used by the Italian Road Specifications (ANAS) — which assumes a design life of 10 years, as shown in figure 1.1 — includes pavement monitoring before, during, and after construction [2].

Before the construction of road pavements, laboratory testing is conducted using virgin materials and asphalt mixtures. This stage serves as a pre-qualification phase for the mixtures. Testing of bitumen, modified bitumen, aggregates, and loose mixtures is performed to evaluate both mechanical and volumetric properties. Mechanical analysis is carried out using conventional bituminous IDT testing, while volumetric analysis focuses on measuring the void content of the mixtures.

During the construction phase, preliminary quality assurance and quality control (QA/QC) checks are performed on collected loose mixtures. These include volumetric evaluations such as bitumen content, gradation analysis, and void percentage, along with mechanical testing, including indirect tensile strength testing (IDT). After construction, road pavements are monitored through field cores, where both volumetric and mechanical properties are assessed. These aspects are examined in this investigation.

Although the monitoring approach of the ANAS applies a sophisticated laboratory testing approach, but the main investigation carried out by this study whether it is capable of distinguishing between conventional bituminous and Alternative Paving Materials APMs? Secondly, the ability to investigate the cracking, rutting and moisture damage, and monitor during the entire service life of road pavements with APMs.

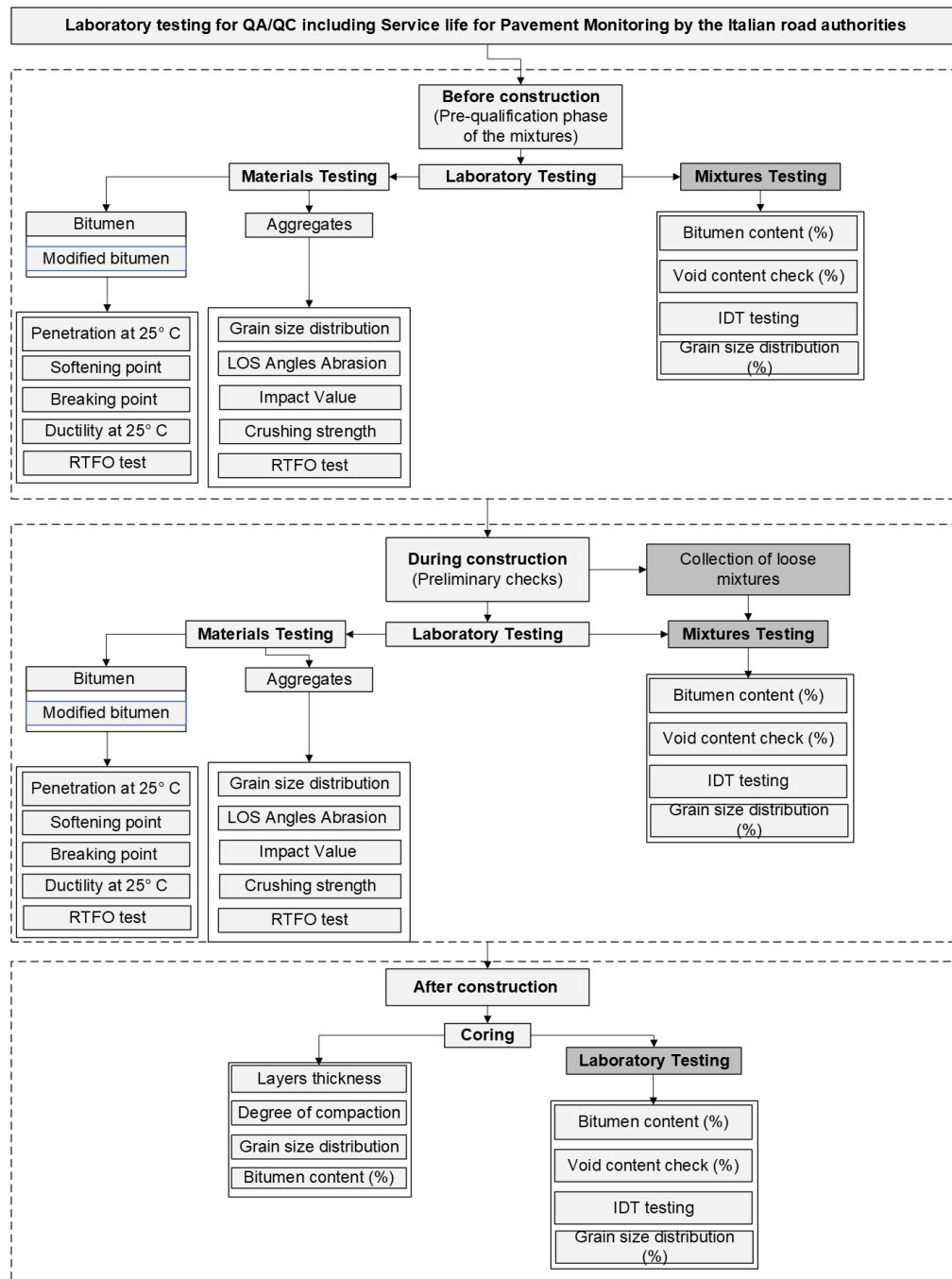


Figure 1.1: Monitoring approach by the Italian road authorities ANAS capitolato,2016. Source: author's explanation

1.2 Research objectives

The main aim of this manuscript is improve the performance related characterization of bituminous materials for QA/QC, pavement monitoring in Italy and to discriminate the conventional bituminous materials and APMs. In order to that this research study has the following research objectives shown in figure 1.2.

Research Objective-I (RO-I):

To assess whether the current Italian road authority characterization by ANAS (including both QA/QC service

life), can discriminate between the performance of conventional bituminous and [APMs](#). This approach is based on the evaluation of the volumetric properties and [IDT](#) testing as follows:

- Indirect Tensile Strength ([ITS](#)) at 25.0 °C for any bituminous materials.
- [ITSR](#) at 25.0 °C for crumb rubber bituminous materials.

Research Objective-II ([RO-II](#)):

To investigate whether a 'basic' damage-related characterization of pavement materials can discriminate between performances of conventional bituminous materials and [APMs](#). This approach is still based on volumetric characterization and [IDT](#) testing for any bituminous materials including [APMs](#),

- Rutting resistance using [HT-IDT](#) at 55.0 °C testing for the Sicily region.
- Cracking resistance using [Ideal-CT](#) testing at 25.0 °C.
- Moisture sensitivity using [ITSR](#) testing at 25.0 °C.

Research Objective-III ([RO-III](#)):

To investigate whether an "advanced" [SVECD](#) damage-related characterization of paving materials can discriminate between performances of conventional bituminous materials and [APMs](#). This approach is based on volumetric characterization and testing with an [AMPT](#) coupled with damage modeling based on [SVECD](#) for any bituminous materials incorporating [APMs](#) as follows:

- Stiffness using [AMPT](#) for dynamic modulus testing and [2s2p1D](#) model
- Rutting resistance using [AMPT](#) for Stress Sweep Rutting Test ([SSR](#)) testing and [shift](#) model.
- Crack resistance using the [AMPT](#) and [SVECD](#) model.
- Thermal analysis using [CTC](#) model

Research Objective-IV ([RO-IV](#)):

To create a data base of tested conventional bituminous materials also incorporating [APMs](#) namely, rubberized and reclaimed asphalt mixtures for wearing course, designed according to [ANAS](#) Italian road specifications [2]. This database could be use as a reference for [QA/QC](#), pavement monitoring and future development of Pavement Management System ([PMS](#)).

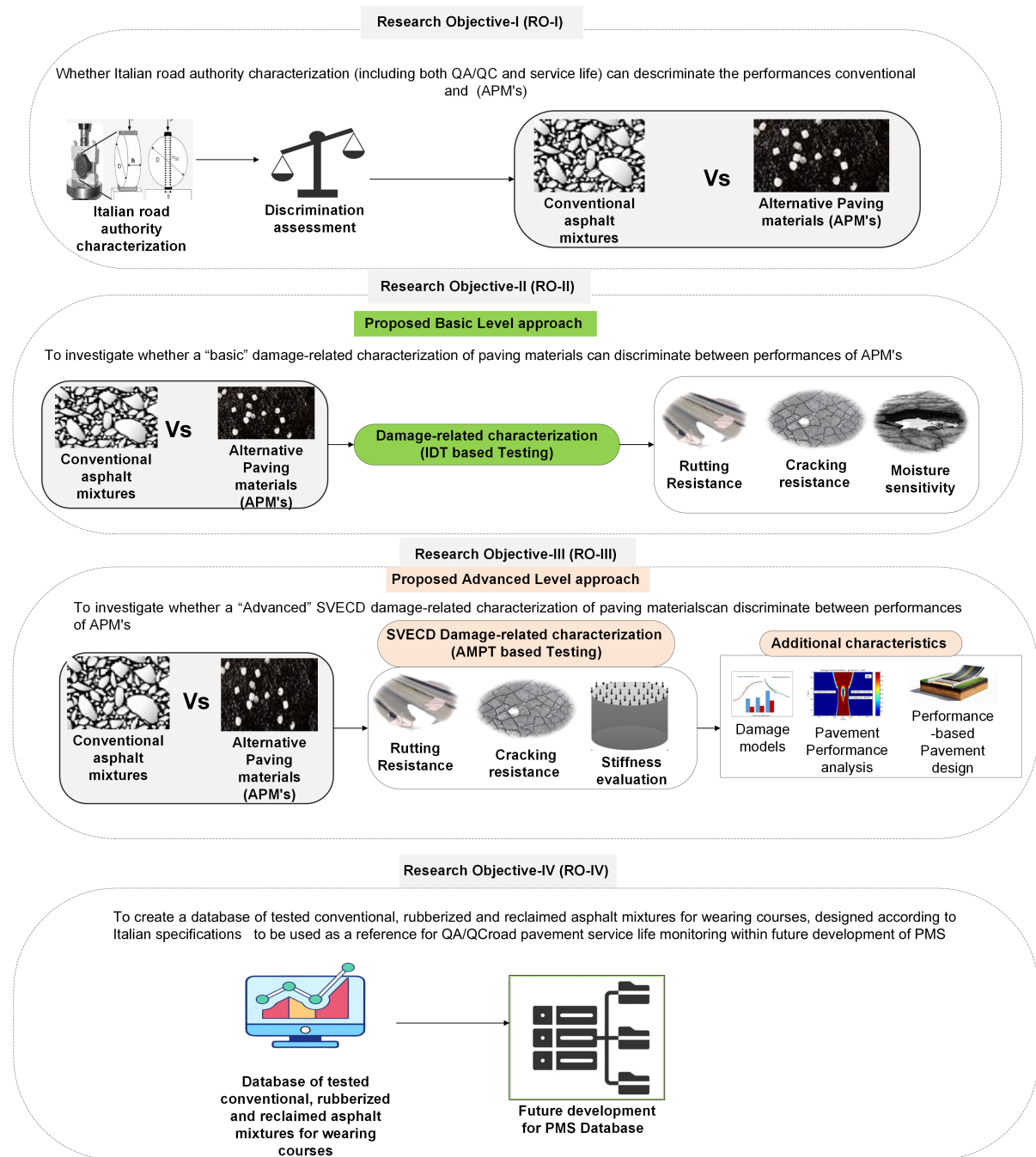


Figure 1.2: Research objectives in general overview of the study, Source: author's explanation

1.3 Aim of the research

The aim of this research is illustrated within a scientific framework in figure 1.3. To address the research question stated in the problem statement—specifically, the need to distinguish between conventional bituminous materials and APMs—this study seeks to improve the monitoring approach currently employed by the ANAS. The focus is on laboratory testing conducted before, during, and after the construction of road pavements.

The proposed laboratory methods are based on damage-related characterization techniques at both basic and

advanced levels, as shown in figure 1.3. The basic-level approach involves IDT testing, while the advanced-level approach incorporates AMPT testing, grounded in the SVECD theory. These methodologies are intended to enhance pavement monitoring and contribute to a more effective M&R scheduling process, serving as a foundation for the future development of PMS.

It is important to note that this study is limited to the improvement of laboratory-based pavement monitoring. Periodic monitoring and non-destructive testing methods currently used by the Italian road authorities are beyond the scope of this research.

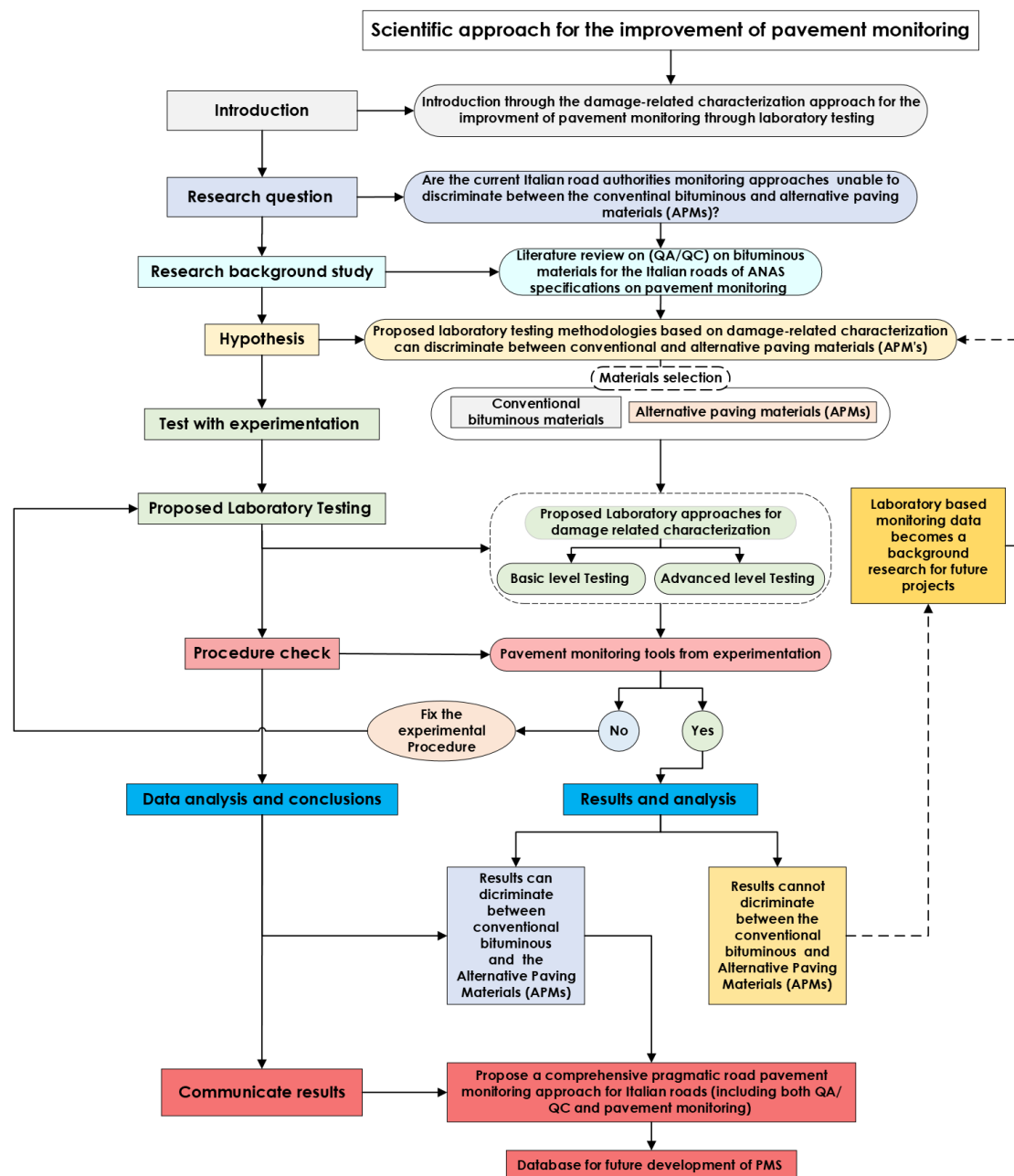


Figure 1.3: Aim of the research study in context of scientific approach Source: author's explanation

1.4 Thesis organization

To consider the problem statement and research objectives, this research study is divided into the following chapters. The summary for the organization of this thesis structure can be seen figure 1.4 .

Chapter 1: This chapter presented an introduction to the problem statement concerning current practices adopted by [ANAS](#) for pavement monitoring and [QA/QC](#), including the evaluation of service life before, during, and after the construction of road pavements. A key issue identified is the inability to effectively distinguish between conventional bituminous materials and [APMs](#), which constitutes the core of the problem statement.

To address this issue, the research aims to enhance pavement monitoring and [QA/QC](#) procedures, including service life assessment, through the implementation of advanced laboratory testing methods—ultimately supporting the development of future [PMS](#).

In alignment with this aim, the objectives of the study focus on improving the pavement monitoring process and enabling the differentiation between conventional and alternative paving materials. Finally, a complete summary of the research approach is outlined in the thesis organization section.

Chapter 2: The literature review outlines the existing body of research, beginning with the historical development of flexible pavements and progressing toward pavement monitoring techniques and the future development of [PMS](#). This chapter critically reviews both conventional bituminous materials and [APMs](#) used in the experimental program. It also discusses testing methods based on both basic that includes [IDT](#) testing and advanced approaches based on [SVECD](#) theory, along with damage modeling techniques and performance-based pavement design.

Following the review, a hypothesis is developed regarding the limitations of the current monitoring approaches used by the Italian road authorities for laboratory testing before, during, and after the construction of road pavements. As illustrated in 1.4, the hypothesis states that existing laboratory testing methods are inadequate for distinguishing between conventional bituminous materials and [APMs](#). This limitation defines the core motivation behind the present research.

Accordingly, this study aims to improve current laboratory testing methodologies. The proposed approach combining basic and advanced testing techniques—serves as the foundation for addressing the central research question: How can conventional bituminous materials be effectively distinguished from [APMs](#)?

Chapter 3: This chapter explains the research methodology in the context of scientific approach adopted to support the hypothesis developed in the literature review—namely, that current testing methods adopted by the [ANAS](#) for pavement monitoring and [QA/QC](#) including both service life are insufficient to distinguish between conventional bituminous materials and [APMs](#). The study involves the characterization of conventional bituminous mixtures, along with six types of [APMs](#), including rubberized, hybrid, reclaimed, rubberized-hybrid,

reclaimed-rubberized, and reclaimed-hybrid asphalt mixtures. The conventional bituminous materials and APMs were characterized according to ANAS.

Subsequently, the proposed methodology, comprising both basic and advanced-level testing approaches, was applied to both conventional bituminous materials and APMs. The basic approach includes tests volumetric analysis and IDT testing for cracking, rutting and moisture resistance, while the advanced approach based on SVECD theory incorporates performance-related characterization, damage-based models and performance based pavement design.

As summarized in figure 1.4, this chapter addresses the key research components:

What is being improved?: the current laboratory testing procedures used by ANAS ,

How it is being improved?: Through the application of damage-related characterization that includes basic-level approach based on IDT testing and more advanced approach based on AMPT testing and SVECD theory,

Why it is being improved?: because the conventional characterization methods by ANAS including QA/QC including service life failed to differentiate between conventional bituminous materials and APMs.

Chapter 4. The Results and Discussions chapter presents the outcomes obtained from both the conventional bituminous characterization—based on the current practices of the ANAS Italian road authorities—and the proposed methodology discussed in chapter 3 . The proposed approach includes both basic and advanced testing procedures. The basic-level tests assess cracking resistance, rutting potential, and moisture sensitivity, while the advanced-level tests evaluate stiffness, fatigue resistance, and rutting resistance through performance-based and damage modeling techniques.

These evaluations were conducted for both conventional bituminous mixtures and APMs. The results are used to support damage modeling, pavement performance analysis, and performance-based pavement design. Collectively, these findings contribute to improving pavement monitoring practices and enhancing the ability to discriminate between conventional bituminous materials and APMs.

Chapter 5: Conclusions and recommendations, Finally the conclusions have been drawn based on the outcomes obtained from chapter 4. To proceed with the future recommendations, the concept of LIVELABS for the improvement of pavement monitoring based on five scenarios have been introduced.

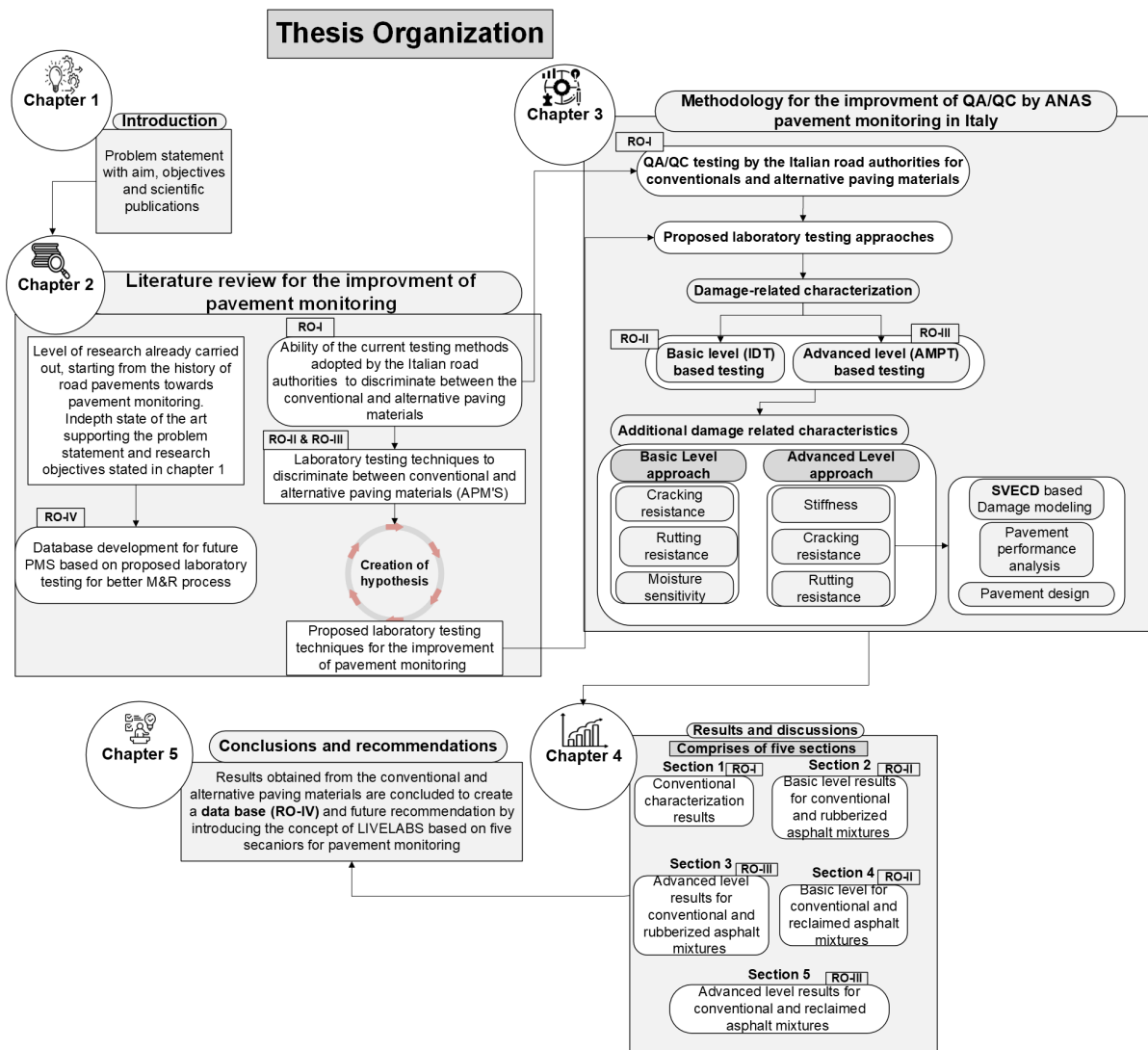


Figure 1.4: Thesis organization, Source: author's explanation

1.5 List of Scientific publications

1.5.1 Related International Journal publications

1. Ghani, U., Milazzo, S., Giancontieri, G., Buttitta, G., Gu, F., & Presti, D. L. (2024). Investigating the Ability of Road Specifications to Discriminate the Rutting Behavior of Rubberized Asphalt Mixtures in Italy. *Infrastructures*, 9(7), 113.
2. Giancontieri, G., Buttitta, G., Ghani, U., & Lo Presti, D. (2025). Advanced Mixing Rheometry for Dynamic Shear Rheometer Testing of Highly Heterogeneous Bituminous Binders. *Journal of Testing and Evaluation*, 53(2).

1.5.2 Related International Conference proceedings

1. Giancontieri, G., Buttitta, G., Ghani, U., & Lo Presti, D. (2024, July). Feasibility Study on High Content Reclaimed Rubberised Asphalt Mixtures for Wearing Courses. In International Conference on Maintenance and Rehabilitation of Pavements (pp. 416-424). Cham: Springer Nature Switzerland.
2. Ghani, U., Milazzo, S., Giancontieri, G., Mignini, C., Buttitta, G., Gu, F., & Lo Presti, D. (2023). Unveiling the Benefits of Engineered Crumb Rubber for Asphalt Mixtures via Performance-Related Characterization: Rutting Behavior. *Engineering Proceedings*, 36(1), 39.
3. Giancontieri, G., Buttitta, G., Ghani, U., & Lo Presti, D. (2024, June). Advanced Mixing Rheometry for Dynamic Shear Rheometer Testing of Highly Heterogeneous Bituminous Binders. In International Symposium on Asphalt Pavement & Environment (pp. 323-329). Cham: Springer Nature Switzerland.

1.5.3 Other International Journal publications

1. Ghani, U., Zamin, B., Tariq Bashir, M., Ahmad, M., Sabri, M. M. S., & Keawsawasvong, S. (2022). Comprehensive study on the performance of waste HDPE and LDPE modified asphalt binders for construction of asphalt pavements application. *Polymers*, 14(17), 3673.
2. Imran, W., Khan, Z. H., Khan, D., Ghani, U., & Bashir, T. (2024). On the practical utility of a continuum traffic flow model on curvy highways in adverse weather conditions. *Transportation research interdisciplinary perspectives*, 25, 101108.

1.5.4 Other International Conference proceedings

1. Buttitta, G., Giancontieri, G., Milazzo, S., Mignini, C., Hennig Osmari, P., Ghani, U., & Lo Presti, D. (2022). Investigating Tools for Sustainability Assessment of Road Pavements in Europe. *Engineering Proceedings*, 17(1), 34.
2. Buttitta, G., Milazzo, S., Mignini, C., Hennig Osmari, P., Ghani, U., Giancontieri, G., & Lo Presti, D. (2022). Life Cycle Management of road pavements. In Sustainability in the context of the PNRR: the contribution of Life Cycle Assessment . Italian LCA Network Association.
3. Buttitta, G., Giancontieri, G., Ghani, U., Milazzo, S., & LO PRESTI, D. (2023). Un'applicazione del Sustainability Assessment Framework per le pavimentazioni stradali in Europa. In ATTI XVII Convegno dell'Associazione Rete Italiana LCA 30 anni di Life Cycle Assessment-sviluppi metodologici e applicativi (pp. 313-321). IT.

Chapter 2

Literature Review for the improvement of road pavement monitoring

2.1 Introduction

This chapter presents a review of the research previously conducted on road pavements, with a particular focus on the monitoring of road infrastructure. Although various scientific approaches have been developed to enhance pavement monitoring, this study specifically aims to critically analyze the existing literature in the context of laboratory testing. Mostly laboratory testing techniques are utilized for the materials improvement and its characterization, but these testing techniques are rarely involved in road pavement monitoring. The last part of this chapter review pavement monitoring techniques outlined in [ANAS](#) Italian road specifications, which are applied before, during, and after the construction of road pavements.

2.1.1 History of road pavements

Although the history of road pavements spans thousands of years, Tillson notes that the first organized road system was developed by the Carthaginians around 500 BC [\[29\]](#). The Romans, known for their expertise in cement technology and street paving, expanded significantly on this foundation. Following political consolidation, the Roman Empire constructed approximately 87,000 kilometers of roads—representing the most extensive road network of the ancient world [\[30, 31\]](#).

It can be concluded that while the origins of roads date back to early civilizations, the Romans were primarily responsible for developing the fundamental structure of modern roadway systems. From ancient roads to today's smart transportation infrastructure, the evolution of road systems reflects both technological advancement and societal needs. A brief summary of this historical progression is provided below.

2.1.1.1 Ancient Roads (Circa 4000 BCE - 500 CE)

- Mesopotamia and Egypt (4000-2000 BCE): Some of the earliest road pavements were found in ancient Mesopotamia and Egypt. These roads were primarily constructed using crushed stone and packed earth. In Mesopotamia, evidence of bitumen (a natural asphalt) was found in road construction [\[32\]](#).
- Indus Valley Civilization (2500-1900 BCE): Streets in cities like Mohenjo-Daro were paved with burnt

bricks, showcasing early forms of urban planning and road paving [33].

- **Roman Roads (300 BCE - 500 CE):** Roman roads are among the most famous ancient pavements. The Romans built an extensive network of roads across their empire, using layered construction methods. They laid a foundation of large stones, followed by smaller stones, gravel, and sand. The top layer was often made of stone slabs, providing a durable and relatively smooth surface. Roads like the Appian Way used this method, enabling efficient military and trade movement [34].

2.1.1.2 Medieval Roads (500-1500 CE)

During the medieval period, road construction and maintenance declined in Europe following the fall of the Roman Empire. Roads often reverted to dirt paths, becoming less structured and maintained [35]. However, in other parts of the world, such as China and the Islamic world, the development and maintenance of roads continued. In China, the Silk Road facilitated trade and included various types of surfaces like stone and compacted earth [36].

2.1.1.3 Early Modern Roads (1500-1800 CE)

- **Telford and McAdam (18th-19th Century):** Road construction saw significant advancements in the 18th century with engineers like Thomas Telford and John Loudon McAdam. Telford developed roads with large stones at the base and smaller stones on top, ensuring drainage and stability. McAdam further refined this method by using small, single-sized crushed stones placed on a compacted earth base. This "macadamized" road required a convex surface for water drainage [37].
- **Turnpike Trusts:** In the 18th and 19th centuries, turnpike trusts in the UK managed road construction and maintenance. These trusts funded road building through tolls, leading to better-maintained roads for trade and transportation [38].

2.1.1.4 The Advent of Asphalt and Concrete (19th - Early 20th Century)

- **Asphalt Roads:** The first modern asphalt road was constructed in Paris in 1824. By the late 19th century, asphalt and bitumen became more widely used, particularly for urban streets. Asphalt was favored for its smooth, durable surface, making it ideal for horse-drawn carriages and later, automobiles [39].
- **Concrete Roads:** Concrete roads also began to emerge in the early 20th century. The first concrete street was laid in Bellefontaine, Ohio, in 1891. Concrete provided a more rigid surface compared to asphalt and became popular for highways and urban streets [40].

2.1.1.5 Modern Pavements (20th Century - Present)

- Flexible and Rigid Pavements: Modern road construction primarily uses two types of road pavements: flexible (asphalt) and rigid (concrete). Flexible pavements consist of multiple layers, including sub grade, base, and asphalt surface, designed to flex under load. Rigid pavements are constructed using concrete slabs that distribute loads over a larger area [41]
- Interstate Highway System (1950s-1960s): The development of the Interstate Highway System in the United States during the 1950s and 1960s marked a significant advancement in road pavement technology. This system used a combination of asphalt and concrete pavements to create a nationwide network of highways [42].

2.1.1.6 Sustainable and Smart Pavements (21st Century)

- Sustainable Pavements: Modern concerns about sustainability have led to innovations like permeable pavements, which reduce runoff and improve groundwater recharge. Recycled materials, such as rubber and plastic, are increasingly used in pavement construction [43].
- Smart Roads: With advancements in technology, "smart" pavements are being developed, incorporating sensors and solar panels. These pavements can provide data on traffic patterns, road conditions, and even generate electricity [44].

The history of road pavements reflects the ongoing evolution of materials and engineering techniques, adapting to the changing demands of transportation and society. The summarized history of the road pavements has been explained in figure 2.1

2.1.2 Types of road pavements

Based on composition of materials there are three main types of pavements i-e flexible, rigid and composite pavements. A flexible pavement is composed of granular and unbound granular materials, while the rigid pavement is composed of cement concrete with or without using the reinforcements. When the asphalt is overlay on hydraulically bound base it becomes a composite structure. This research study has focused on flexible pavements and especially the top wearing course made up of asphalt concrete along with the use of APMs. Here it is to note that Italian pavement structures as shown in figure 2.2 are typically layer wise different as compared to the pavement structures in the United States [17, 45]. Italian road pavement structures have three asphalt concrete surfaces. The thickness limit according to the ANAS for wearing surface is 4-6 cm, while the binder course is 8-12 cm, and the base course is 12-15 cm. The granular material is laid up to 25 cm [46]. And typically, in these layers along with the asphalt concrete the alternative paving materials to strengthen these layers are added during the mixing process and different laboratory tests are conducted for QA/QC by the Italian road authorities [2]. The

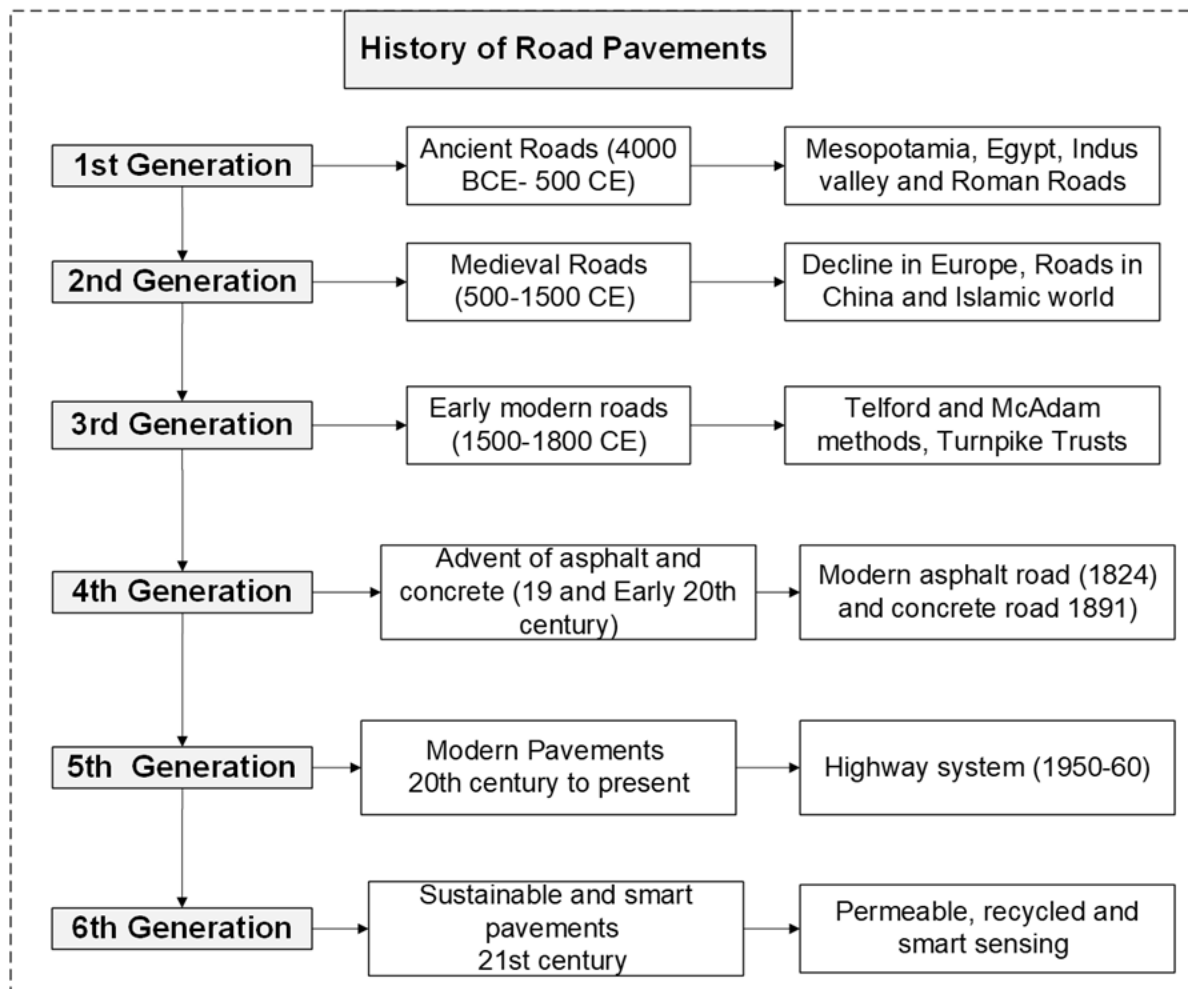


Figure 2.1: History of road pavements, Source: author's explanation

further details about the pavement materials and its laboratory testing have been discussed in the section materials characterization and technology development.

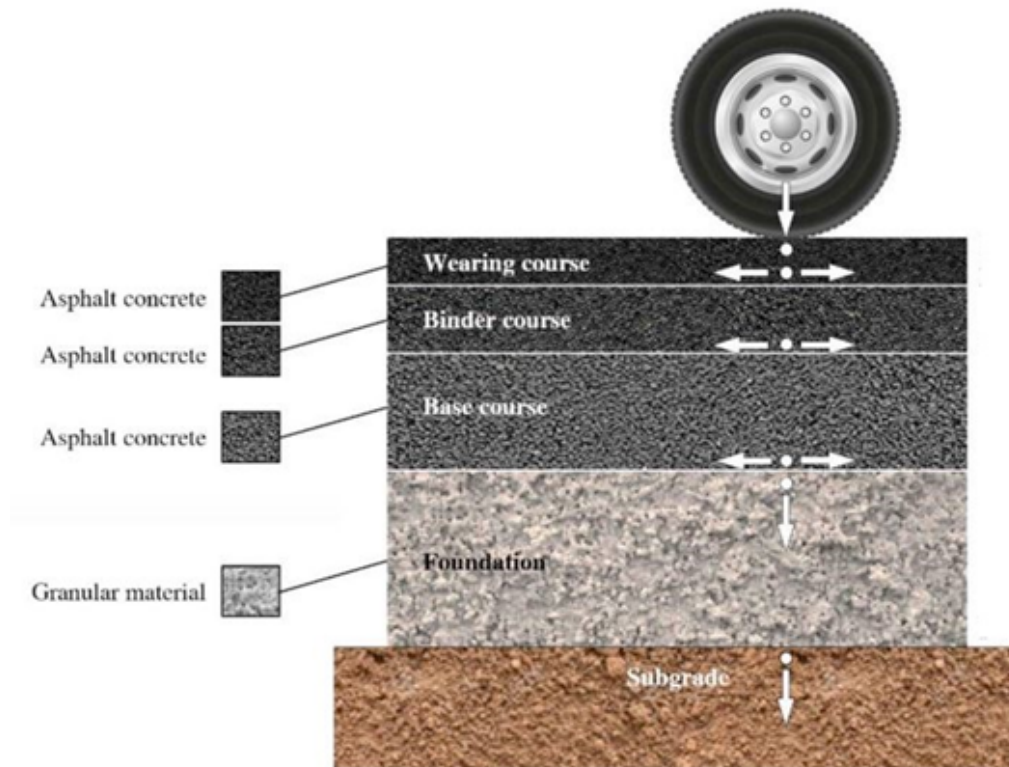


Figure 2.2: Asphalt pavement structure in Italy,[17]

2.2 Materials characterization and technology development

With the advent of science and technology there has always been a development in both sectors of road pavement materials and its related laboratory testing. Laboratory testing of road pavement materials plays a key role, from the phase of design, construction, use and even in [M&R](#) of the road pavement structures. Performance of the road pavement materials is one of the main important factors that affect the road pavement performance and its service life. After the construction of road pavements, the materials are exposed to the heavy traffic and adverse climatic conditions, therefore the monitoring of the road pavement materials is of the same importance as that of road pavement monitoring. But before accenting the comprehensive details regarding the pavement materials testing and its technology development, it is incumbent to explicate the 'performance related' and 'performance based' laboratory tests. Empirical or performance related tests involves the material characteristics correlating the engineering properties that predict the performance of road pavements. The performance-based tests directly involve the prediction and primary relationships of the pavement performance [\[5\]](#). The parameters of fatigue life are based on mechanistic approach, that estimates its average values resulting the rise in maintenance cost of the road pavements [\[47\]](#). These parameters involve an extensive laboratory testing. On the other hand, the conventional

testing is not fully focusing on thermo-rheological properties of Hot Mix Asphalt (HMA) over a wide range of the temperatures. There is also a lack of real traffic and climatic induced stresses, upon which the road pavements are exposed for their service life [48]. To overcome this issue, in the past few years, the researchers tried to focus on performance-based tests and some of the key characteristics implemented in European standards as shown in Table 2.1. For the development of numerical models, these standards provide a key input parameters that can predict the in-service performance of the pavement structures. However, there is a lack of method available to continuously monitor the stresses under realistic traffic and climatic conditions.

Table 2.1: Performance-Based European Test Standards

Asphalt Layer	Stiffness of Material	Fatigue of Material	Performance at Low Temperature	Rutting
Surface Course	X	(X)	X	X
	X	(X)	X	X
Binder Course	X	X	(X)	(X)
Base Course				
Testing Procedures				
	2-Point Bending Test (2PB-TR, 2PB-PR)	Cyclic Indirect Tensile Test (CIDT)	Temperature Stress Restrained Specimen Test (TSRST)	Triaxial Cyclic Compression Test (TCCT)
	3-Point Bending Test (3PB)	4-Point Bending Test (4PB)	Uniaxial Tension Stress Test (UTST)	Uniaxial Cyclic Compression Test (UCCT)
	4-Point Bending Test (4PB)		Uniaxial Cyclic Tension Stress Test (UCTST)	
	Cyclic Indirect Tensile Test (CIDT) Direct Tension-Compression Test (DCT)			
EN Standards	EN 12697-26	EN 12697-24	EN 12697-46	EN 12697-25

* x mandatory performance characteristics
*(X) Additional performance characteristics

The stress or strain levels in the laboratory are used to evaluate the fatigue life of the pavement structures. The strain values measured in the laboratory are limited to the 70 microns [49]. Testing specifically below the 70 microns is the elastic range for asphalt concrete. To have a reliable pavement design and its performance, the realistic prediction can only be possible under the cyclic loading. The shift factor is used to achieve the field-based performance by the laboratory testing. The shift factor is capable of accommodating certain parameters like aging, moisture damage, temperature fluctuations, healing and traffic intensity [18]. Field based fatigue estimate is 15 to 20 times as that laboratory estimated value. Laboratory based fatigue is mainly based on mix variables like (aggregate properties, stiffness and binder content, mix density and gradation of aggregates) and loading conditions [1]. The behavior of the asphalt concrete under different number of loading cycle and stress/strain level along with mode of loading is depicted in figure 2.3.

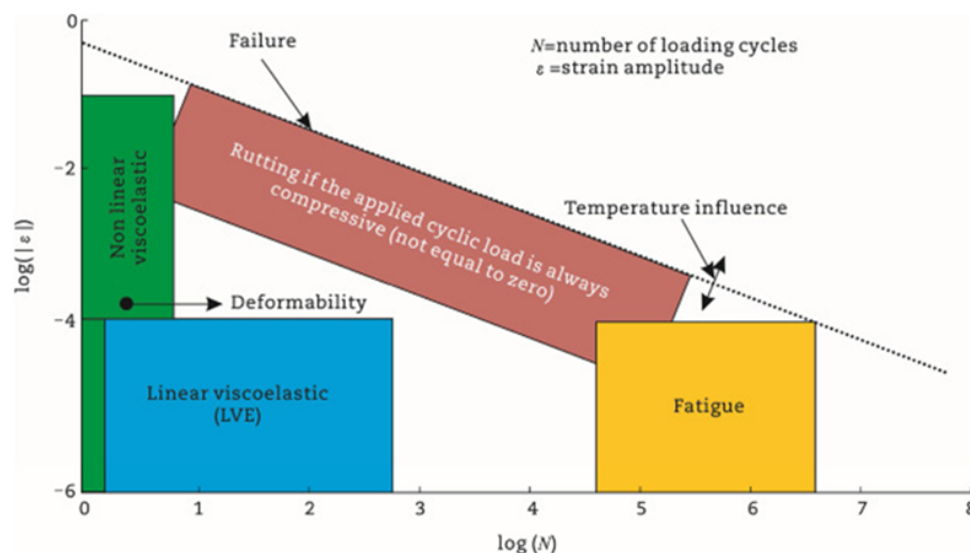


Figure 2.3: Viscoelastic behavior of the asphalt materials [18].

Considering the stresses induced by heavy loading and environmental effects and the extensive laboratory testing, [AMPT](#) is an advanced asphalt mixture performance tester, capable of evaluating the mixture properties and performance of mixture and pavements. The monitoring of the mix design and the quality assurance can be done through [AMPT](#). It improves mix designs linked with the structural design, the pavement service life, creates master curves, analyze the pavement failure and reduce the maintenance cost in a single equipment. The complete setup includes [AMPT](#) for mixture testing, FlexMAT for the asphalt mixture analysis, FlexPAVE for the pavement performance and analysis program, and FlexMIX for performance engineered mix design [12]. However, the accuracy of the transfer functions of FlexPAVE requires real life monitored data and the original paving materials. Some of the equipment for the assessment of the mixture performance with properties that need to be monitored in the field are listed in Table 2.2.

2.2.1 Road pavement materials

Road pavement materials are one of the important characteristics acting as building blocks of the road pavement structures. To have a long-life cycle of the road pavements, the materials in general and especially the bituminous materials are getting more importance for the damage quantification including economical cost in construction and [M&R](#) process. The road pavement materials include soils, granular materials, hydraulically bound materials, bituminous-bound materials, and other distinct types of materials. Every type of a road pavement structure except bridges are laid on several types of soil like silts, sands, heavy clays and high strength rock materials. Usually, the soils are unpredictable in nature and water sensitive, while some of the clays are impermeable [50, 51, 52]. Here it is to be noted that this research work has aimed to study the bituminous materials of top wearing surface of the road pavements that comes directly in contact with the moving load and environmental conditions. The factors that affect the performance and design of road pavements are the stiffness under the moving load and resistance to the permanent deformation in relation with the shear strength. The damage to the sub grade can be caused by high compressibility, insertion of water content and poor bearing capacity of soil [53]. On the other hand, under the repeated deformation the use of expansive soils can cause cracking, frost heaving and boiling due to the

Table 2.2: Performance-based Testing for Routine Mix Designs [1]

Property	Testing Method	Method Precision		Mix Design Pass/Fail	Performance Prediction Model	Method	Analysis	Availability	Reliability	Cost
Modulus	AMPT Dynamic Modulus	Yes	Yes	Yes	Yes	Moderate	Moderate	Yes	High	Moderate
	Simple Shear	Yes	Yes	NA	Yes	Moderate	Moderate	Yes	Moderate	High
	Indirect Tension Dynamic Modulus	No	No	Yes	Yes	Moderate	Moderate	No	Moderate	High
Permanent Deformation	AMPT Flow Number	Yes	Yes	Yes	Yes	Moderate	Low	Yes	High	Moderate
	Repeated Shear	Yes	Yes	Yes	Yes	Moderate	Moderate	Yes	Moderate	High
	High Temperature Indirect Tensile Strength	Yes	No	Yes	No	Low	Low	Yes	High	Low
	Asphalt Pavement Analyzer	Yes	No	Yes	No	Moderate	Low	Yes	High	Moderate
	Hamburg Wheel Tracking Device	Yes	No	Yes	No	Moderate	Low	Yes	High	Moderate
Load Associated Cracking	Flexural Fatigue	Yes	No	No	Yes	High	Moderate	Yes	High	Moderate
	AMPT Continuum Damage	No	No	No	Yes	High	High	Yes	High	Moderate
	Energy Ratio	No	No	Yes	Yes	High	High	Yes	High	High
	Fracture Energy	No	No	No	Yes	Low	Low	Yes	High	High
Thermal Cracking	IDT Creep and Strength	Yes	No	No	Yes	High	High	Yes	High	High
	Disc Shaped Compact Tension	Yes	Yes	Yes	Yes	High	Low	Yes	High	High
	Semi Circular Bend	Yes	No	Yes	No	Moderate	Low	Yes	High	High
Moisture Sensitivity	Tensile Strength Ratio	Yes	Yes	Yes	No	Moderate	Low	Yes	High	Low
	Hamburg Wheel Tracking Device	Yes	No	Yes	No	Moderate	Low	Yes	High	Moderate

hydrophilic characteristics. In case of bituminous bound materials, the top wearing surface is mainly made up to [HMA](#). It is composed of bitumen, aggregates and fine particles. Being on top of the structure, these particles bear the heavy traffic, and uncertain environmental conditions can have a substantial influence on the performance of road structure.

The fundamental key performance parameters include resistance to fatigue cracking, rutting, thermal cracking along with complex stiffness of these materials. The performance of the fatigue life and flexure strength is significantly dependent upon the type and selection of materials [54, 55]. The fatigue life of the asphalt concrete is to resist the cracking under repeated loads. It mainly depends upon the stress degradation ratio, energy and its relative stiffness. However, asphalt binder has main impact on the resilient modulus. To have better maintenance and rehabilitation approach the pavement materials requires proper monitoring either before during or after the road construction. Therefore, this study also highlighted the current monitoring approaches by the [ANAS](#) for

QA/QC including both service life through several laboratory tests before, during and after construction [2]. The further details are provided in 2.2.2 section of this chapter.

2.2.2 Quality control and quality assurance by the Italian road authorities

According to the Italian road authorities, although it is commonly believed that interventions on the road surface are characterized by long lasting pavement design along with reduction in construction complexity, there are many aspects, sometimes overlooked, which have a decisive impact on the QA/QC of the road infrastructure [2]. Although there are series of the tests conducted by the Italian road authorities before, during and after the construction of road pavements that are described in section 2.5.1.9 of this chapter. One of the conventional testing methods that is most used for the asphalt mixtures characterization is IDT testing.

2.2.2.1 Conventional IDT testing

The conventional characterization in terms of its mechanical characteristics involves the examination of ITS parameter using IDT testing. The IDT test carried out using UNI EN 12697-23 at 25.0°C, usually three trials conducted for each examined mixture [2]. At this temperature, there is no need to modify the specimens for further conditioning. Refer to the Table 2.3, ANAS for QA/QC including both service life recommended minimum tensile strength between 0.72 to 1.60 MPa for asphalt mixtures. While ANAS CSA ACUSTICA QA/QC specification recommended a minimum range of greater than or equal to 0.6 MPa. The peak load at the specimen's fracture and its dimensions are used to compute the ITS, as indicated by equation 2.1. The schematic representation of ITS testing has been shown in figure 2.4. The formula for ITS is expressed as

$$ITS = \frac{2P}{\pi DH} \quad (2.1)$$

Here, P stands for the peak load, H denotes the specimen's height, and D indicates the diameter of the specimen. The of Indirect Tensile Coefficient (CTI) both the mixtures were calculated by the formula in equation 2.2 [2].

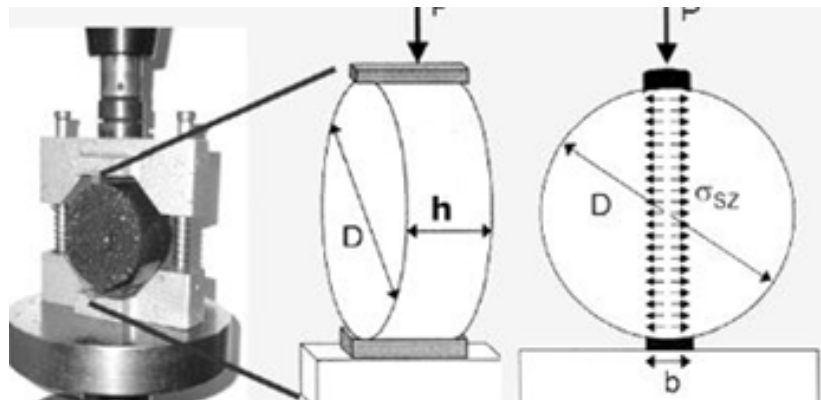


Figure 2.4: Schematic representation of IDT testing [2]

The minimum recommended range by the ANAS for the CTI, is greater than or equal to 65.0 MPa while for the ANAS CSA ACUSTICA, the minimum recommended range is between 50.0 to 150.0 MPa.

$$CTI = \frac{\pi ITS D}{2 \delta V} \quad (2.2)$$

Here in equation 2.2, δV represents the vertical displacement of the specimen recorded at the point of failure. D is the diameter of the specimen. Considering this basic IDT test which is unable to characterize the conventional bituminous materials and APMs in terms of rutting and moisture damage, therefore keeping in view the table 2.2, this research has proposed a laboratory testing methods to characterize the conventional bituminous materials and APMs based on cost and availability of apparatus. For the proper pavement monitoring there is a need to quantify the additional characterization and modeling the damage inside the asphalt mixtures in terms of stiffness, fatigue resistance, rutting and moisture sensitivity of the conventional bituminous materials and APMs. The basic and advanced approach in terms of damage-related characterization from a critical review is explained below.

Table 2.3: Thresholds for QA/QC characterization [2]

		Benchmark Values		Standards/Specifications
		No of Gyration	Void Content (%)	
QA/QC Characterization	Volumetric	N10	11-15	UNI EN 12697-5/ANAS
		N120	3-6	UNI EN 12697-5/ANAS
		N210	≥ 2	UNI EN 12697-5/ANAS
	Mechanical	IDT Testing	Range	
		ITS (MPa)	0.72–1.60	UNI EN 12697-5/ANAS
		ITS (MPa)	≥ 0.6	UNI EN 12697-5/ANAS CSA
		CTI (MPa)	≥ 65	UNI EN 12697-5/ANAS
		CTI (MPa)	50–150	UNI EN 12697-5/ANAS CSA

2.2.3 Basic-level approach

To quantify the difference between conventional bituminous materials and APMs in terms of damage-related characterization and keeping in view the availability and low cost of ITS parameter testing apparatus this research study has reviewed the some of the basic tests regarding the fatigue resistance, rutting and moisture sensitivity. The details of these tests are given in section 2.2.4, 2.2.6 and 2.2.5 of this chapter

2.2.4 Ideal-CT test for fatigue cracking

The Ideal-CT is a fracture test developed by Zhou et al used to find out the cracking resistance of the asphalt mixtures [24]. The specimen size is 150.0 mm diameter by 50.0 mm height, and the test temperature is 25.0°C. The test is easy to perform and rapid, as the loading rate is 50.0 mm/min. The CT-index parameter is related to field performance and can be calculated from the applied load and corresponding displacement [24]. Furthermore,

from the area under the force-displacement curve, fracture energy (G_f) can be determined. Through a series of simplifications, it is possible to see that the rate of increased cracking is highly related to the fracture energy, the post peak slope at the maximum load 75% (m_{75}), the displacement corresponding to 75% of the maximum load measured (l_{75}) and the diameter of the specimen. Equation 2.3 shows the equation used to calculate the CT index.

$$\text{CT index} = t (62 \times G_{aff}/l_{m-75} | x(l_{75}/D)) \quad (2.3)$$

The benchmark values adopted in this study for the **Ideal-CT** have been identified with a CT-index threshold of ≥ 80 . These criteria are derived from those initially suggested by the test developers or other agencies employing similar benchmarks [56, 57, 58].

2.2.5 Moisture sensitivity ITSR test

Moisture damage was first recognized between 1930 and 1940 as a significant issue in flexible pavements. During these years, engineers noticed that water penetration into asphalt pavements reduced its strength, leading to stripping (separation of the asphalt binder from aggregate). Researchers began to explore the mechanisms of moisture damage and sought ways to test pavement performance under such conditions [59]. The **ITSR** test became a standard procedure for evaluating moisture sensitivity, particularly in Europe and the United States. It is now widely used as part of routine asphalt mix design and quality control processes.

In Today's era, **ITSR** testing is commonly used in combination with other performance tests (e.g., rutting resistance, fatigue testing) to ensure that asphalt mixtures meet durability and performance standards under varying environmental conditions [60, 61]. Regarding the water sensitivity, it can be determined according to the European standard EN 12697-12:2018 Method A. It is possible to investigate the loss of performance suffered by the different mixes due to water effect (moisture damage) [60]. A set of six cylindrical test specimen were divided into two equally sized subsets and conditioned: half of the specimens were conditioned in dry temperature at 25.0 °C while the other half was saturated and stored in water at elevated conditioning temperature of 40.0°C for 68.0 to 72.0 hours [60]. At the end of the test, the sensitivity to water is determined by calculating the **ITSR** (Indirect Tensile Strength Ratio) as a percentage ratio of the indirect tensile strength in wet and in dry conditions. The calculation of **ITSR** (%) is shown in equation 2.4.

$$\text{ITSR} = \frac{\text{ITS}_w}{\text{ITS}_d} \times 100 \quad (2.4)$$

where the Average Indirect Tensile Strength of Wet Specimens (**ITS_w**) and Average Indirect Tensile Strength of Dry Specimens (**ITS_d**) can be calculated during the moisture test.

2.2.6 High Temperature-IDT testing for rutting evaluation

The high temperature HT-IDT test has been developed to evaluate the performance of asphalt mixtures at high temperature. This test assesses the resistance of asphalt pavements to rutting, cracking, and other forms of deformation that occur under high-temperature conditions. The history of this test is closely tied to the evolution of pavement testing methods, particularly for performance evaluation in hot climates or heavy traffic conditions. As the use of asphalt pavements increased, especially in warmer regions, researchers began to realize the importance of understanding how asphalt behaved under high temperatures. Early studies revealed that asphalt pavements were prone to rutting—permanent deformation in the wheel paths—under high temperatures, especially in areas with high traffic loads. Like IDT conducted at 25.0°C, the fundamental performance-related test can be executed under elevated temperature using a high-temperature HT-IDT [62]. The specimens should be conditioned at the testing temperature, set as 10.0°C lower than the average temperature observed over a week, considering the highest pavement temperature. The HT-IDT can be conducted by following the specifications of ALDOT-458. The mixtures having equal or greater strength than 0.44 MPa strength has an excellent rutting resistance while less than 0.20 MPa are considered as the poor rutting resistant mixtures [62].

2.2.7 Advanced-Level approach

To have more advanced approach for damage-related characterization, based on SVECD theory for conventional bituminous materials and APMs the AMPT can be utilized to test the any kind of asphalt or non asphalt mixtures [57]. The advanced laboratory testing includes AMPT testing, damage modeling tools along with pavement performance analysis and design software as shown in figure 2.5. The AMPT is an asphalt mixture testing machine that can be utilized for the performance related characterization of asphalt mixtures. The further damage analysis can be performed by using FlexMAT cracking and rutting tool developed by the North Carolina State University USA. For the pavement performance, response and design the FlexPAVE tool is used to simulate the design life of pavement structures by inputting the real traffic and climatic data. Here it is to be noted that in this section only tests for stiffness, fatigue cracking and rutting are discussed. The damage modeling along with performance and response analysis are discussed in section 2.4 of this chapter.

2.2.8 Dynamic modulus Testing for stiffness evaluation

The linear visco-elastic properties and the stiffness of the conventional bituminous materials and APMs can be investigated using an AMPT according to AASHTO TP 79-13 [57]. It can perform uniaxial tests of asphalt mix samples and is a hydraulically powered machine capable of testing bituminous conglomerate specimens by subjecting them to load cycles with a wide range of temperatures and frequencies. (Asphalt Institute, 2014) [63]. To conduct the test three different temperatures (4.0, 20.0 and 40.0 °C) with three different frequencies of (0.1, 1.0 and 10 Hz) were considered on conventional and the rubberized asphalt mixtures. Three samples of 100x150 mm diameter on each temperature were assessed and the results were utilized to generate master curves by using the

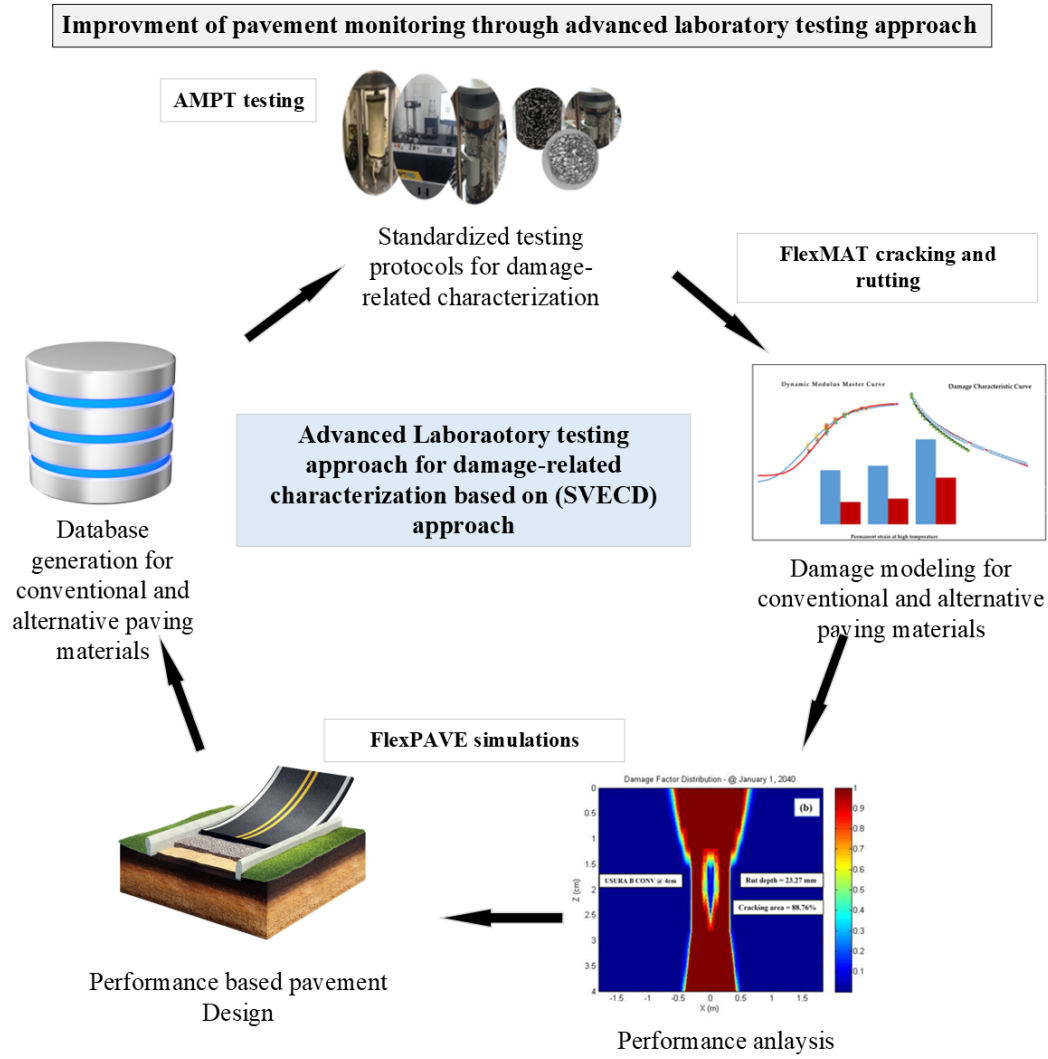


Figure 2.5: PASSFlex System using advanced laboratory testing, source: author's explanation

2s2p1D model [64], where at a given temperature, the introduced model has 7 constants, and its complex modulus is given by the following expression in equation 2.5:

$$E^*(i\omega\tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h} + (i\omega\beta\tau)^{-l}} \quad (2.5)$$

It must be emphasized in equation 2.5 that only seven constants δ , k , h , β , E_0 , E_∞ , τ_0 are needed to entirely determine the linear viscoelastic behavior of the considered material, at a given temperature.

2.2.8.1 Cyclic fatigue test for fatigue cracking

The fatigue life of an asphalt mixture plays a vital role in its performance and damage-related characterization. The propagation of cracks due to the severe loading and environmental conditions lead the researchers to have the performance-based characterization [47]. For this reason, cyclic fatigue tests can be conducted with three repetitions of conventional and APMs mixed specimens using different micro-strain levels by following the

specifications of AASHTO TP 133. The geometry of the specimen was 100x130 mm in diameter, and the strain levels applied for the conventional asphalt mixtures were (150 $\mu\epsilon$, 200 $\mu\epsilon$ and 250 $\mu\epsilon$). While for the APMs the input strain levels were (200 $\mu\epsilon$, 250 $\mu\epsilon$ and 300 $\mu\epsilon$) at a testing temperature of 20.0 °C selected based on AASHTO standard [65]. After the failure of each specimen the output data was recorded for damage modeling tool for fatigue cracking (FlexMATTM cracking) to develop SVECD model and to calculate the Apparent Surface Damage Capacity Index (Sapp) for the Sicily region [66]. For the analysis of cracking performance, the cracking percentage was evaluated and compared for both the mixtures using Pavement Performance and Analysis Simulation Based Program (FlexPAVETM) simulation program.

2.2.8.2 Stress sweep rutting test for permanent deformation

To study the rutting resistance of the conventional bituminous materials, APMs the SSR test can be conducted using an AMPT, in accordance with (AASHTO TP 79-19). Taking into the account of environmental and loading conditions for the pavement structure this test was primarily developed by the NC state university USA. This test was Three cylindrical specimens of 100X150 mm diameter were tested for each mixture [67]. For SSR test, two temperatures, high temperature T_H at 54.0°C calculated by degree-days parameter equation in AASHTO TP-134-19 and low temperature T_L 20.0°C based on the climatic Performance Grade (PG) were considered. For each temperature, a vertical load is applied for 600 cycles at three deviatoric stress levels of (200 loading block for each segment) [68, 58]. A confining stress of 69.0 KPa was applied with a loading pulse of 0.4 s and followed by a rest time of 3.6 s for T_H and 1.6 s for T_L . The SSR test allows to determine the shift modeling for permanent strain using damage modeling tool for rutting (FlexMATTM rutting) the tool and the rut depth analysis FlexPAVETM.

2.3 Performance-based pavement design

Pavement design is considered a dynamic and structurally long-term evaluation process based on the availability of the new data obtained from the existing road pavement structures. It involves the selection of materials and the calculation of the required thickness for the different layers of the road pavement structures. The engineered structure produced by the pavement design is to withstand heavy loads and environmental impacts that helps to minimize the materials, failure criteria, construction deficiencies, maintenance hurdles and whole life cost of the road pavement structure [45]. Stiffness is considered as one of the most important property that should be considered by the selection of materials for each layer of the pavement structure. The stresses and strains produced in the different layers of the pavement structure are beared by the materials used in the structure. As the top layer of the road pavement structure has more stiffness and less thickness as compared to the underneath layers of the road pavement [69]. Conventional design process of the pavement involves the mechanistic approach to predict its factors like loading characteristics, environmental conditions, materials, layer properties and failure criterion. These factors are further explained in section 2.3.1 of this chapter.

2.3.1 Factors affecting the pavement design

2.3.1.1 Loading Characteristics

The load coming from the vehicles is one of the most important factors that influences the performance of pavement design. Saraf et al, investigated the effect of loading conditions on a limited number of different road pavements in the states of Ohio and Michigan in USA for the assessment pavement damage [70]. Mostly, rutting was caused in flexible pavements during the critical distress measurements, while cracking and faulting were observed in rigid pavements due to heavy traffic loading. The speed of the vehicle, contact area of the tire, and axle load of the vehicle affects the overall performance and the materials used in the pavement structure. The high speed of the vehicle produces smaller strains due to the larger modulus produced in the pavement structure that increases its fatigue life. This is especially relevant to flexible pavements due to visco-elastic behavior found in asphalt concrete [42, 69, 19].

The contact area of the tire with the pavement surface is essential to predict the stress distribution imposed by loads. For the distribution of loads along the pavement structure, the SHRP and Post-SHRP have focused on three main types of distress, i.e., permanent deformation or Rutting, fatigue cracking and thermal cracking. As shown in figure 2.6., under the standard axle load, the horizontal tensile strain at the bottom of the asphalt layer causes fatigue cracking, while the vertical compressive strain at the top of the subgrade layer causes rutting. The contact area and the pressure of the tire play an important role for the evolution of these distresses and the pavement performance based on various empirical and mechanistic-empirical methods [19].

For the uniform distribution of axle load, the contact area between the tire and pavement needs to know. For the evaluation of the damage that occurred in pavements using traffic Mechanistic Empirical Pavement Design Guide (MEPDG), there is a requirement of a full axle load spectrum coming from the traffic.

2.3.1.2 Environmental conditions

The performance and the design of the pavement can be effected by the environmental conditions. The external factors such as temperature, moisture, and freeze-thaw actions have a significant role in defining the impacts of the environment on the performance and design of the road pavements [71]. There is an importance of climatic conditions in MEPDG and the materials used in pavement structures. If the material of the pavement structure is susceptible to a moisture, becomes saturated, and the stiffness of the material is reduced. The stresses taken by the dry material lead the layers of the pavement to a failure. The elastic moduli of the various pavement layers are affected by temperature and moisture [72]. The visco-elastic behavior of the asphalt changes the resilient modulus of the asphalt due to the change in temperature. For example, rutting occurs at high temperature due to the softness of the asphalt, while fatigue and thermal cracking occur at low temperature due to the hardness of the asphalt. The unprecedented climatic change is adversely affecting the performance and design of the pavement infrastructure. According to Federal Highway Administration (FHWA) a report published in 2015 Climate change adoption of pavements emphasized on the calibration of prediction models used for the pavement performance

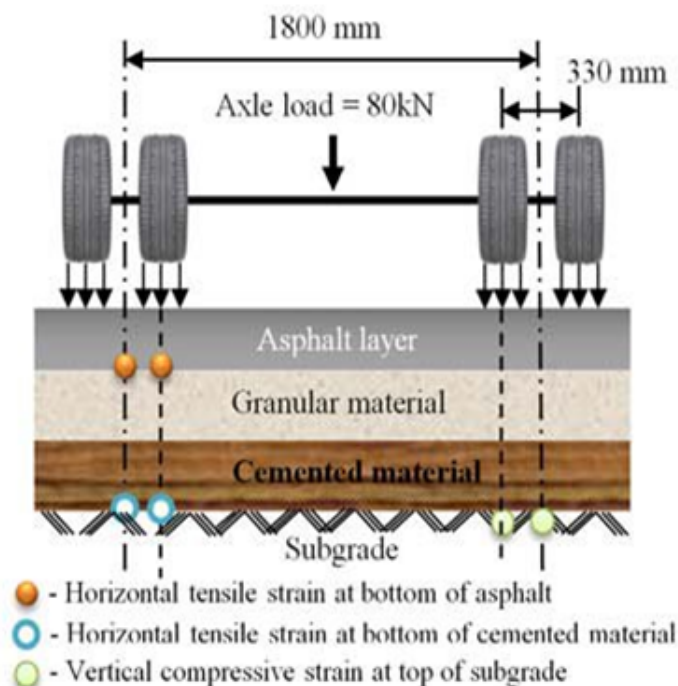


Figure 2.6: Standard load application on pavement structure [19]

in terms of rutting, fatigue cracking and riding quality. These models are currently based on the historical data. There is a need of proactive monitoring to have safe, durable and efficient road infrastructures. Katie et al, studied the impact of climatic change on pavement design and rehabilitation practices and concluded that a research gap exists in PMS due to the lack of continuous monitoring of the climatic data [3]. The key pavement indicators for a climatic change that needs to be monitored for flexible and rigid pavements are listed in Table 2.4. The continuously monitored climatic data and its routine record will help the pavement asset managers for the proactive adjustment of their selection of materials and pavement design.

Table 2.4: Key Parameters to Monitor for Climatic Changes [3]

Sr. No	Flexible Pavements Indicators	Rigid Pavement Indicators
1	Fatigue cracking/potholes	Freeze-thaw durability
2	Rutting of bound and unbound layers	Faulting, corner breaks, and pumping actions
3	Stripping action	Slab warping
4	Block cracking	Punch-outs (CRCP)
5	Raveling	Joint spalling in concrete pavements
6	Low-temperature cracking	Concrete slab cracking

2.3.1.3 Layer Properties

A pavement structure is composed of different layers, i-e surface layer, a base layer, and a soil subgrade, as shown in figure 2.7, depending upon the load and climatic conditions [73]. For the ideal road pavement structures the

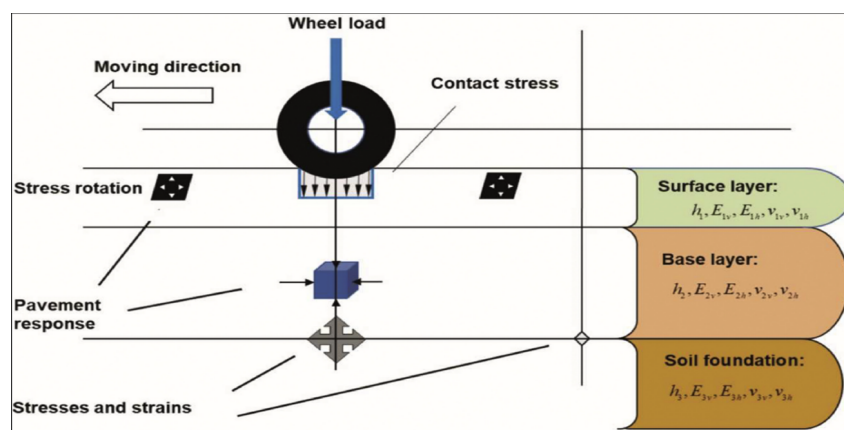


Figure 2.7: Stress strain distribution in flexible pavement structure [4]

modulus of the materials in pavement layers, should be reduced from top to bottom [59]. The reaction of the greater stresses produced on the top layer of the pavement structure depends upon the stiffness of the material. Besides making the road pavement construction economical, one of the reasons to put the superior quality materials on top layers is to make the wearing surface stiffer and easy to distribute and transfer the stresses to the lower layers without inducing any damage. As a result, lower moduli values can be used for the lower layers. Each of the layer's thickness and modulus has the direct impact on the bearing capacity of the road pavements. But significantly, the properties of the subgrade and its each individual structural layer are enhanced by the bearing capacity of the top layer, as the stresses induced due the concentration of loads (top to bottom) [74].

Mostly research studies focused on in-situ pavement testing, mathematical modeling and numerical simulations to find out the behavior of the stresses induced by the wheel loads. Each method has several limitations; for example, the stresses and strains induced in multi layers of the road pavement structures need to be monitored under the wheel load as shown in figure. 2.7. To have a realistic remaining life of the road pavements, recycling of layers, accurate pavement design, better evaluation for maintenance and rehabilitation along with the management activities of the road pavements, and the essential layers properties summarized in Table.2.5, needs to be properly monitored in the field [4]. Most of these properties are rarely monitored based on real time conditions.

2.3.1.4 Failure criteria in road pavements

Failure in road pavement structures revolved around the progression of the damage. In fact, the damage in the road pavement structure is a change that affects its current or future performance by providing a loss in optimal or complete functionality. Since 1960, the evaluation of the road pavement surfaces as being satisfactory or unsatisfactory was purely based on expert opinion [75]. To overcome this issue, it Present Serviceability Index (PSI) was introduced by investigating 74 road pavement sections to evaluate the condition on a scale of 0 (poor) to 5 (excellent). The average rating of each road pavement section for converting the discrete data into continuous form was termed as Present Serviceability Rating (PSR) [76]. The later advancements, like PSI, Pavement Quality Index (PQI) was used to evaluate the pavement surface conditions based on manual data collection methods [77, 78]. International Roughness Index (IRI) were developed as a standard tool for FHWA the Monitoring System. However, the IRI measurement requires costly devices. Another example of evaluating

Table 2.5: Pavement layer properties for pavement monitoring [4]

Sr. No	Property Name	Category			
		Pavement Design	Mix Design	Pavement Monitoring	Pavement Management
1	Binder Content (Bound Layers)	X	X	X	X
2	Layer thickness	X	X	X	X
3	Permanent Deformation properties	X	X	X	X
4	Residual stresses (in-situ)	X	X	X	X
5	Fatigue Properties (Load and thermal)	X	X	X	X
6	Elastic stiffness (Each layer) [Elastic modulus or stress-strain curve properties stress dependent materials]	X	X	X	X
7	Other properties	X	X	X	X

road pavement conditions includes VIZIR, based on three damage approaches, i.e., rutting cracking and thermal cracking, by manual data collection or LCPC's DESYROUTE device [78]. Beside these methods the Pavement Surface Evaluation and Rating Manual (PASER) used the photography method [78]. But this method was very costly in terms of field implementation.

The Australian Road method was based on visual assessment [79]. The advantages, deficiencies, cost, field application, and integration with PMS has been summarized in Table 2.6. The effectiveness and the reliability of these discussed methods are based on manual data collection or the reactive monitoring techniques. The most important research gap is the consideration of proactive approach either based on static or dynamic monitoring, to monitor the data and easily available resources for feeding, PMS as shown in figure 2.8. An improved and proactive monitoring approach can handle the inaccurate measured data, cost effectiveness, field deficiencies, and the reliability issues.

Failure criteria of the road pavements generally depend upon the adopted guidelines and methods of the pavement design [80]. For example, the American Association of State highway and Transportation officials (AASHTO) method is used PSI for the condition of the pavements [20]. In the mechanistic empirical design approach three principal types of distresses that effects pavement design and its performance, i-e fatigue cracking, rutting, and, to some extent, thermal cracking [20].

The distresses occurred in pavements are the external indicators of their performance, requiring monitoring on real time conditions. For example, under the repeated cyclic loading the horizontal tensile strain occurring at the bottom of the asphalt layer causes fatigue cracking, and the vertical compressive strain at the top of the subgrade causes permanent deformation, while the thermal fatigue cracking can be caused by the horizontal tensile strain at the bottom of the asphalt layer with daily cyclic temperature [20]. The root cause and severity of these distresses can

Table 2.6: Pavement distress evaluation methods reproduced from [5]

Features of methods	PSI	PCI	PASER	VIZIR	AUSTRALIAN
Benefits	No experts required	Accuracy measurement	Less cost and user friendly	User friendly	Accurate and predict the future conditions
Deficiencies	Less distress information	Expert panel requirement	Subjective	Less distress information	Use of special equipment's
Reliability	Moderate	High	Moderate	Moderate	Highest
Simplicity	Fair	Fair	High	Yes	No
PMS integration	X	X	X	X	X
Cost analysis	Moderate	High	Lowest	Moderate	Very high
Real time monitoring	Not evident	Not evident	Not evident	Not evident	Evident but reactive
Scope of Field Implementation of pavement Performance	Pavement maintenance on priority	Pavement maintenance (objective)	Pavement maintenance (objective)	Asset of PMS	

be departure from the loading, environmental conditions, materials and construction deficiencies of the pavements requires a proper and real time monitoring in the different field conditions.

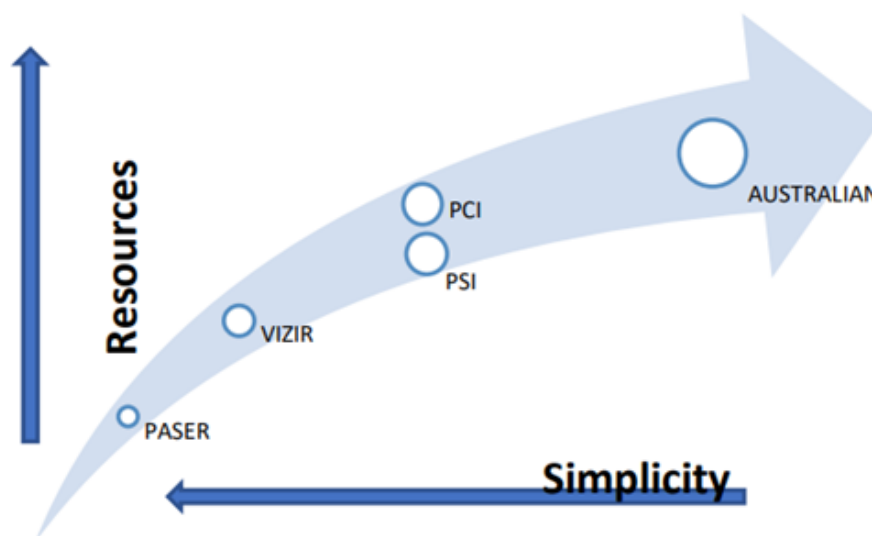


Figure 2.8: Types of pavement management system [20]

2.3.2 Types of flexible pavement design

Flexible pavement design is divided into five categories based on the failure approaches [81].

1. Empirical method
2. Limiting shear failure method
3. Limiting deflection method
4. Regression method based on pavement performance or road test

5. Mechanistic-empirical method

2.3.2.1 Empirical method

The empirical method was purely based on the soil classification system by the public roads (PR). Later, the highway research board modified the method by classifying the soils from group A1 to A8 [28]. But Steele in 1945 suggested an approach to consider the thickness of the pavements. But this method was having a drawback that there was no onsite or laboratory testing involved to quantify the strength of materials.

2.3.2.2 Limiting Shear Failure Methods

This type of method was developed between 1943 and 1946 by Barbar to evaluate the thickness of pavement using Terzaghi bearing capacity to limit the shear failure. This method was especially focusing on the subgrade using the cohesion and internal angle of friction. One of the main disadvantages of this method was not taking into the account of traffic speed and volume that may affect the quality of ride for a road user [82].

2.3.2.3 Limiting deflection method

This method of the pavement design was used to limit the vertical deflection, and the thickness of the pavement structure was determined. Kansas State Highway Commission (1947) has limited the deflection upto 0.1 inch. The U.S. Navy (1953) applied Burmister's two-layer theory (Burmister, 1943) has modified the surface deflection upto 0.25 inch. Although this method has an advantage of measuring the deflection in field but the stress and strain measurement which is a main source of failure was not under consideration [83].

2.3.2.4 Regression method based on pavement performance or road test

A common example of using regression equations in pavement design is the AASHTO method, which is derived from road test data [82]. However, a key limitation is that the design equations only apply to the specific conditions at the test site. When trying to use them in different environments, significant adjustments based on theory or practical experience are required. While these equations help demonstrate how different factors impact pavement performance, their overall value in pavement design is limited due to the many uncertainties involved.

2.3.2.5 Mechanistic-empirical method

The Mechanistic-empirical design approach focuses on understanding how materials behave under specific conditions, such as when a wheel load is applied. It connects these inputs to measurable pavement responses like stress or strain. These response values help engineers predict potential pavement deterioration by analyzing

both lab experiments and real-world performance data. Since theory alone doesn't provide a complete picture, incorporating observed performance is crucial for designing pavements that hold up in real-world conditions. Dormon and Metcalf first introduced this concept in the U.S. back in 1965 [82]. The method offers several advantages: it enhances the reliability of pavement designs, allows engineers to forecast different types of pavement failures, and enables predictions even when only limited lab and field data are available. By combining theory and empirical data, the Mechanistic-empirical method helps make more informed and accurate design decisions.

2.4 Road pavement performance

Transport infrastructures are considered as an important pillar of pavement engineering, with an aim of providing aid in terms of economical and vast mobility to modern society. Growing traffic and expansion of road networks increase and complicated a demand of maintenance and rehabilitation activities of road infrastructures [21]. One of the biggest challenges for the pavement engineers and decision maker's is to effectively maintain and rehabilitate the pavement networks with limited resources and increasing demand [21]. The effective maintenance and rehabilitation decision taken at the right time and in the right place, helps to improve the pavement performance, extend the service life, and provide the economic benefits to pavement networks.

As shown in figure 2.9., Galehouse et al. (2003) concluded that the spending of \$1 on right pavement structure at a right time can save or delay from \$6 to \$10 in rehabilitation or reconstruction process [21]. In this era the pavement engineers are trying to find out the solutions that improves the service life of the pavement structures that has a direct impact on the performance of the road pavements.

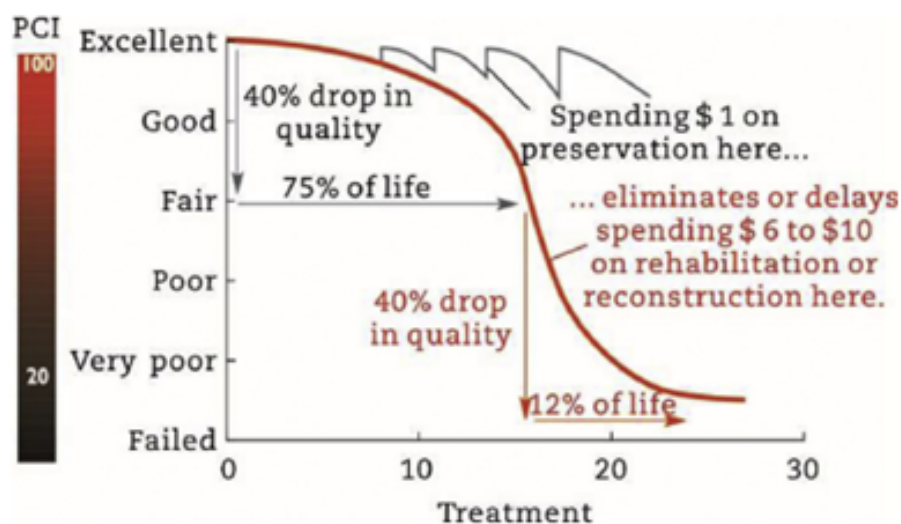


Figure 2.9: Pavement condition, reconstruction and rehabilitation treatments and its cost reproduced from [21]

2.4.1 Types of performance models

The modeling of the road pavement structure plays an important role in terms of its management and the whole life cycle assessment. In broader terms, there are two types of models that defines the performance of the road pavements, i.e., deterministic models and probabilistic models. The structural, functional, damage performance, along with the primary response by the road structures, are included in deterministic models, while the probabilistic models include survivor curves and Markov and semi-Markov transition processes [84, 85]. Here it is to note that this research study will solely focus on the structural performance models as a main type of deterministic model that are empirical in nature that requires the structural data as a main input for its calibration.

2.4.1.1 Structural Performance Models

Structural performance models are the types of models that include modeling of the distresses that occur in pavement structures after severe loading and environmental conditions [86]. It is observed that these models are either empirical or mechanistic-empirical in nature. These types of models record the pavement response coming from the loading. According to FHWA, the Performance Related Specifications (PRS) utilized structural models, predict how pavements behave under load. However, for more precise strain predictions, it's necessary to define a relationship between stress and strain in each model layer that closely mirrors real-world conditions. Given that asphalt concrete behaves as a viscoelastoplastic material, its actual responses are influenced by a combination of elastic, plastic, and time-dependent (viscous) factors. These responses are also affected by variables like temperature, the intensity and duration of the load, and the level of strain. Therefore, it's critical to include mixture-specific linear viscoelastic properties to achieve a more accurate analysis of pavement performance [87]. The models developed, the FHWA like linear viscoelastic model, SVECD and shift model are mechanistic performance models can estimate the stress strain relationship in a more accurate manner.

2.4.1.2 Linear viscoelastic model

The viscoelastic behavior of the asphalt concrete is difficult to measure directly in the field due to the small strain levels and different parameters like loading frequency and temperature. Usually, it is believed that small strain occurs in asphalt concrete between 50 and 75 micro strains [87]. This theory evaluates the effect of loading frequency and the temperature by a single parameter that is known as the reduced frequency to generate the master curve for a dynamic modulus $|E^*|$ [88]. This curve can define the behavior of material by considering the effect of loading and temperature. Equation 2.6 can be utilized to evaluate the dynamic modulus at any frequency

$$\log |E^*| = a + \frac{b}{1 + \frac{1}{e^{d+g \cdot \log(fr)}}} \quad (2.6)$$

- e = Euler's number ($e \approx 2.718$).
- a , b , d , and g = optimized constants.

- f_r = reduced frequency.

The time-temperature superposition principle can be applied to generate a single master curve. This shifted frequency is also known as reduced frequency and can be calculated using the following equation: 2.7 and 2.8

$$f_r = a_T f \quad (2.7)$$

$$\log(a_T) = a_1 T^2 + a_2 T + a_3 \quad (2.8)$$

Where:

a_T = $t - T$ shift factor,

f = loading frequency (in Hz),

a_1, a_2, a_3 = shift factor coefficients,

T = temperature.

The dynamic modulus is known to be beneficial rather than calculating the resilient modulus because of its combination with phase angle that becomes a complex modulus (E^*) which is quite helpful in providing the information regarding the stress and strain under the sinusoidal loading. One of the major advantages of complex modulus is that it can be produced mathematically from frequency dependency to time dependency like a relaxation modulus $E(t)$. The $E(t)$ in equation 2.9 can be used to evaluate the stress at applied strain. The $E(t)$ can be calculated using the following equation

$$E(t) = E_\infty + \sum_{i=1}^m E_i e^{-\frac{t}{\rho_i}} \quad (2.9)$$

Where:

E_∞ = elastic modulus,

t = time,

m = number of Maxwell elements,

E_i = modulus of the i th Maxwell element,

ρ_i = relaxation time of the i th Maxwell element.

2.4.1.3 Fatigue performance model

The main purpose of explaining the linear viscoelastic model is to provide a background about its simplified version discussed in section 2.4.1.4 of this chapter. The simplified version considers the behavior of asphalt materials on a macroscale rather than involving a microscale, as shown in figure 2.10. The Viscoelastic Continuum Damage (VECD) theory explains how stress and strain interact as microcracks form and grow within a material, but it doesn't account for how the material ultimately fails [89].

There are two basic components of the simplified theory: one is the damage, and the other one is the stiffness of the asphalt materials. The structural integrity of the materials can be evaluated using the pseudo stiffness, which can be calculated using the secant modulus in experimentation [22]. Damage in materials is frequently challenging to measure, often requiring detailed and rigorous theoretical frameworks. A prominent example is Schaper's work potential theory, which is designed to describe elastic materials as they undergo progressive damage [90]. This theory is rooted in the principles of thermodynamics, particularly in the context of irreversible processes. It introduces the concept of an index Internal State Variable (ISV), which serves as a quantitative measure of damage. This ISV reflects the internal microstructural changes happening within the material as it deteriorates over time, providing a clearer way to assess and understand the extent of damage [12].

The theory employs the correspondence principle as a key step to manage the time-dependent nature of viscoelastic materials. By doing so, it effectively decouples the time-related effects tied to viscoelasticity. This is achieved through transforming the physical strain experienced by the material into a pseudo strain. Once this transformation occurs, the viscoelastic problem can be treated similarly to an elastic one, allowing the same mathematical equations used for elastic materials to be applied [12]. The VECD theory is built around this foundational approach and is composed of a series of core equations. These equations allow for a detailed analysis of how viscoelastic materials respond under stress, including how damage progresses over time. VECD theory thus bridges the gap between elastic and viscoelastic behavior by using pseudo strain as a tool to simplify complex time-dependent damage processes [12].

The pseudostrain energy density function, the stress–pseudostrain relationship, and the damage evolution law are fundamental components of the theory and are mathematically expressed through equations 2.10, 2.11, and 2.12

$$W^R = f(\varepsilon^R, S) \quad (2.10)$$

$$\sigma = \frac{\partial W^R}{\partial \varepsilon^R} \quad (2.11)$$

$$\frac{dS}{dt} = \left(-\frac{\partial W^R}{\partial S} \right)^\alpha \quad (2.12)$$

Where:

W_R = pseudostrain energy density,

ε_R = pseudostrain,

S = damage parameter (ISV),

α = damage evolution rate.

The uniaxial mode of loading is computed using equation 2.13 and can be expressed as follows:

$$W^R = \frac{1}{2}(\varepsilon^R)^2 C \quad (2.13)$$

In this context, C , is known as the pseudo stiffness, is the only variable that depends directly on the damage state

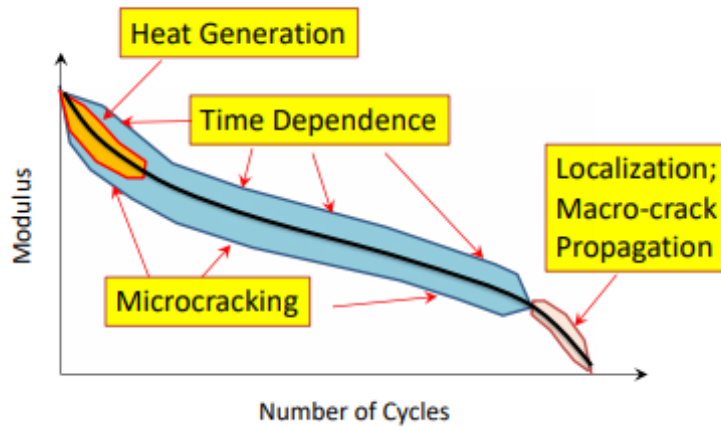


Figure 2.10: Cracking mechanics in asphalt concrete [22]

S. As damage progresses within the material, the pseudo stiffness C changes, reflecting the material's decreasing ability to resist deformation. This relationship between C and S is key to understanding how damage affects the material's mechanical properties, as it links the material's stiffness to the extent of internal degradation over time. When incorporating the relationship between pseudo stiffness C and the damage state S , the damage evolution law can be expressed through a equation 2.14

$$\frac{\partial S}{\partial t} = \left(-\frac{1}{2}(\varepsilon^R)^2 \frac{\partial C}{\partial S} \right)^\alpha \quad (2.14)$$

Eventually the chain rule has been applied by Kim and Lee shown in equation 2.15

$$\frac{dC}{dS} = \frac{dC}{dt} \frac{dt}{dS} \quad (2.15)$$

After simplification, the equation for calculating damage at each time increment is presented as equation 2.16. In this formulation, the standard time step Δt is replaced with the reduced time interval $\Delta \xi$. This modification is important because it improves the alignment with the time–temperature superposition (t–TS) principle, especially in cases where damage develops within the material.

By employing $\Delta \xi$ instead of Δt , the equation 2.16 more accurately represents the behavior of the material as damage builds up. The t–TS principle indicates that time and temperature effects can be treated as interchangeable under specific conditions, which is particularly applicable to viscoelastic materials that undergo changes in mechanical properties over time due to damage in equation 2.16.

$$dS_i = \left(-\frac{1}{2}(\varepsilon_i^R)^2 \Delta C_i \right)^{\frac{\alpha}{1+\alpha}} \times (\Delta \xi_i)^{\frac{1}{1+\alpha}} \quad (2.16)$$

2.4.1.4 SVECD model

One of the main problems in the viscoelastic model was that the calculation of pseudostiffness, pseudostrain, and damage requires extensive loading, and it was a difficult, time-consuming, and expensive process for its computation [12]. To resolve this issue, Underwood et al, has developed a cyclic loading for SVECD model [91]. The procedure was to define the important parameters as shown in figure 2.11. This figure shows two types of pseudostiffness: one is a total value based on pseudostrain $\bar{\varepsilon}$, and the other is a value based on cyclic magnitude (F). Figure 2.11 and equation 2.17, 2.18 and 2.19 illustrate the relationship between these pseudostiffness values, both visually and mathematically.

$$\bar{C} = \frac{\sigma_{0,ta}}{\varepsilon_m^R * I} = \frac{\sigma_{0,ta}}{(\varepsilon_{0,ta}^R + \varepsilon_s^R) * I} \quad (2.17)$$

$$F = \frac{\sigma_{0,ta}}{\varepsilon_{0,ta}^R * I} = \frac{\sigma_{0,pp}}{\varepsilon_{0,pp}^R * I} = \frac{\sigma_{0,ta}}{(\varepsilon_m^R - \varepsilon_s^R) * I} \quad (2.18)$$

$$\bar{C} = F \frac{\varepsilon_m^R - \varepsilon_s^R}{\varepsilon_m^R} \quad (2.19)$$

where

ε_{R_m} = absolute pseudostrain at peak,

$\varepsilon_{0R,ta}$ = pseudostrain tension amplitude,

$\varepsilon_{0R,pp}$ = peak-to-peak pseudostrain amplitude,

ε_{sR} = permanent pseudostrain,

$\sigma_{0,pp}$ = peak-to-peak stress amplitude,

I = specimen-to-specimen variability factor.

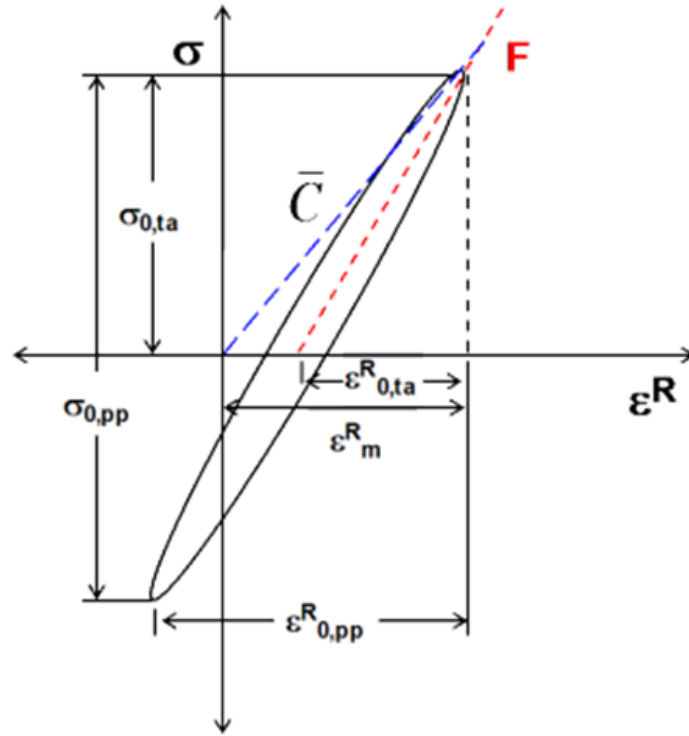


Figure 2.11: Explanation of stress, pseudostiffness and pseudostiffness in SVECD model [12]

Underwood et al, emphasizes that this approach ensures accuracy during the initial loading phase, where damage accumulation is most significant. By using a rigorous calculation in equation 2.20 and 2.21, the model captures the critical early-stage behavior [91]. However, for the remaining cycles, where damage grows more slowly, the simplified method is both efficient and sufficient. This stepwise method reduces the complexity of calculations without sacrificing accuracy in later stages. Therefore, both pseudostrain and pseudostiffness follow a piecewise definition, ensuring the model adapts to varying levels of damage across the loading history.

$$\varepsilon^R = \begin{cases} \varepsilon^R = \frac{1}{E_R} \int_0^\xi E(\xi - \tau) \frac{d\varepsilon}{d\tau} d\tau & \text{for } \xi \leq \xi_p \\ \varepsilon_{0,ta}^R = \frac{1}{E_R} \cdot \frac{A_{\text{tension}}}{2} + 1 ((\varepsilon_{0,pp}) \cdot |E^*|_{LVE}) & \text{for } \xi > \xi_p \end{cases} \quad (2.20)$$

Where: ER = reference modulus

$$C = \begin{cases} C = \frac{\sigma}{\varepsilon^R \cdot DMR} & \text{for } \xi \leq \xi_p \\ \bar{C} = \frac{\sigma_{0,ta}}{\varepsilon_{0,ta}^R \cdot DMR} & \text{for } \xi > \xi_p \end{cases} \quad (2.21)$$

To handle the potential for significant damage in the initial loading phase, the team used a detailed damage calculation method for the first cycle, shown in equation 2.22 . After this, a simpler calculation was used for subsequent cycles. The first part of the damage calculation is called the transient calculation (dsTransient), and the rest are referred to as cyclic calculations (dsCyclic). The simplified method assumes that the pseudostrain remains steady throughout each cycle. To correct any inaccuracies from this assumption, the team introduced an adjustment factor, K1 calculated using 2.23, which works well when damage growth during a cycle is minimal.

$$dS = \begin{cases} (dS_{\text{Transient}})_{\text{timestep } j} = \left(-\frac{1}{2}(\varepsilon_j^R)^2 \frac{\partial C}{\partial S}\right)^\alpha \cdot (d\xi)_j & \text{if } \xi \leq \xi_p \\ (dS_{\text{Cyclic}})_{\text{cycle } i} = \left(-\frac{1}{2}(\varepsilon_{0,ta}^R)^2 \frac{\partial C^*}{\partial S}\right)^\alpha \cdot (d\xi_p) \cdot (K_1) & \text{if } \xi > \xi_p \end{cases} \quad (2.22)$$

$$K_1 = \frac{1}{\xi_f - \xi_i} \int_{\xi_i}^{\xi_f} (f(\xi))^{2\alpha} d\xi \quad (2.23)$$

Where:

ξ_i = starting time of a cycle,

ξ_f = ending time of a cycle.

The research team carried out displacement-controlled crosshead (CX) cyclic fatigue tests at a frequency of 10 Hz, adjusting the temperature according to the binder PG to assess its viscoelastic damage properties [91]. They conducted these tests using three to four different strain amplitudes, ranging from high to low, to capture a broad range of cycles to failure (Nf) spanning from 1,000 to 100,000 cycles. Prior to starting the CX cyclic fatigue tests, they performed fingerprint dynamic modulus tests to evaluate the consistency of the test specimens and ensure reliable results [91]. The dynamic modulus measured in this test is denoted as $|E^*|_{\text{fingerprint}}$, which is utilized in equation 2.24 to determine the Dynamic Modulus Ratio (DMR). The term $|E^*|_{\text{LVE}}$ refers to the linear viscoelastic dynamic modulus of the material at the specific temperature and frequency of the test. The DMR value between 0.9 and 1.1 ensures that the linear viscoelastic properties derived from the dynamic modulus tests are effectively incorporated in the SVECD analysis. To minimize variability, the research team employed the DMR in their calculations using equation 2.24 .

$$DMR = \frac{|E^*|_{\text{fingerprint}}}{|E^*|_{\text{LVE}}} \quad (2.24)$$

2.4.1.5 Failure Criterion of asphalt mixtures

One of the established failure criteria for asphalt concrete is that the modulus value should not decrease to 50% during fatigue tests [88]. While the tests can readily measure the modulus value of the mixture, and the analysis can easily compute it, evidence suggests that the relationship between this 50% modulus reduction indicator and the actual failure of the material is uncertain [92]. The model for determining failure criteria should be based on the fundamental variables used in testing, namely strain, stress, and strain energy. This approach would enable predictive models to incorporate these variables effectively. By deriving the failure criteria from these fundamental elements, it would be possible to predict fatigue failure under various loading conditions, including those commonly encountered in pavement simulations.

In 2010, Hou et al. presented a failure criterion for the **SVECD** model that assumes failure occurs when C reduces to a critical value, C_f . However, these researchers observed high variability from their experiments [6]. Therefore, the research team does not consider this parameter as a reliable failure criterion for use in the **VECD** and **SVECD** models. The summary of all the previous criterion is summarized in table 2.7

Table 2.7: Fatigue Failure Criteria [6]

Sr No	Description	Advantage	Research Gap
1.	50% reduction in asphalt modulus	Easily measurable and computable	Real failure of the material is questionable
2.	Drop in phase angle	More reliable than 50% reduction in asphalt modulus	Predictive models such as structure models cannot use it as failure criterion
3.	Dissipated strain or pseudostrain energy using asphalt concrete	Accurate relation between plateau value and number of cycles till failure (N_f)	Only based on loading mode
4.	Dissipated pseudostrain energy Dissipated Pseudo strain Energy (DPSE)	Elastic-viscoelastic correspondence principle can better determine DPSE due to damage in viscoelastic materials	Can only apply this method in experiments for which they know the histories of stress, strain, and phase angle in advance
5.	A failure criterion for the SVECD model	Assumes failure occurs when C reduces to a critical value, C_f	High variability from experiments

2.4.1.6 New failure criterion

As discussed in the above section, the limitations in failure criterion, the researchers have developed the two new criterion for **SVECD** model i-e G^R and D^R failure criterion [23, 12]. These criteria were mainly developed from the experimental results obtained from dynamic modulus and cyclic fatigue testing using AASHTO T 378 and AASHTO TP 107 [89]. To understand these criterion, there is a need to explain the background theory and the testing modes used in cyclic fatigue testing. One of the modes was control actuator displacement **CX**. The other mode was control stress mode coded as **CS** while the third mode control on specimen strain mode **COS**. The

researchers conducted all cyclic tests at a frequency of 10 Hz, using varying amplitudes to encompass a broad range of N_f values, spanning from 1,000 to 100,000 [89]. The main theme to define the failure was by observing the drop in phase angle. But in COS mode the phase angle was observed almost constant. For the purpose of consistency, the COS mode was used by considering DPSE in cyclic tension testing. When asphalt concrete experiences CX cyclic loading, it follows wave-like patterns, as demonstrated in the equations provided. Equation 2.25 and 2.26 demonstrates the behavior of the stress and strain at i th cycle.

$$\varepsilon = \varepsilon_i \sin(\omega t) \quad (2.25)$$

$$\sigma = \sigma_i \sin(\omega t + \phi_i) \quad (2.26)$$

Where:

ω = angular frequency,

σ_i = stress amplitude measured at cycle i ,

ε_i = strain amplitude measured at cycle i ,

ϕ_i = phase angle measured at cycle i .

When the research team graphed the data against strain shown in Figure 2.12, the area within the hysteresis loop represented the total energy lost during each cycle [23]. This energy dissipation stems from two primary sources: viscous damping due to the material's viscoelastic properties and the formation of new microcracks as fatigue damage progresses. By applying the correspondence principle and replacing physical strain with pseudostrain, the impact of viscoelasticity is eliminated from the total energy dissipation, as demonstrated in equation 2.27. After the test reaches a quasistatic state, expressing the pseudostrain amplitude at cycle i in equation 2.28 simplifies the calculations.

$$\varepsilon^R(t) = \int_0^t E(t - \tau) \frac{d\varepsilon}{d\tau} d\tau \quad (2.27)$$

$$\varepsilon_i^R = |E^*| \cdot \varepsilon_i \cdot \sin(\omega t + \phi) \quad (2.28)$$

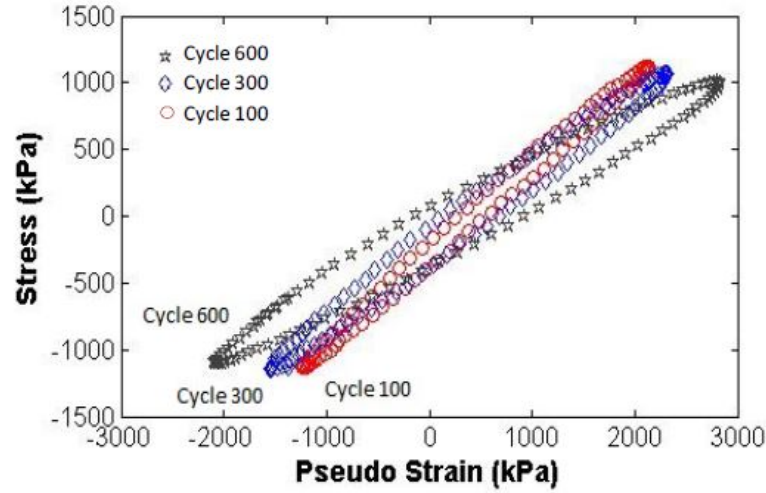


Figure 2.12: Pseudohysteresis analysis in CX mode of cyclic fatigue test [12]

Where:

$\varepsilon_R(t)$ = pseudostrain at time t ,

ε_{R_i} = pseudostrain amplitude at cycle i ,

$E(t)$ = relaxation modulus of the asphalt mixture,

$|E^*|$ = dynamic modulus value at the corresponding frequency and temperature.

The area of the newly generated hysteresis loop in the stress-pseudostrain plot represents the DPSE (WRi). DPSE and pseudostiffness are calculated using equations 2.29 and 2.30 respectively.

$$W_i^R = \pi \sigma_i \varepsilon_i^R \sin(\phi_i - \phi) \quad (2.29)$$

Where: π = Archimedes' constant, approximately equal to 3.14159.

$$C = \frac{\sigma_i}{\varepsilon_i^R} \quad (2.30)$$

During the process of cyclic loading, Highest Amount of Stored Pseudostrain Energy (WRmax) for each cycle provides insight into the material's current damage level. WRmax occurs at the point of maximum stress in each cycle, which coincides with the peak pseudostrain, and the greatest damage sustained during that cycle. Utilizing this concept of C, researchers can calculate WRmax for a particular cycle using equation 2.31 and 2.32

$$W_{\max}^R = \frac{1}{2} (\sigma_{0,ta}) (\varepsilon_{0,ta}^R) \quad (2.31)$$

$$W_{\max}^R = \frac{1}{2} C \cdot (\varepsilon_{0,ta}^R)^2 \quad (2.32)$$

Where:

$\sigma_{0,ta}$ = tension amplitude of the stress,

$\varepsilon_{R0,ta}$ = pseudo strain amplitude for the duration of the stress being in tension.

As damage builds up, the material's ability to store energy diminishes. The gap between the currently stored $W_{\max}$ and the equivalent state of undamaged material represents the WRC . Equation 2.33 provides the formula for calculating WRC at a specific cycle, while figure 2.13 illustrates the method in detail.

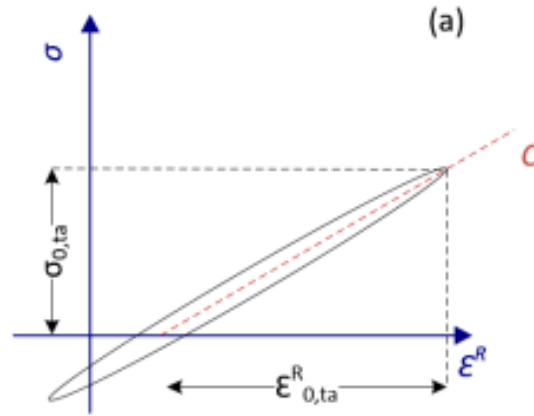


Figure 2.13: Total dissipated pseudostrain energy (WRC) [12]

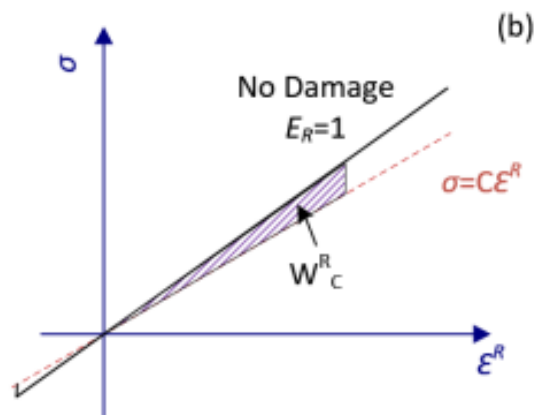


Figure 2.14: Extent of damage occurrence [12]

Moreover, the $SVECD$ model's underlying theory demonstrates a relationship between this total $DPSE$ and the damage's current state shown in figure 2.14. The damage evolution law in the $SVECD$ model is shown in equation 2.33.

$$W_C^R = \frac{1}{2}(1 - C) (\varepsilon_{0,ta}^R)^2 \quad (2.33)$$

This ISV is "associated with distributed damage or, perhaps a better term, microstructural changes," according to Schapery in equation 2.34.

$$W_C^R = \frac{1}{2}(1 - C) (\varepsilon_{0,ta}^R)^2 \quad (2.34)$$

Therefore, in addition to microcracking, these modifications may also involve local plastic deformation, dislocation motion, and generation [23, 12]. Additionally, as seen in equation 2.35, researchers have discovered a monotonic connection with pseudostiffness.

$$1 - C = aS^b \quad (2.35)$$

Where a , b can be obtained as regression coefficients Temperature, stress and strain amplitude, and loading mode have no effect on this connection. Equation 2.36 is created by combining equations 2.34 and equation 2.35 and integrating both sides of the equation after separating damage (S) and temperature (t).

$$S^{\alpha+1} = \frac{b^\alpha}{(\alpha+1)} \int_0^t \left(\frac{1}{2}(1 - C) (\varepsilon^R)^2 \right)^\alpha dt \quad (2.36)$$

Equation 2.34 and equation 2.35 change when expressed in a discrete form for cyclic loading (more information is available in Underwood et al.). The equations becomes,

$$S^{\alpha+1} = \frac{b^\alpha}{\alpha+1} \sum_1^N \left(\frac{1}{2}(1 - C_i) (\varepsilon_{0,ta}^R)_i^2 \right)^\alpha K_{1,i} t_i \quad (2.37)$$

$$S^{\alpha+1} = \frac{b^\alpha}{\alpha+1} \sum_1^N (W_{C,i}^R)^\alpha K_{1,i} t_i \quad (2.38)$$

Here, the pseudostrain amplitude under tension at cycle $i = (\varepsilon^R 0, ta)_i$.

C_i = cycle i pseudostiffness,

t_i = cycle i 's loading time,

$K_{1,i}$ = loading pattern-related shape function.

The right side of equation 2.37 refers to the complete loading history prior to the current cycle, while the left

side represents the damage status at cycle i . Furthermore, the damage during that cycle is connected to the total DPSE when it is elevated to the power of alpha shown in equation 2.38 [23, 12]. The modulus term $(1-C)$ denotes the ability of the material to accumulate damage. The relationship between damage and the total DPSE is covered in the following sections, along with how failure criteria are created using them [23, 12].

2.4.1.7 G^R Failure Criterion

The G^R (pseudostrain energy release rate) technique, which assesses energy in a cumulative manner, was proposed by Sabouri and Kim [93, 94]. Under cyclic loading, the point of peak stress, which also happens to be the point that correlates to the highest pseudostrain, and maximum damage sustained at that cycle, is where the maximum stored pseudostrain energy at each cycle occurs. As a result, WR_{max} represents both the material's present damage state and energy storage capacity.

Two elements influence WRC: the decrease in C and the pseudostrain amplitude in tension ($\epsilon R 0, ta$). Therefore, WRC is regarded as an all-encompassing energy metric that uses the material itself as well as the external stress to quantify the energy wasted.

The results of a Vermont asphalt mixture's cyclic fatigue test with a 30% RAP under various loading scenarios are shown in Figure 2.15 and Figure 2.16

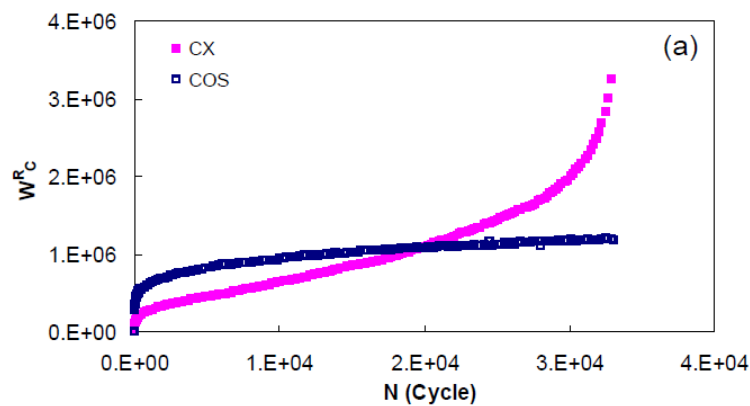
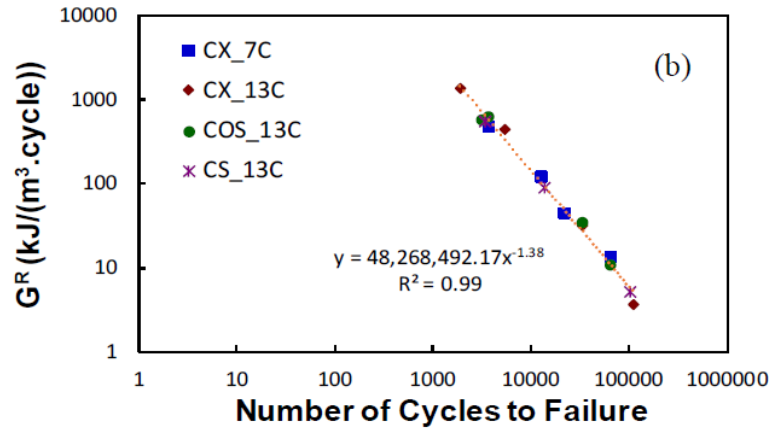


Figure 2.15: WRC versus N with strain and stress modes [23, 12]

In the case of the CX mode, more energy is released near the conclusion of the loading history when the material is quickly losing its structural integrity, according to a comparison of the WRC histories of the two displacement mode tests (the COS and CX tests produced almost the same N_f , as shown in figure 2.16. But more energy is produced at the start of the test since the specimen is made to endure greater COS from the start in the COS mode. Therefore, the optimal failure criterion ought to be able to quantify the impact of the whole loading history. Consequently, the new word, G^R , denotes the average rate of change. released pseudostrain energy (per cycle) across the test's whole duration, as calculated in equation 2.39. The G^R is still dependent on the rate of change of the pseudostrain energy, but it additionally encapsulates the loading history's influence [93, 94] .

Figure 2.16: G^R versus N_f for (CS COS and CX modes) [23, 12]

$$G^R = \frac{\int_0^{N_f} W_C^R dN}{N_f} = \frac{\int_0^{N_f} \frac{1}{2}(1 - C) (\varepsilon_{0,ta}^R)^2 dN}{N_f} \quad (2.39)$$

The research team hypothesizes that there must be a correlation between G^R and the final fatigue life N_f because the G^R approach characterizes the overall rate of damage accumulation during fatigue testing. The faster the damage accumulates (i.e., reaches a higher W_C^R during fewer numbers of cycles, the faster the material fails.

N_f was phenomenologically examined in several loading modes and at various load amplitudes; in each test, the G^R showed a strong connection with the power function. Figure 2.16 and equation 2.40 both display this relationship [67]. The loading mode is indicated by the first part of the name in the legend of figure 2.16, and the test temperature in degrees Celsius is indicated by the second portion

$$G^R = \gamma N_f^\delta \quad (2.40)$$

One of the main disadvantages of G^R failure criterion by kim et al, is that the number of load cycles and the G^R values in the field were above the laboratory testing range depicted in figure 2.16 because actual pavements may have millions of load cycles over the course of their lifetime [12].

2.4.1.8 D^R Failure Criterion

The goal of the study team's newly suggested failure criterion by Kim et al, which is discussed in this part, is to reduce the sensitivity of extrapolation without weakening the G^R failure criterion shown in equation 2.41 [12].

$$\int_0^{N_f} \frac{1}{2}(1 - C) (\varepsilon_{0,ta}^R)^2 dN = \gamma N_f^\lambda \quad (2.41)$$

Where:

$$\lambda = \delta + 2.$$

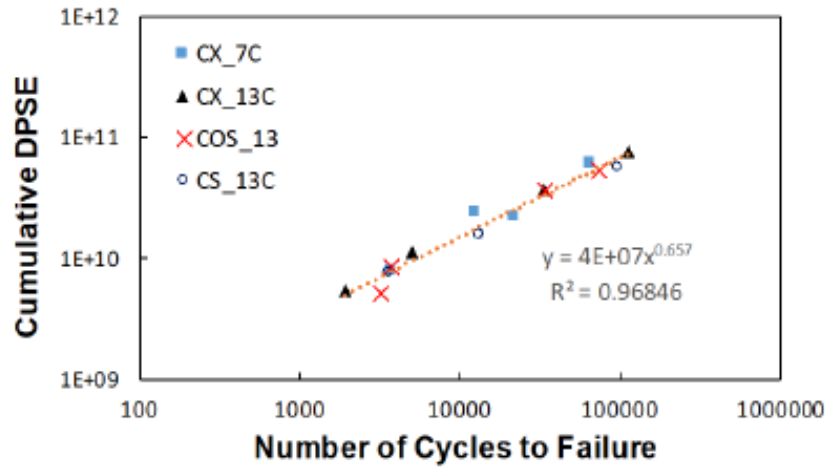


Figure 2.17: WRC vs N_f on log-log scale at different test modes and temperature [12]

The connection indicated in equation 2.41 is represented in a log-log scale in 2.17 using the same data that are given in figure 2.16 for various loading types and temperatures. Given varying loading modes and temperatures, the power relationship in equation 2.41 appears to be true, as indicated by the high R^2 value (about 0.97).

The fatigue life of asphalt mixes is exclusively defined by the cumulative DPSE up to failure, independent of loading method and temperature, as demonstrated by equation 2.42 and its verification in figure 2.18. The damage characteristic connection, shown in equation 2.35, is another material function that the study team discovered to be independent of temperature and loading mode.

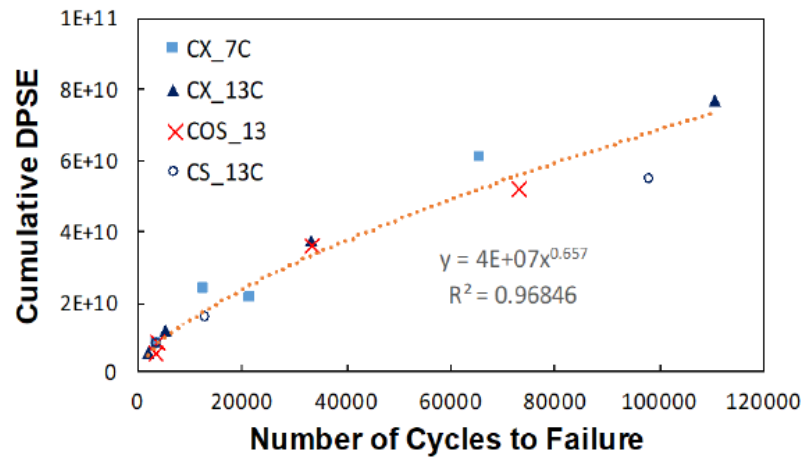


Figure 2.18: WRC vs N_f on arithmetic scale at different test modes and temperature [12]

While the connection illustrated in equation 2.41 indicates when fatigue failure occurs, the damage characteristic relationship in equation 2.35 describes how the damage increases when the material is loaded. The study team investigated this since the (1-C) word appeared in both correlations. Plotting of the same data as in figure 2.18 was done between the number of loading cycles and cumulative (1-C), which is shown in the figure 2.19 as "Sum (1-C)". Figure 2.19 illustrates how, under various loading and temperature conditions, the trends between the cumulative (1-C) and the number of loading cycles are comparable.

The goal of this analysis was to find a failure criteria that was independent of temperature, loading mode, stress

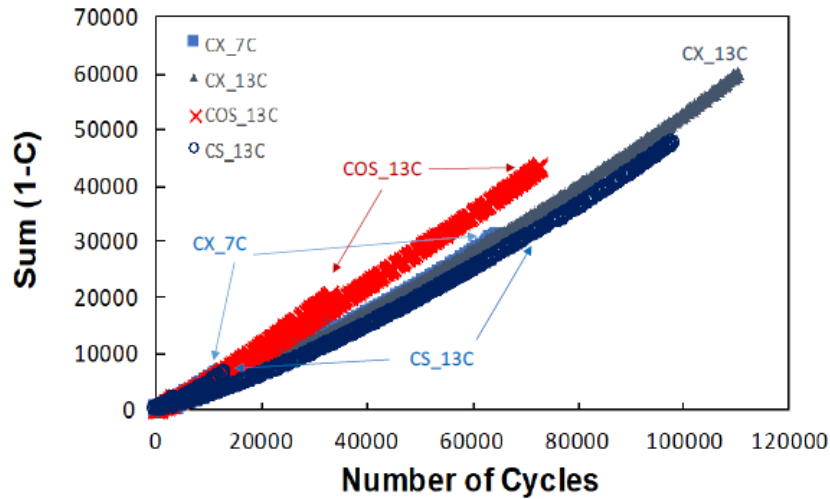


Figure 2.19: Cumulative (1-C) relation with number of cycles at different test modes and temperature [12] and strain amplitudes, and thus the research team only used the data at failure to plot the data in figure 2.20 . Each point in the map denotes a fatigue test that was conducted at a particular temperature, loading method, and beginning strain level.

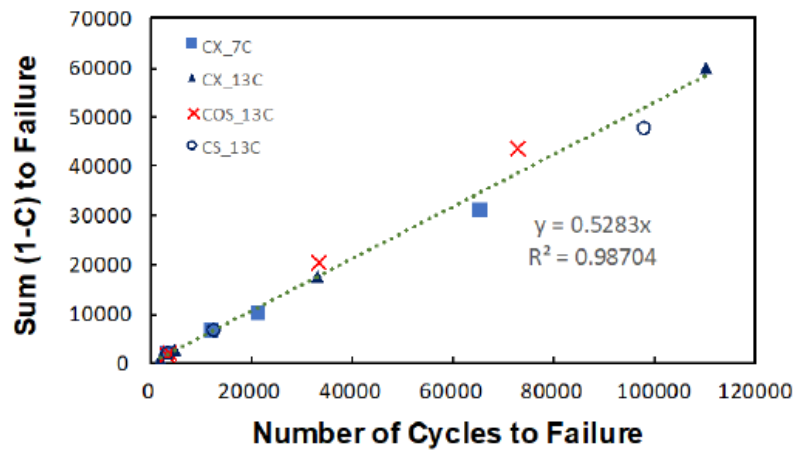


Figure 2.20: Cumulative (1-C) relation with number of cycles at different test modes and temperature [12]

Three key observations were made by the study team from figure 2.20. First, there is a strong correlation between the N_f values and the Sum (1-C) values under various loading types and temperatures. Secondly, at an arithmetic scale, the relationship between Sum (1-C) and N_f is linear. This fact reduces the sensitivity that was present in the G^R criterion in log-log scale due to experimental variability [12]. Thirdly, there are fewer fitting coefficients in the linear regression i-e, one for the slope and one for the intercept, because the link between Sum (1-C) and N_f goes via the origin. With the same number of test samples, this decrease in fitting coefficients might lead to a more accurate description of the failure threshold for a particular combination [12].

The slope in the connection between sum (1-C) and N_f is the material attribute that characterizes the failure of the material, according to the linear relationship between the two that passes through the origin. Equation 2.42

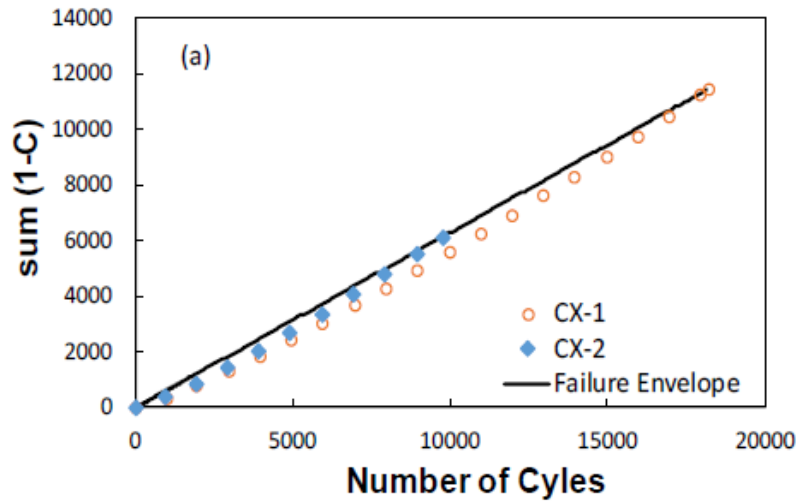


Figure 2.21: Definition of failure in cyclic fatigue test [12]

defines this slope, which is represented by the symbol D^R

$$D^R = \frac{\int_0^{N_f} (1 - C) dN}{N_f} \quad (2.42)$$

where the average C decrease up to failure is represented by D^R . The process of defining the failure in the cycle fatigue test data using the D^R failure criterion, which is represented by the Sum $(1-C)$ against N_f line, is illustrated in Figure 2.21, the sum $(1-C)$ increases as cyclic loading is applied to a specimen in research, with two distinct CX displacement amplitudes shown by the points CX-1 and CX-2. N_f is defined as the load cycle at which the slope D^R -defined point meets the Sum $(1-C)$ versus N_f line.

2.4.2 Permanent deformation models

One major form of pavement damage is permanent deformation. The [shift](#) model, which describes the asphalt mixture's persistent deformation behavior under traffic stress, is introduced in this section.

Roadway rutting, or permanent deformation, is a major cause of traffic accidents, particularly during wet or snowy weather. Rain creates hazardous driving conditions including hydroplaning and significantly reduces sight due to water spray. Similar issues arise from snow because snow and ice buildup in the wheel path's ruts. There are two types of models that may be used to forecast the permanent deformation of asphalt pavements: Power-law-type models and mechanistic visco plastic models [95, 96, 97]. These models can predict the quantity and pace of rutting.

Classical plasticity serves as the foundation for mechanistic viscoplastic models. To forecast the three-dimensional behavior of asphalt concrete under compression, Gibson and Desai et al, used a Perzyna-type flow rule in conjunction with hierarchical single-surface yield criteria [96, 95]. But their models weren't restricted in their capacity to replicate rate-dependent properties of asphalt mixes, such as rate-dependent softening [96, 97]. Darabi et al. established the notion of a memory surface to address the limitations associated with rate-dependent

behavior[98]. Nevertheless, despite asphalt concrete's sensitivity to temperature, this model was unable to capture its impacts. To understand the time-dependent viscoplastic behavior of asphalt concrete, Yun and Subramanian used viscoelastic notions like the convolution integral[99].

Even though these models can predict permanent strain in various loading conditions like stress, load time, and/or temperature, and may be able to shed light on the three-dimensional performance of a pavement structure through numerical implementation, the concepts developed by Yun and Subramanian are difficult for practicing engineers to fully and simply apply [99, 100].

Viscoelastic principles also come with a hefty computational and calibration cost. As a result, even when mechanistic visco plastic models provide accurate predictions, agencies have not generally adopted them.

2.4.3 Power law type model

Models of the power law kind are comparatively easy to implement and understand. A strain ratio model from the [MEPDG](#) serves as the representative model and is displayed in equation 2.43.

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_1 T^{\beta_2} N^{\beta_3} \quad (2.43)$$

ε_p = constant strain,

ε_r = resilient strain,

N = quantity of cycles,

$\beta_1, \beta_2, \beta_3$ = regression parameters.

The effects of stress and temperature on permanent strain are described by this model. The stress determined using the theory of resilience and the resilient strain derived from the dynamic modulus by Von Quintus and colleagues have shown that this assumption leads to a rise in inaccuracy [96, 95]. In other words, while the dynamic modulus has significance, it is not a reliable indicator of the plastic deformation properties across a wide variety of mixes and climates [101] Furthermore, since equation 2.43 functional form only applies to the behavior of the secondary region, the model is unable to explain the behavior of the primary region.

2.4.4 Incremental Model

Choi et al. expressed the incremental model, written as equation 2.44, to characterize the permanent deformation behavior of asphalt concrete. [102]

$$\varepsilon_{vp} = \frac{A_0 + \varepsilon_0 \cdot N}{(N_I + N)^\beta} \quad (2.44)$$

In this stage of development, $1-\beta$ represents the slope of the log-log connection between the permanent strain and the number of loading cycles. The major region's persistent strain behavior is determined by A_0 and N_I . The model coefficients ε_0 and β govern the growth of the secondary region. A_0 is the original, unchangeable strain. N_I is the number of cycles in which the main region changes into the secondary region.

The attributes of the incremental model by Choi et al. are summed up as follows [102]:

- How asphalt concrete behaves permanently under large viscoplastic strain
- Supports the theory of Time to Temperature Superposition Principle (t-TS).
- Rather of utilizing load time and temperature independently, the decreased load time analyzes the behavior of asphalt materials using the t-TS concept.
- The slope of the secondary region in the log-log scale is constant regardless of the reduced load time and deviatoric stress; thus, β is a material constant.
- The regression coefficients, A_0 , ε_0 , and N_I , change according to reduced load time (i.e., as a function of temperature and load time) and deviatoric stress.

The slopes of the secondary regions for the two mixtures varied at low temperatures, which is one of the main drawbacks of the incremental model. Nevertheless, since most of the deformation happened at higher temperatures, the constant slope assumption did not significantly alter the asphalt pavement's prediction of permanent strain [102].

2.4.4.1 Shift Model

Using a single constant slope, the permanent strain curves of the different decreasing load durations and deviatoric stress levels may be shifted horizontally down the x-axis according to the number of loading cycles. If the secondary area acts as the focus point, then all the values of the permanent strain may be transferred to construct. In log-log space, there is just one strain development curve as their slopes are equal. The strain curve that follows the horizontal shift is referred to as the permanent strain master curve, and the amount of shifting is known as the shift factor. Applying the shift function does not produce the same dynamic modulus master curve and related predictions as this shifting strain technique does. The method used to derive the dynamic modulus master curve and associated predictions using the shift function is fundamentally identical to this shifting strain technique. Characterizing permanent deformation behavior with less experimental needs is made easier by the shift model technique. Because several researchers have previously demonstrated the applicability of the t-TS principle, the shift model and its experimental verification are based on the time-stress superposition Time to Stress Superposition Principle (t-SS) concept, which is described in the following section [103, 104, 105, 106]

2.4.4.2 t-SS Principle

The phenomenological foundation for moving the permanent strain curves was established by the incremental model characterization, and this idea is further supported by a brief examination of the literature about theoretical underpinnings. To comprehend the t-SS principle, it is necessary to grasp the t-TS concept. Thermorheologically Simple (TRS) materials have the property known as the t-TS principle. Researchers frequently use the t-TS principle to analyze the behavior of asphalt concrete, one of the TRS materials, while evaluating its viscoelastic behavior. Significant viscoplasticity and damage can also be addressed with the use of the t-TS concept. The theory behind these results is demonstrated by the single integral nonlinear viscoelastic constitutive equation by Schapery, which is given as equation 2.45, 2.46 and 2.47 [107].

$$\varepsilon(t) = g_0 D_0 \sigma(t) + g_1 \int_0^t D(\psi - \psi') \frac{dg_2 \sigma}{d\tau} d\tau \quad (2.45)$$

$$\psi \equiv \int_0^t \frac{dt'}{a_\sigma(\sigma(t'))} \quad (2.46)$$

$$\psi' \equiv \psi'(\tau) \equiv \int_0^\tau \frac{dt'}{a_\sigma(\sigma(t'))} \quad (2.47)$$

D = creep compliance,

D_0 = elastic component of the creep compliance,

ϕ' = a variable of integration,

t' = integration variable,

g_0, g_1, g_2 = material constants,

A_σ = stress function.

According to published research, viscoelastic materials fall within the t-SS concept. To reflect the nonlinearity or the stress-induced microstructural changes of the materials, in certain circumstances, vertical shifting i-e, changing the curve along the creep compliance axis is necessary [108]. In other circumstances, moving is simply necessary horizontally. Based on the behavior of the material as seen in carefully planned tests, the required shift procedure for a particular material may be determined. To summarize, the load-time shift function, the stress shift function, and the reference curve make up the shift factor model. The permanent strain growth curve derived from a single Triaxial Repeated Load Permanent Deformation (TRLPD) test conducted under reference loading conditions is known as the reference curve. The reference curve is acquired via a separate test to ascertain the datum line, as opposed to the creation of a dynamic modulus master curve. The permanent strain master curve, the

ultimate reference curve shape, is created by fitting the incremental model to the permanent strain. This permanent strain can be calculated using the 2.49 The reference curve is constructed by translating the strains obtained from the TRLPD tests horizontally for load-time shifting and again for stress shifting. As a result, the overall shift factor is the sum of these two shift factors and can be calculated using 2.48. The deviatoric stresses and permanent strains can be evaluated using equation 2.50 and equation 2.51

$$a_{\text{Total}} = a_{\xi_p} + a_{\sigma_d} \quad (2.48)$$

$$\varepsilon_{vp} = \frac{\varepsilon_0 \cdot N_{\text{red}}}{(N_I + N_{\text{red}})^\beta}, \quad (2.49)$$

$$a_{\xi_p} = p_1 \log(\xi_p) + p_2, \quad (2.50)$$

$$a_{\sigma_d} = d_1 \left(\frac{\sigma_v}{P_a} \right)^{d_2} + d_3. \quad (2.51)$$

ε_{vp} = viscoplastic strain,

p_1, p_2, d_1, d_2, d_3 = model coefficients,

P_a = atmospheric pressure at which the tension is normalized,

N = reduced number of cycles (given by $N \times 10^a$ total).

2.5 Pavement Monitoring

The practice of evaluating road surfaces (pavement) to make sure they continue to be reliable, sturdy, and affordable is known as "pavement monitoring." During this monitoring, pavements are examined for rutting, cracks, potholes, and surface wear, among other signs of deterioration [109]. To maximize repair schedules, extend pavement life, and detect possible problems early on, it is an essential component of road maintenance and management.

2.5.1 Types of Pavement Monitoring

To maintain road surfaces' longevity and safety, pavement monitoring entails evaluating its performance and state [109]. Pavement monitoring is accomplished by a variety of techniques and technologies, each with unique capabilities and methodologies. The primary types are as follows:

2.5.1.1 Visual Inspection

Manual surveys: A straightforward, old-fashioned method in which skilled workers visually examine the pavement to find cracks, potholes, ruts, and other surface problems [110]. Depending on the inspector's level of experience, this procedure can be subjective. However, this process is cheap and quick for repair. The main disadvantage is intuitive, labor-intensive, and unsuited for large-scale networks.

2.5.1.2 Automated detection

Laser Scanning and Imaging Systems: Automated systems that are fitted with infrared sensors, lasers, or high-speed cameras may take precise pictures of the pavement's surface [111]. To detect and measure surface distresses, these photos are processed utilizing algorithms.

2.5.1.3 3D laser scanners

These scanners accurately identify flaws like cracks and rutting in pavement by providing a three-dimensional profile of the surface [112]. It provides an excellent accuracy, reliable findings, and quick data collection over wide regions. While it is the expensive initial cost of technology and equipment.

2.5.1.4 Ground Penetrating Radar (GPR)

- GPR uses electromagnetic waves to detect subsurface conditions such as the thickness of pavement layers, moisture content, and the presence of voids or anomalies beneath the surface [113]. It is effective in identifying potential issues before they become visible.
- Pros : Non-destructive, provides data on subsurface conditions.
- Cons: Requires interpretation by experts, expensive.

2.5.1.5 Falling Weight Deflectometer (FWD)

- Falling Weight Deflectometer (FWD) is used to assess the structural integrity of pavements. The system drops a weight onto the pavement surface, and the resulting deflections are measured to estimate the stiffness of the underlying layers [114].
- Pros : Provides detailed structural data, effective for large-scale networks.
- Cons: Expensive and requires lane closures during testing.

2.5.1.6 Inertial Profiling Systems

- These systems measure pavement smoothness and surface irregularities using accelerometers and gyroscopes mounted on a vehicle. Data are collected at highway speeds, which minimizes traffic disruption [115].
- Pros : Fast data collection, suitable for high-speed monitoring.
- Cons: Limited to surface profiling.

2.5.1.7 Embedded Sensors and IoT-based Monitoring

- Strain Gauges and Accelerometers: Sensors embedded within the pavement structure measure strain, temperature, and vibration in real-time. Data is transmitted to a central system for monitoring pavement performance [116].
- Smart Pavements (IoT): Internet of Things (IoT) technology integrates multiple sensors in the pavement network to provide continuous, real-time monitoring [117].
- Pros : Continuous data collection, early warning of potential failures.
- Cons: High installation costs, complex maintenance.

2.5.1.8 Non-Destructive Testing (NDT) Techniques

- Ultrasound Testing: Measures the propagation of ultrasonic waves through the pavement layers to detect cracks and internal defects.
- Infrared Thermography: Detects variations in heat emission from the pavement surface, which can indicate underlying damage or moisture accumulation [118].
- Pros : Non-invasive, able to detect internal defects.
- Cons: Requires specialized equipment and expertise.

2.5.1.9 Current monitoring approach by the Italian road authorities

The laboratory testing methods for road pavements monitoring and quality assurance by Italian road authorities before, during, and after construction focus on evaluating material quality and structural integrity at various stages [2]. These methods include both non-destructive and destructive testing shown in figure 2.22, ensuring that the materials used, and the constructed layers meet the required standards for performance, durability, and safety [2]. The summary of laboratory testing is summarized in figure. Here it is to be noted that this section only explores the laboratory testing techniques by the Italian road authorities for the improvement of pavement monitoring. The functional performance and other NDT testing techniques for pavement monitoring used by the Italian road authorities are beyond the limitations of this study

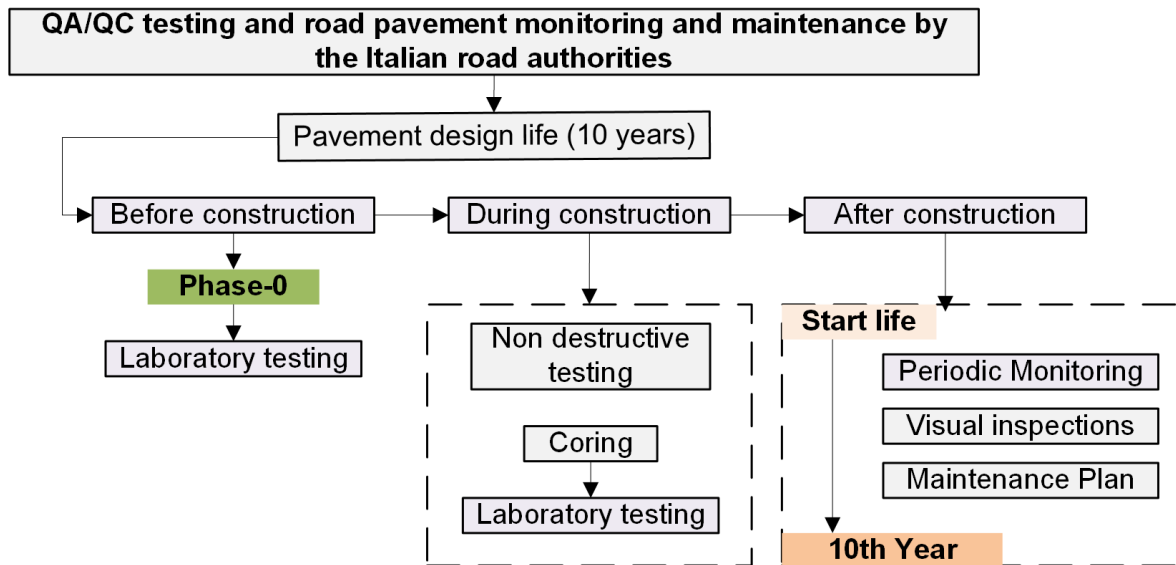


Figure 2.22: Pavement monitoring approach by the Italian road authorities, Source: author's explanation

2.5.1.10 Before road construction

Bitumen Quality Testing [119]:

- **Penetration Test (UNI EN 1426):** Measures the hardness and consistency of bitumen at room temperature.
- **Softening Point Test (UNI EN 1427):** Determines the temperature at which bitumen becomes semi-liquid.
- **Fraass Breaking Point Test (UNI EN 12593):** Establishes the low-temperature brittleness of bitumen.

Aggregate Analysis:

- **Grain Size Distribution (UNI EN 933/1-2):** Assesses the particle size distribution of aggregates to ensure they meet specification requirements.

Mixture Design:

- **Gyratory Press Compaction (UNI EN 12697-31):** Compacts specimens to evaluate voids and mechanical resistance.

2.5.1.11 During road construction [2]

1. Temperature Control:

- Temperature is critical during the laying process of bituminous mixtures. It is measured near the paving screed to ensure optimal compaction temperature.

2. Roller and Compaction Control:

- The number of compactions passes by the roller is monitored for uniformity across layers [2]

3. Non-Destructive Testing:

- **Dynamic Plate Load Test (LWD):** Measures the dynamic elastic modulus to assess soil compaction and strength.

2.5.1.12 After the road construction [2]

1. Core Sampling:

- Core samples (carrots) are extracted post-construction to analyze the thickness, density, and material compaction.

2. Skid Resistance Testing:

- **Pendulum Test:** Measures the surface's resistance to skidding, especially in wet conditions.

3. Indirect Tensile Strength (ITS) Testing:

- **ITS Test (UNI EN 12697-23):** Evaluates the mechanical resistance to indirect tensile stress at 25°C on cylindrical specimens made by gyratory compaction. This test is key in determining the road's load-bearing capacity and susceptibility to cracking.

4. Drainage Testing:

- **Draining Capacity:** Assessed using a water column permeameter, this test evaluates the porous asphalt's ability to drain water.

These tests ensure that the road pavements meet the durability, load bearing, and safety requirements through rigorous quality control.

2.6 Summary of the chapter

This chapter provided an extensive overview of the road pavements, especially focusing on the pavement monitoring. Different laboratory testing methods are explored to evaluate the principal distresses occurred in flexible pavements. Up till now the different damage modeling techniques and pavement design types and factors affecting the pavement design have been discussed. Consequently, the monitoring approaches by the Italian road authorities through the laboratory testing techniques before, during and after the road construction has been discussed in this chapter. As a major summary point of this chapter, and a background study as a hypothetical approach, this study summarized that currently laboratory testing approaches by the Italian road authorities are unable to characterize the conventional

and alternative paving materials. To proceed with the research methodology this study proposed two methods of laboratory testing based on basic and advanced level to support its hypothesis and provide the monitoring tools for the improvement of road pavements. This study also suggested the future development of the pavement management system for the Italian roads by utilizing this laboratory tested data base for the more sustainable road pavements.

Chapter 3

Methodology for the improvement of QA/QC by ANAS pavement monitoring in Italy

3.1 Introduction

This chapter provides a detailed methodology about the improvement of the pavement monitoring through the proposed laboratory testing approaches based on damage-related characterizations. Initially, the laboratory testing approach by the Italian authorities for QA/QC testing including service life is considered for the discrimination of the conventional and APMs. Keeping in view a scientific approach towards the improvement of pavement monitoring as shown in figure 1.3 in chapter 1 of this study. As stated in research question that "Are the current pavement monitoring approaches by the Italian road authorities are able to discriminate between the conventional and APMs"? The level of research already carried out and background study discussed in chapter 2 comes to a hypothesis that the basic and advanced level testing based on damage-related characterization are able to discriminate between the conventional and APMs?" Please note that the basic level approach is based on IDT testing, while advanced-level approach is based AMPT testing and SVECD approach. The whole summary of this study has been summarized in Figure 3.1 Finally, the main outcomes of both these testing approaches are proposed to be used as a monitoring tools for more sustainable road pavements.

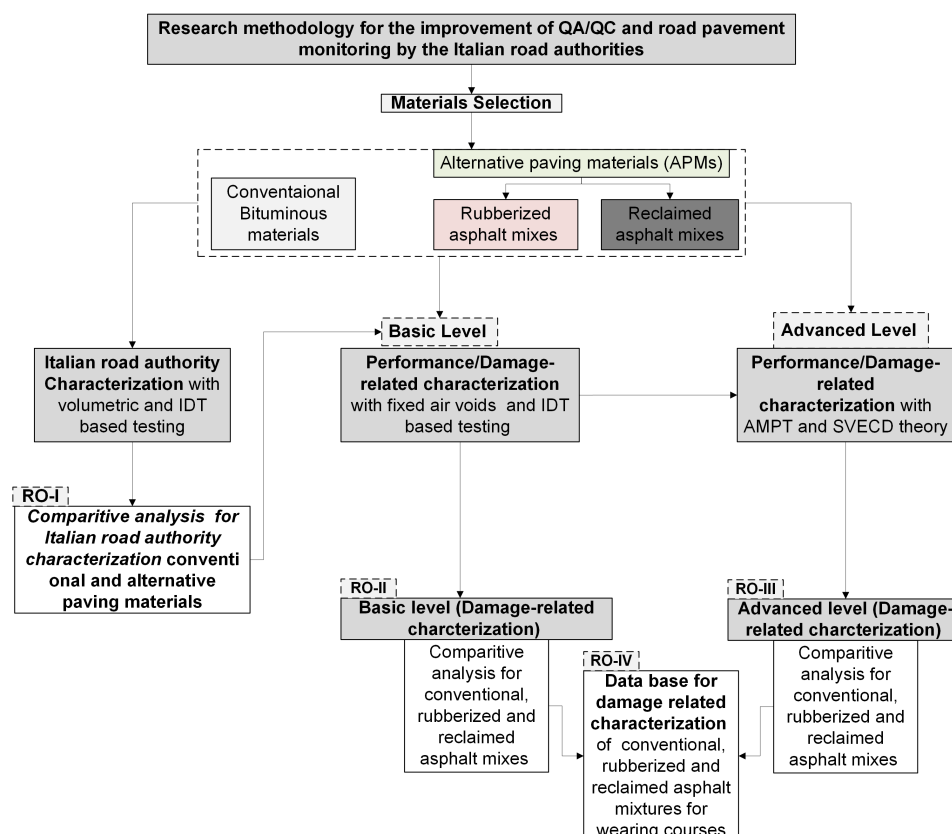


Figure 3.1: Summary of the research methodology

3.1.1 Materials

To proceed with the selection of materials, this research study has selected six different mixtures using Italian Gradation Specifications for Wearing Surfaces ([USURA B](#)) gradation and 50/70 penetration grade binder [ANAS](#) [2]. These mixtures include conventional, rubberized, and [RAP](#) mixtures. The conventional mixtures are coded as AC10 (control), and the mixtures containing Engineered Crumb Rubber ([ECR](#)) by dry process were coded as AC8-Rd. The addition of polymers along with rubberized particles (hybrid) mixture is coded as AC10-Rh. The mixture obtained from the addition of 50% [RAP](#) with conventional material is coded as AC10-RA50. The combination of rubber with 50% [RAP](#) is coded as AC10-RdRA50, while the combination of hybrid material and 50% [RAP](#) is coded as AC10-RhRA50. Please note that from this section all the mixtures will be written by their respective codes. The gradation characteristics of all six mixtures have been summarized in Figure 3.2. The upper and lower limits are obtained from [USURA B](#), the Italian road specifications for wearing course [2]. As can be seen in Figure 3.2, the six mixtures are within the upper and lower limits. The further details of each mixture are provided in the sub-section of this chapter.

3.1.1.1 Conventional asphalt mixture AC10 (control)

The AC-10 (control) mixture was obtained using 50/70 grade bitumen and [USURA B](#) gradation characteristics for wearing surfaces [2]. The nominal maximum size of the aggregate was 10 mm. It is designed for use on both low

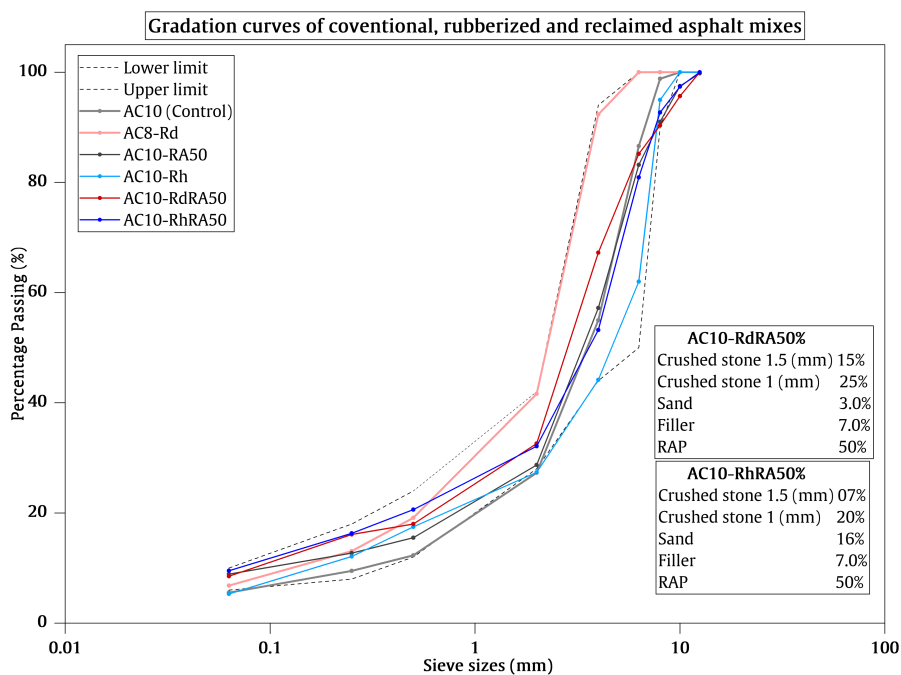


Figure 3.2: Gradation analysis of conventional and alternative paving mixtures and medium traffic roads, following the guidelines of the Italian Road Agency.

3.1.1.2 Rubberized dry mixture (AC8-Rd)

AC8-Rd was produced by using 50/70 bitumen and incorporating [ECR](#) through a dry process where the rubber was added to the plant mixer as an additional aggregate [120, 121]. The [ECR](#) is an innovative rubber chemically treated to prevent swelling during laying and compaction operations. This mixture is specifically designed for urban roads with low traffic and features a finer grading distribution.

3.1.1.3 Rubberized hybrid mixture (AC10-Rh)

The AC10-Rh was produced using a polymer modified bitumen along with the addition of [ECR](#) by dry process [122]. This mixture is specifically designed for urban roads with heavy traffic and features a finer grading distribution.

3.1.1.4 Reclaimed asphalt mixture (AC10-RA50)

This AC10-RA50 was produced using 50/70 grade bitumen [123]. This mixture is a combination of 50% conventional mixture and 50% Reclaimed asphalt mixture [123].

3.1.1.5 Rubberized dry with 50% reclaimed asphalt mixture (AC10-RdRA50)

AC10-RdRA50 created using 50/70 bitumen blended with 50% virgin aggregates and 50% obtained from the AC8-Rd mixture plus the addition of a rejuvenating additive [124].

3.1.1.6 Rubberized hybrid with 50% RAP mixture (AC10-RhRA50)

AC10-RhRA50 created using 50/70 bitumen blended with 50% virgin aggregates and 50% obtained from the AC8-Rh mixture plus the addition of a rejuvenating additive [125].

The summary for the addition of APMs containing rubberized and RAP mixtures is summarized in table 3.1 respectively.

Table 3.1: Summary of the mixtures utilized in the research study

Mix ID	Bitumen	Bitumen	Bitumen	Rubber	RAP (%)
		Content (%)	Modifier (%)	Content (%)*	
AC10 (control)	50/70	4.5	-	-	-
AC8-Rd	50/70	6.2	-	0.6	-
AC10-Rh	50/70	6.9	SBS	0.9	-
AC10-RA50	50/70	5.0	-	-	50% AC10
AC10-RdRA50	50/70	4.8	-	-	50% AC8-Rd
AC10-RhRA50	50/70	4.5	-	-	50% AC10-Rd

* By mixture weight

3.2 Experimental program

The experimental program of this research study comprises of laboratory-based selection and preparation of six different mixtures including conventional, rubberized and reclaimed asphalt mixtures. Before mechanical testing, two types of air void studies were carried out, i.e. energy compaction for ANAS and the target air voids study for basic and advanced level testing [8, 2]. To proceed with the experimental program and to ensure the QA/QC by the ANAS characterization was carried out. This characterization comprises of two parts, the volumetric study of the conventional and alternative paving mixtures and mechanical testing that is based on IDT testing. Furthermore, to support hypothesis of this research study, the basic level testing includes HT-IDT testing for rutting, Ideal-CT testing for fatigue cracking and ITSr testing for moisture sensitivity. In advanced level damage-related characterization based on SVECD theory, the conventional and APMs are undergone through stiffness evaluation using dynamic modulus testing, fatigue cracking evaluation using cyclic fatigue test and permanent deformation evaluation using SSR testing. Advanced-level characterization was further extended for FlexMATTM cracking and FlexMATTM rutting, and FlexPAVETM. The entire summary of the experimental program, damage modeling along with pavement design and performance analysis has been summarized in figure 3.3.

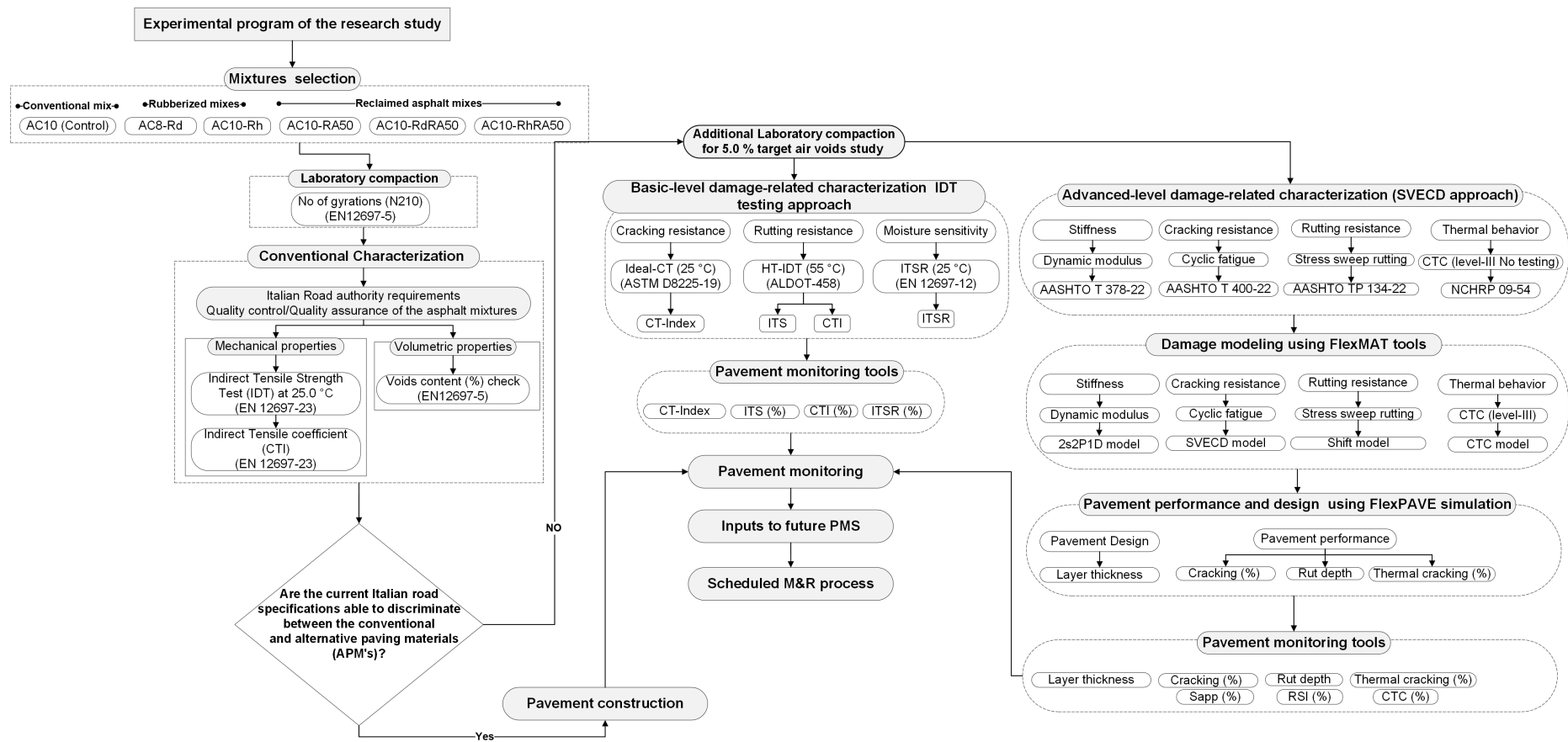


Figure 3.3: Experimental program of the research study

3.2.1 Specimen preparation

The mixture compositions are summarized in Table 3.1. The specimens were produced using a gyratory compaction method, applying a vertical pressure of 600 kPa [126]. The compaction process involved a gyration speed of 30.0 rpm, an external gyration angle set at 1.25°, and utilized a mold with a diameter of 150 mm. Two compaction methods were adopted, based respectively on compaction energy and target air voids study. The first method involved energy compaction to check the air void content of the mixtures and to ensure compliance with the Italian road authority ANAS [2]. The air void contents were determined using the hydrostatic process according to UNI EN 12697-5 [126]. After successful compliance of the mixes, the second compaction method was carried out by targeting 5.0% air voids for performance-based characterization, using a height of 180.0 mm as required for advanced laboratory testing [126]. For the conventional and basic level characterization that includes IDT testing, the dimensions of specimen were 50 mm (height) X 100 mm (diameter). After achieving the targeted mass for 5.0% air voids, the new superpave gyratory specimen were compacted have been cored and cut to obtain the dimensions necessary to carry out the basic and advanced laboratory tests. The details of coring has been discussed in section 3.2.3 of this chapter.

3.2.2 Coring and cutting of the specimen.

To conduct the conventional and basic level testing three repetitions on each mix were conducted. The 100x150 mm diameter samples with 200 mm thickness were prepared as shown in Figure 3.4(a) and (b). For basic level testing 100x50 mm cores were taken out from the main compacted sample as shown in Figure 3.4 (a). While for advance level characterization, 100x150 mm diameter samples were cored for dynamic modulus testing and stress sweep rutting test (SSR) by following the specifications of (AASHTO TP 134-19) [67]. Here it is to note that for advanced level testing the mixtures were short termed aged for four hours at 135 °C. For cyclic fatigue

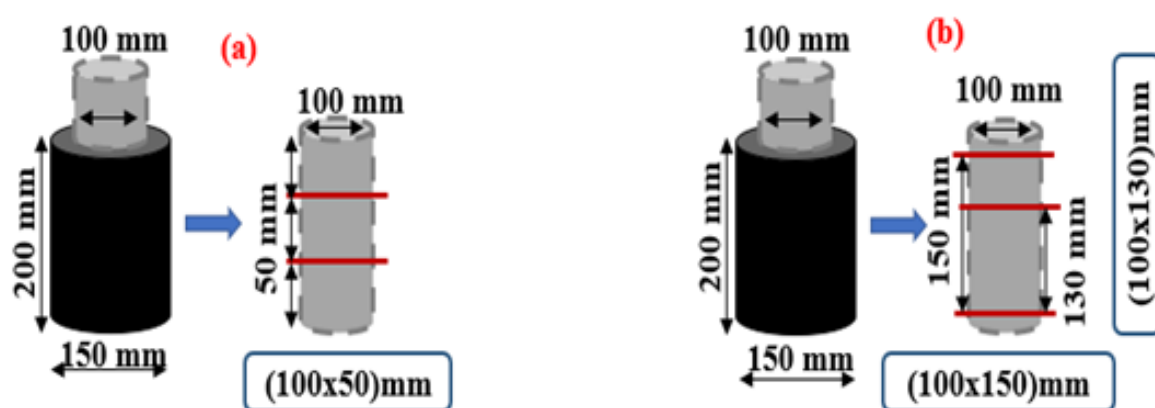


Figure 3.4: Specimen cutting for basic level 3(a) Cutting for advanced level 3(b)

test 100X130 mm cores were taken out in accordance with the specifications of AASHTO TP 133 as shown in figure 3.4(b). The same type of super pave gyratory compacted mixture was prepared for both the basic and advanced-level characterization of conventional and APMs.

3.3 QA/QC by the Italian road authorities

3.3.1 Volumetric Characterization

To investigate the volumetric requirements by the Italian road authorities, the specimens were compacted until 210 gyrations using energy compaction technique [2]. The requirements for Italian road authorities are that the voids of the specimen, determined according to EN 12697-8, must be comprised between 11 and 15 % at 10 gyrations, between 3 % to 6 % at 120 gyrations and higher than 2 % at 210 gyrations. The Air voids contents with BS EN 12697-8:2003 standard was obtained by calculating the bulk density (BS EN 12697-6:2012 Procedure C) and the maximum density of a bituminous (BS EN 12697-5:2018) [127]. The description of the for void content control has been summarized in Table 3.2. Table 3.2.

Table 3.2: Thresholds for QA/QC by the Italian road authorities [2]

<i>Characterisation</i>	<i>Property</i>	<i>Benchmark values</i>	<i>Standards/ Specifications</i>	<i>Reference (s)</i>
Conventional QA/QC-Italian road authorities)	Void Content check	No of Gyrations Void content (%) N10: 11-15 N120: 3-6 N210: ≥ 2	UNI EN 12697-5/ANAS	[1]
	Indirect Tensile Strength at 25.0 °C	ITS (MPa): 0.72-1.60 ITS (MPa): ≥ 0.6	UNI EN 12697-23/ANAS	
	Indirect Tensile Coefficient at 25.0 °C	CTI (MPa): ≥ 65 CTI (MPa): 50-150	UNI EN 12697-23/ANAS	

3.3.2 Conventional IDT Testing

The conventional characterization involves the examination of IDT. The IDT test, was carried out using UNI EN 12697-23 at 25.0°C, three trials were conducted for each examined mixture [128]. At this temperature, there was no need to modify the specimens for further conditioning. Refer to the Table 3.2, ANAS Italian road specifications for quality as-assurance and quality control QA/QC recommended minimum tensile strength between 0.72 to 1.60 MPa for asphalt mixtures. While ANAS CSA ACUSTICA QA/QC specification recommended a minimum range of greater than or equal to 0.6 MPa. The peak load at the specimen's fracture and its dimensions are used to compute the ITS, as indicated by equation 3.1. The formula for ITS is expressed as:

$$ITS = \frac{2P}{\pi DH} \quad (3.1)$$

Here, P stands for the peak load, H denotes the specimen's height, and D indicates the diameter of the specimen.

The **CTI** of both the mixtures were calculated by the formula in equation 3.2 [2]. The minimum recommended range by the **ANAS** for the **CTI**, is greater than or equal to 65 MPa while for the **ANAS** CSA ACUSTICA, the minimum recommended range is between 50 to 150 MPa.

$$CTI = \frac{\pi ITS D}{2 \delta V} \quad (3.2)$$

Here in equation 3.2, δV represents the vertical displacement of the specimen recorded at the point of failure. Where "D" is the diameter of the specimen.

3.4 Proposed methodology to improve pavement monitoring through laboratory testings

The general idea of the proposed methodology has already been discussed at the start of this chapter. This section elaborates on the proposed basic and advanced approaches based on laboratory testing methodologies in detail to discriminate between conventional and **APMs** and for the improvement of pavement monitoring for more sustainable road pavements.

3.4.1 Basic-level damage-related approach to improve pavement monitoring

Three major types of principal distresses have been addressed through the basic approach, including fatigue cracking, rutting, and moisture damage. The schematic representation of the basic level approach is summarized in figure 3.5. To provide the additional cracking, rutting, and moisture damage characterization, the below sections describe the testing methodologies.

3.4.1.1 **Ideal-CT** for additional cracking characterization

In order to proceed with the cracking resistance test, the conventional and alternative paving materials were tested according to EN 12697-23 and ASTM D8225-19. The **Ideal-CT** is a fracture test developed by Zhou et al used to find out the cracking resistance of the asphalt mixtures[24]. The specimen size is 150 mm diameter by 50 mm height, and the test temperature is 25°C. The test is easy to perform and rapid, as the loading rate is 50 mm/min. The CT-index parameter is related to field performance and can be calculated from the applied load and corresponding displacement [24]. Furthermore, from the area under the force-displacement curve, fracture energy (Gf) can be determined as shown in figure 3.6. Through a series of simplification, it is possible to see that cracking grown rate is highly related to the fracture energy, the post-peak slope at 75% maximum load (m75), the displacement corresponding to 75% of the maximum measured load (l75), and the specimen diameter. Equation 3.3 shows the

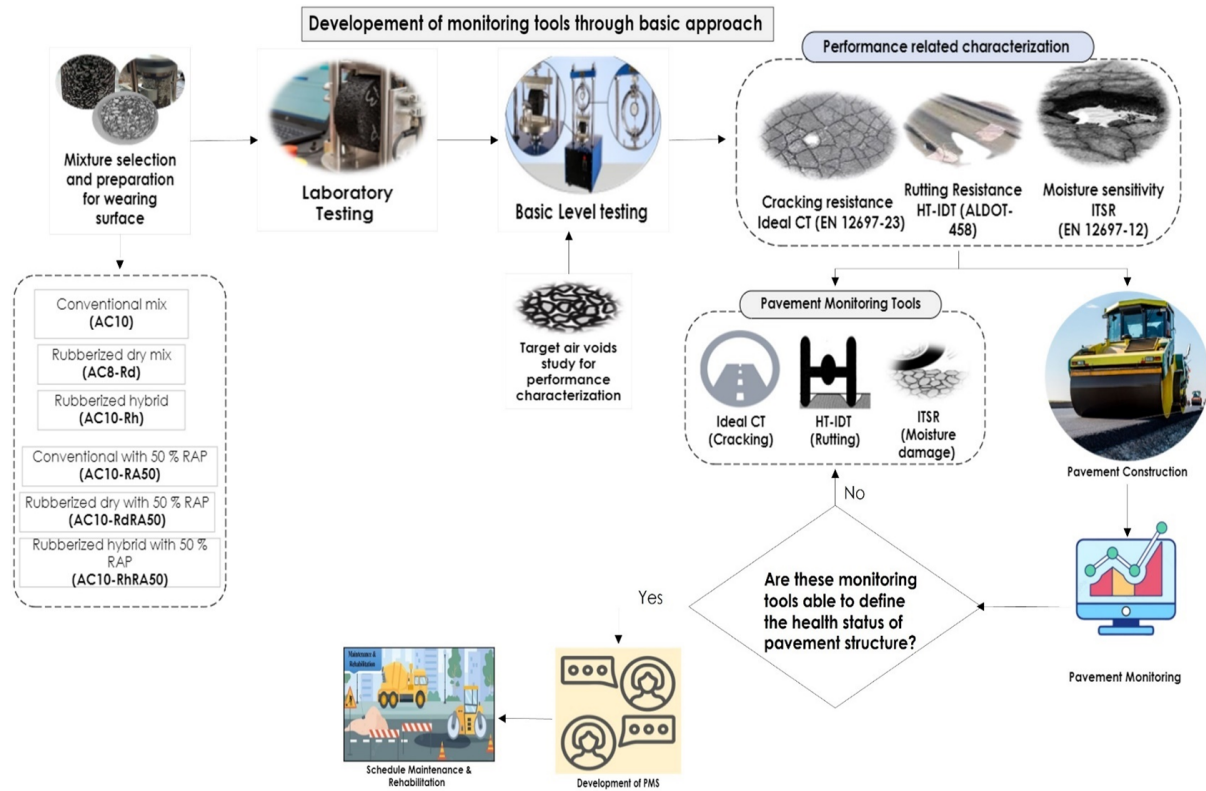


Figure 3.5: Improvement of pavement monitoring through basic-level damage-related approach
equation used to calculate the CT index

$$CT_{\text{index}} = \frac{t}{62} \times \frac{G_f}{|m_{75}|} \times \left(\frac{I_{75}}{D} \right) \quad (3.3)$$

Table 3.3, summarizes the threshold ranges for the cracking resistance, as it is observed from the literature that **Ideal-CT** is sensitive to **RAP** content, change in binder type and gradation of the aggregates. The addition of the alternative paving materials like **ECR**, polymers and hybrid materials enhanced the cracking resistance of the asphalt mixtures.

Table 3.3: Threshold values for additional cracking characterization [7]

Property	Standard(s)	Parameter	Testing conditions	Performance evaluation
Cracking resistance	ASTM D8225-19	IDEAL-CT CT_{index}	Temperature 25°C Vertical monotonic load 50 mm/min	$CT \text{ index} \geq 80$
	EN 12697-23	Indirect tensile strength - ITS		$ITS \geq 0.9 \text{ MPa}$

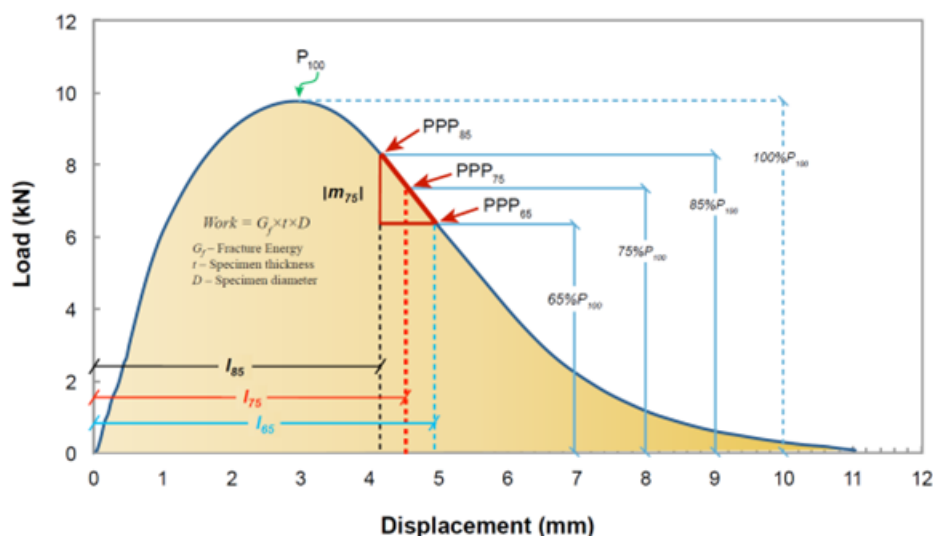


Figure 3.6: Fracture energy calculation for cracking resistance [24]

3.4.1.2 HT-IDT testing for additional rutting characterization

Similar to IDT tests conducted at 25.0 °C, the fundamental performance-related test was executed under elevated temperatures using a high-temperature HT-IDT test at 54.0 °C [62]. The specimens were conditioned at the testing temperature, set as 10.0 °C lower than the average temperature observed over a week, taking into account the highest pavement temperature. The HT-IDT test was conducted by following the specifications of ALDOT-458. Furthermore, the benchmark values for characterizing the rutting resistance are provided in Table 3.4. Mixtures having an equal or greater strength than 0.44 Mpa have an excellent rutting resistance while a mixture with a strength less than 0.20 MPa is considered a poorly rutting-resistant mixture [62]. Similar to the CTI obtained at 25 °C, the CTI of both the mixtures were calculated using the formula in equation 3.2 with a temperature of 54.0 °C.

Table 3.4: Threshold values for additional rutting characterization [8]

Characterisation Level	Standard	Parameter	Performance
Rutting resistance (Basic)	ALDOT-458HT-ITS (MPa)	> 0.44	Excellent
		$0.32 \leq 0.44$	Good
		$0.20 \leq 0.32$	Fair
		≤ 0.20	Poor

3.4.1.3 ITSR testing for moisture damage resistivity

Regarding the water sensitivity, it was determined according to the European standard EN 12697-12:2018 Method A. It is possible to investigate the loss of performance suffered by the different mixes due to water effect (moisture damage) [129]. A set of six cylindrical test specimen were divided into two equally sized subsets and conditioned: half of the specimens were conditioned in dry temperature at 25.0 °C, while the other half was saturated and stored in water at an elevated conditioning temperature of 40°C for 68 to 72 hours. At the end of the test, the sensitivity to water is determined by calculating the ITSR as a percentage ratio of the indirect tensile strength in wet and in

Table 3.5: Threshold values for moisture sensitivity [9]

Property	Standard(s)	Parameter	Testing conditions	Performance evaluation
Moisture sensitivity	EN 12697-12	Indirect tensile strength ratio - ITSR	Testing Temperature 25°C Conditioning in a water bath at (40 ± 1) °C for 72 hours	ITSR ≥ 80

dry conditions. The calculation of $ITSR$ (%) is shown in equation 3.4.

$$ITSR = \frac{ITS_w}{ITS_d} \times 100 \quad (3.4)$$

where ITS_w is the average indirect tensile strength of wet specimens and ITS_d is the average indirect tensile strength of dry specimens. The threshold values for the moisture damage resistivity have been summarized in table 3.5 respectively.

3.4.1.4 Summary of the basic-level approach

The basic laboratory testing approach, with additional damage-related characterization, provides additional monitoring tools like cracking resistance with **Ideal-CT**, rutting characterization with **HT-IDT** testing, and moisture damage sensitivity with testing $ITSR$. According to the authors point of view, this additional characteristic may help the Italian road authorities with the improved monitoring approach if the **APMs** are considered.

3.4.2 Advanced level approach to improve pavement monitoring

The advanced-level approach includes laboratory testing for stiffness, cracking, and rutting evaluation. The advanced methodology specifically includes **SVECD** theory for the damage-related characterization of conventional and **APMs**. The methodology was further extended using the **PASSFlex** system for damage modeling along with conventional and **APMs** to analyze the viscoelastoplastic characteristics of the asphalt mixtures. The analysis tool **FlexMAT** cracking and rutting was utilized to assess the 2s2p1D model for stiffness, the **SVECD** model for cyclic fatigue, and the shift model for rutting behavior. The procedure was further extended to analyze the pavement performance and response analysis along with the performance-based pavement design. The schematic representation of the advanced level approach for the improvement of the pavement monitoring is summarized in figure 3.7.

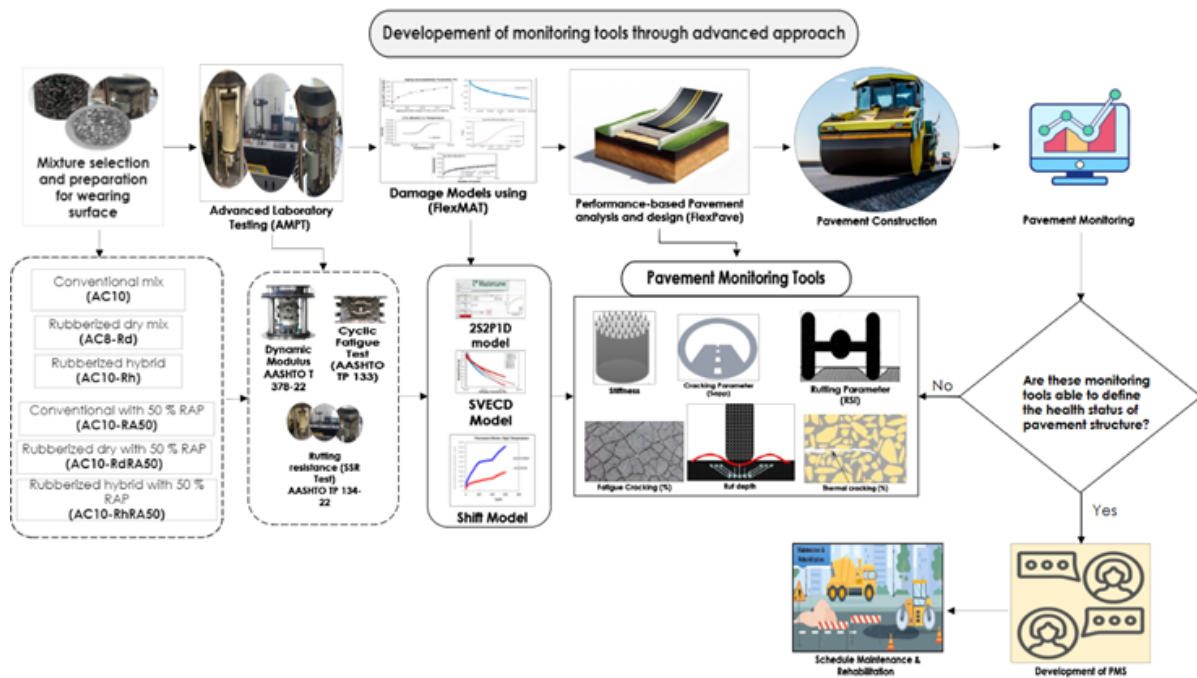


Figure 3.7: Improvement of pavement monitoring through advanced approach

3.4.2.1 Stiffness evaluation using dynamic modulus testing

The linear visco-elastic properties and the stiffness of the conventional and alternative asphalt mixtures were investigated using an asphalt mixture performance tester (AMPT) according to AASHTO T 378-22 [57, 130]. It can perform uniaxial tests of asphalt mix samples and is a hydraulically powered machine capable of testing bituminous conglomerate specimens by subjecting them to load cycles in a range of different temperatures and frequencies. (Asphalt Institute., 2014) [63]. To conduct the test three different temperatures (4, 20 and 40 °C) with three different frequencies of (0.1, 1.0 and 10 Hz) were considered on conventional and the alternative asphalt mixtures. The data quality statistics are summarized in Table 3.6 for 100x150 mm dimension specimen. These data quality indicators allow the analysis of the dynamic modulus testing. If these limits exceeds the user has to repeat the test to have quality assurance of the test data.

Table 3.6: Data quality Statistics for the dynamic modulus testing [10]

Indicator	Limit
Drift deformation	Applied load direction
Peak to peak strain	75 to 125 micro strains (unconfined tests)
Load standard error	10%
Deformation standard error	10%
Deformation uniformity	30%
Phase uniformity	3 degrees

For each mixture this dynamic modulus data was further utilized to generate master curve using 2s2P1D model for its stiffness evaluation and the same data was utilized for the SVECD model for fatigue cracking.

3.4.2.2 Cyclic fatigue test for fatigue cracking

The fatigue life of an asphalt mixture is essential for evaluating its long-term performance under real-world conditions. Cracks develop and propagate due to heavy traffic loads and harsh environmental factors, which has led researchers to focus on performance-based characterization methods [64]. To assess this, a cyclic fatigue test was carried out, involving three repetitions for both conventional and alternative asphalt mixtures. These specimens were tested at varying micro-strain levels according to the AASHTO TP-133 specifications. The testing temperature was taken according to the PG grade as per AASHTO TP-133 standards. The specimens had a geometry of 100 mm in diameter and 130 mm in height. For the conventional asphalt mixtures, the applied strain levels were 150 $\mu\epsilon$, 200 $\mu\epsilon$, and 250 $\mu\epsilon$. In comparison, the alternative asphalt mixtures were subjected to higher strain levels of 200 $\mu\epsilon$, 250 $\mu\epsilon$, and 300 $\mu\epsilon$ [64]. All tests were conducted at a controlled temperature of 20°C, following AASHTO TP 133 standards [131]. Once each specimen reached failure, data was collected and analyzed using the FlexMAT system. This data was crucial for developing a simplified viscoelastic continuum damage (SVECD) model, which helps in predicting the material's fatigue behavior. Additionally, the apparent surface capacity index (Sapp) for the Sicily region was calculated based on the test results [64]. To further evaluate the cracking performance, the percentage of cracks was measured for both conventional and alternative mixtures. This information was then analyzed using the FlexPAVETM simulation program, allowing for a detailed comparison of the cracking resistance of the two types of mixtures under similar conditions. This performance evaluation is key for understanding how each mixture type would behave under long-term service in real-world applications. In order to check the quality of cyclic fatigue data, Table 3.7 shows some limitations for finger print E^* which should be taken into the account while proceeding the cyclic fatigue test [11].

Table 3.7: Fingerprint test limitations [11]

Indicator	Suggested Limit
Applied stress error	$\leq 10\%$
Average standard error of the measured strains	$\leq 10\%$
Uniformity coefficient of the measured strains	$\leq 35\%$
Uniformity coefficient for the phase angle measurements	$\leq 3^\circ$

3.4.2.3 Stress sweep rutting test for permanent deformation

To study the rutting resistance of conventional and alternative asphalt mixtures the Stress Sweep Rutting (SSR) Test was conducted using an AMPT, in accordance with (AASHTO TP 79-19). Three cylindrical specimens of 100X150 mm diameter were tested for each mixture [67]. For Stress Sweep Rutting (SSR) test, two temperatures, high temperature (TH) at 54°C calculated by degree-days parameter equation in AASHTO TP-134-19 and low temperature (TL) 20°C based on the climatic performance grade (PG) were considered. For each temperature, a vertical load is applied for 600 cycles at three deviatoric stress levels of (200 loading block for each segment) [67, 15]. A confining stress of 69 KPa was applied with a loading pulse of 0.4 s and followed by a rest time of 3.6 s for TH and 1.6 s for TL. The SSR test allows to determine the shift modeling for permanent strain using FlexMATTM

rutting tool and the rut depth analysis using FlexPAVETM [132].

3.5 Damage models using advanced laboratory testing

The testing data from AMPT testing using dynamic modulus test, cyclic fatigue test and SSR test was further utilized for visco-elastoplastic continuum damage modelling. FlexMAT cracking and rutting was utilized to analyze the 2s2p1D, SVECD and Shift model. FlexMAT is an Excel-based analysis tool developed by the NC state university USA to conduct the damage modelling. The reason for the separate tools of cracking and rutting is that rutting requires a large data base, and it is difficult to arrange and analyze the cracking and rutting data in single excel file [133].

3.5.1 2S2P1D Model for stiffness analysis

The dynamic modulus data obtained from the AMPT was further analyzed by FlexMAT cracking tool, which can analyze the 2s2p1D model, SVCED model, CTC and AMAC model [134]. The 2s2p1D model presented by Olard and Di Benedetto in 2003 is summarized in equations 3.5 to 3.10 which is utilized by the FlexMAT cracking to analyze the dynamic modulus data [135]. The interface for the dynamic modulus analysis using FlexMAT carking tool has been shown in figure 3.8.

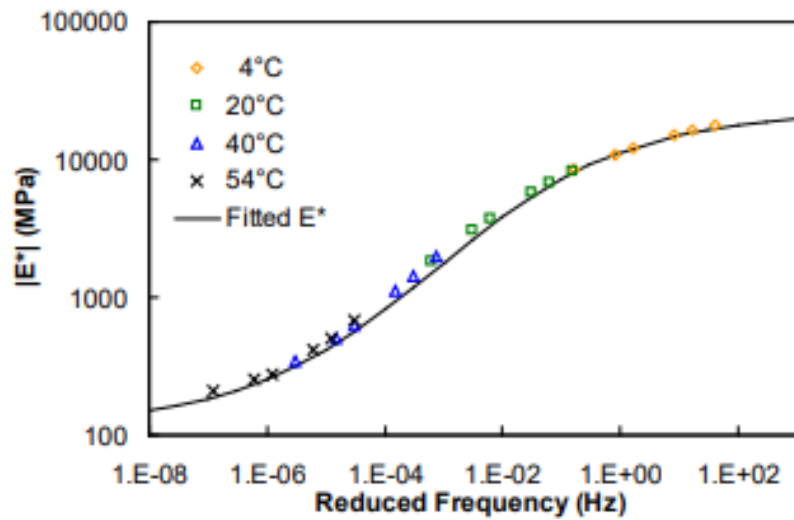


Figure 3.8: Dynamic Modulus Master curve [25]

$$E'_1 = (E_0 - E_{00}) \left[1 + \delta (2\pi f \tau_E)^{-\kappa} \cos\left(\frac{k\pi}{2}\right) + (2\pi f \tau_E)^{-h} \cos\left(\frac{h\pi}{2}\right) \right] \quad (3.5)$$

$$E'_2 = (E_0 - E_{00}) \left[\delta (2\pi f \tau_E)^{-\kappa} \sin\left(\frac{k\pi}{2}\right) + (2\pi f \tau_E)^{-h} \sin\left(\frac{h\pi}{2}\right) + (2\pi f \tau_E \beta)^{-1} \right] \quad (3.6)$$

$$DEN = \left(\frac{E'_1}{E_0 - E_{00}} \right)^2 + \left(\frac{E'_2}{E_0 - E_{00}} \right)^2 \quad (3.7)$$

$$\text{Re}(E^*) = E_{00} + \frac{E'_1}{DEN} \quad (3.8)$$

$$\text{Im}(E^*) = \frac{E'_2}{DEN} \quad (3.9)$$

$$|E^*| = \sqrt{[\text{Re}(E^*)]^2 + [\text{Im}(E^*)]^2} \quad (3.10)$$

Where,

E_{00} = minimum storage modulus value (kPa);

E_0 = maximum storage modulus value (kPa);

$\kappa, \delta, h, \delta E$ = fitting coefficients; and

f = loading frequency (Hz); and

E^* = dynamic modulus (kPa).

To adjust the coefficients of the 2S2P1D function, a specialized fitting approach was designed to address the issue of initial value dependency highlighted by Mangiafico et al. in 2019 [12, 136]. The process for fitting the 2S2P1D model in FlexMAT™ involves the following steps:

Step 1: Matching the experimental storage modulus data to a sigmoidal curve. This is achieved by reducing the squared logarithmic difference between the observed data (within the tested reduced frequency range) and the predicted modulus using the sigmoidal curve. The parameters in Equations 3.11 and 3.12 are fine-tuned in a single optimization process, with initial parameter estimates provided in Table 3.7.

$$\log(E'(f_R)) = b + \frac{\log(\max E') - b}{1 + e^{d+g \log(f_R)}} \quad (3.11)$$

$$f_R = f \cdot 10^{c_1 T^2 + (c_2 - 2c_1 T_{\text{ref}})T + (c_1 T_{\text{ref}}^2 - c_2 T_{\text{ref}})} \quad (3.12)$$

Where,

E' = storage modulus (kPa);

f_R = reduced frequency (Hz);

$\max E'$, b , d , g = sigmoidal function's fitting coefficients;

$c1$, $c2$ = fitting coefficients;

f = loading frequency (Hz);

T_{ref} = dynamic modulus' reference temperature ($^{\circ}\text{C}$); and

T = temperature ($^{\circ}\text{C}$).

Table 3.8: Starting Parameters for Sigmoidal-Experimental Data Fitting Process[12]

Fitting coefficient	Initial values
$\max(E')$	25×10^6
b	4
d	-1.5
g	-0.5
$C1$	0.005
$C2$	-0.15

Step 2: Selection of the reduced frequency range. This process is currently being reviewed, but for now, the method in place generates a vector of 21 values, spaced logarithmically across decades, ranging from 10^{-15} to 10^5 .

Step 3: Establishing starting values for 2S2P1D fitting. Prior to applying the experimental data to the 2S2P1D model, the initial parameters are obtained by aligning the 2S2P1D function with the sigmoidal curve determined in Step 1, across each frequency in the range from Step 2. The necessary starting points for minimizing the error can be found in Table 3.8, while the coefficient limitations are listed in Table 3.9.

Table 3.9: Starting Parameters for 2S2P1D-Sigmoidal Fitting Procedure[12]

Fitting coefficient	Initial values
δ	2.5
κ	0.1
h	0.5
β	10^{12}
E_{00}	$10^{(b_{\text{sigmoidal}} - 3)}$
E_0	-0.15
$\log(\epsilon)$	-3

Step 4: Determining the coefficients for the 2S2P1D model. In this final step, the actual experimental data collected during testing consistent with the conditions in Step 1 is used to compute the error against the 2S2P1D model. The error minimization algorithm utilizes the initial values derived from the fitting process in Step 3 for each coefficient, adhering to the constraints outlined in Table 3.10. As a result, the dynamic modulus master curve is established using the provided experimental data. The time-temperature shift-factor coefficients are calculated using Equations 3.13, 3.14, 3.15, and 3.16, with a reference temperature set at 21.1°C .

Table 3.10: Constraints for 2S2PID-Sigmoidal Fitting[12]

Fitting coefficient	Initial values
δ	≥ 0 and ≤ 20
κ	≥ 0 ≤ 1
h	$\leq h$ ≥ 0 ≤ 1
β	$= 10^{12}$
E00	$10^{(b_{\text{sigmoidal}} - 3)}$
E0	$= 40000$
$\log(\tau E)$	≥ -10 and ≤ 10

$$a_T = 10^{(a_1 T^2 + a_2 T + a_3)} \quad (3.13)$$

$$a_1 = c_1 \quad (3.14)$$

$$a_2 = c_2 - 2c_1 T_{\text{ref}} \quad (3.15)$$

$$a_3 = c_1 T_{\text{ref}}^2 - c_2 T_{\text{ref}} \quad (3.16)$$

Where,

aT = time-temperature superposition's shift-factor;

Tref = dynamic modulus' reference temperature (°C);

c1, c2 = fitting coefficients;

T = temperature (°C); and

a1, a2, a3 = shift-factor coefficients.

Finally, the parameter for the rate of damage evolution (α) is determined using the specified equation 3.17.

$$\alpha = \left(\frac{1}{m} \right) + 1 \quad (3.17)$$

α = damage evolution rate parameter; and

m = maximum slope of the relaxation modulus in log-log scale.

3.5.2 SVECD model using cyclic fatigue testing

The approach used for characterizing damage is the simplified viscoelastic continuum damage (S-VECD) model[12, 137]. The theoretical foundations of the S-VECD model is provided in chapter 2 of this research study. In order to proceed with FlexMAT cracking the outputs of cyclic fatigue testing were utilized to produce four main outputs of significance:

1. **Damage Characteristic Curve Characterization (C vs. S curve):** This illustrates the relationship between damage and the corresponding stress levels.
2. **Failure Criterion (DR)** This defines the conditions under which failure occurs within the material).
3. **Dissipated Pseudo Strain Energy Characterization:** This quantifies the energy lost in the system due to damage and deformation.
4. **Apparent Damage Capacity (Sapp):** This metric assesses the material's ability to sustain damage before reaching failure.

The 'Output Fatigue Screen' in FlexMAT cracking showcases three key plots:

1. **C versus S:** This plot displays the representative damage characteristic curve for the material, featuring individual curves and the fitted representative behavior for the tested sample.
2. **DPSE vs. Reduced Strain Rate:** This graph illustrates the dissipated pseudo-strain energy as a function of the reduced strain rate, considering four different conditions of initial integrity.
3. **Cumulative (1-C) versus Nf:** This plot depicts the failure criterion (DR)points for each specimen.

These plots, along with the cyclic fatigue characterization process, can be organized into the following steps:

Step 1: Calculation of Individual C vs. S Curves.

In this initial phase, the data for each specimen is examined independently to derive a C vs. S curve for every sample. This calculation adheres to the methodologies specified in the relevant standards (AASHTO TP 107-18, PP 99-19, and TP 133-19) [91]. The individual curves generated in this step will provide essential information for the calculations performed in the following steps, ensuring that the analysis is accurate and comprehensive.

Step 2: Calculation of the Representative C vs. S Curve.

To generate a sample representative C vs. S curve, Equation 3.18 is fitted using the individual C vs. S curves obtained in Step 1. This fitting process allows for the establishment of a collective curve that accurately represents the overall behavior of the material based on the individual specimen data.

$$C(S) = 1 - C_{11}S^{C_{12}} \quad (3.18)$$

Where,

C = integrity;

S = damage; and

C11, C12 = fitting coefficients

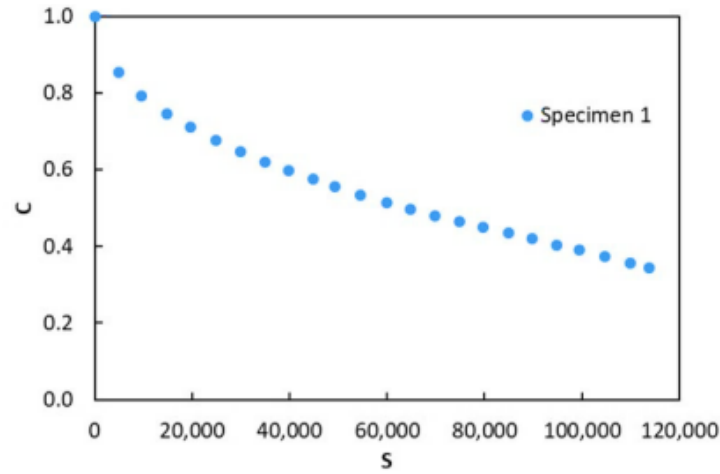


Figure 3.9: Filtering process for C Vs S curve[12]

Step 3: Calculation of the DR Failure Criterion. To determine the DR failure criterion, as currently implemented in FlexMAT™, it is essential to identify the failure point of the specimen [26, 138]. The definition of failure for the current specimens is based on identifying the cycle (N_f) in which the peak-to-peak stress, multiplied by the cycle number, reaches its local maximum. This critical point is illustrated in Figure 3.9, highlighting the specific cycle where failure occurs. After determining N_f each specimen, the DR failure criterion is computed using Equation 3.19 for each individual specimen. This equation allows for the assessment of the failure threshold based on the specific characteristics of each sample. Furthermore, to obtain a mixed representative value of DR equation 3.20 is utilized. This approach integrates the failure criteria from all specimens, providing a comprehensive overview of the material's behavior under stress conditions [26].

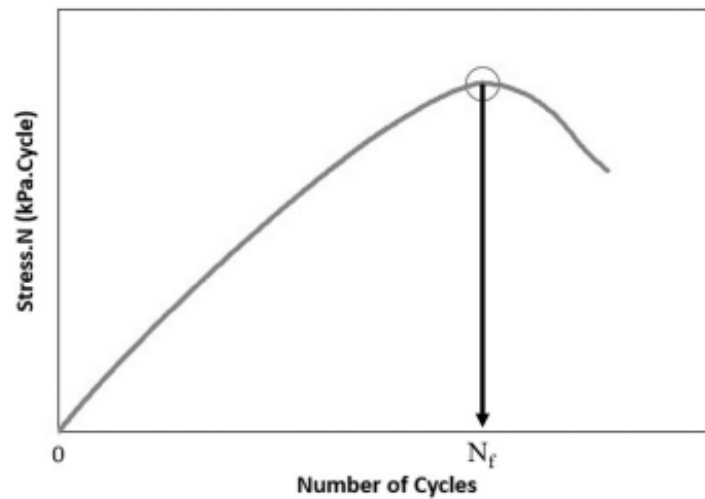


Figure 3.10: C Vs S curve stress representation[26]

$$D_i^R = \frac{\int_0^{N_{f,i}} (1 - C(N_i)) dN}{N_{f,i}} \quad (3.19)$$

$$D^R = \sum_{i=1}^M \frac{D_i^R}{M} \quad (3.20)$$

Where

C(N)_i=integrity level at cycle N for the i-th specimen

N_{f, i} =failure cycle number for the i-th specimen

M= Total number of tested specimens

Step 4: Characterization of DPSE as a Function of Reduced Strain Rate.

To compute the DPSE in relation to reduced strain rate for a specific initial material integrity, a monotonic tensile test is simulated [139]. This simulation utilizes the coefficients from the damage characteristic curve along with a Prony series representation of the dynamic modulus to derive the relaxation modulus of the mixture. The process is outlined in the following steps:

Step 4.1:

Establishing the Reduced Strain Rate Vector. This step involves creating the reduced strain rate vector, which will be used to calculate DPSE values during the simulated strain-controlled monotonic test. In FlexMAT™, the reduced strain rate vector includes 20 unique values, starting from 4×10^{-6} s⁻¹ and progressing with consistent increments up to 5×10^{-3} s⁻¹. The strain input for the simulated monotonic test is represented by Equation 3.21. This formulation ensures a comprehensive range of strain rates is evaluated, allowing for an accurate characterization of the material's behavior under varying loading conditions.

$$\varepsilon = k \xi \quad (3.21)$$

Where,

ε = input strain.

k = strain rate (s⁻¹); and

ξ = reduced time.

Step 4.2: Calculation of Pseudo Strain. In this step, the pseudo strain is calculated using Equation 3.22 in conjunction with the Prony series for the relaxation modulus, as outlined in Equation 3.23 [140]. This calculation enables the determination of how the material responds under the specified reduced strain rates, integrating the effects of viscoelastic behavior into the analysis.

$$\varepsilon^R(\xi) = \frac{1}{E_R} \int_0^\xi E(\xi - \tau) \frac{d\varepsilon}{d\tau} d\tau \quad (3.22)$$

$$E(\xi) = E_\infty + \sum_{i=1}^{17} E_i e^{-\frac{\xi}{\rho_i}} \quad (3.23)$$

Where,

εR = pseudo-strain.

E_R = reference modulus (kPa);

ξ = reduced time (s);

ε = input strain.

$d\tau$ = differential of time (s);

$E(\xi)$ = relaxation modulus (kPa);

E_∞, E_i = Prony coefficients (kPa); and

ρ_i = Prony coefficients (s).

Step 4.3: Solving the Ordinary Differential Equation (ODE). In this step, the ordinary differential equation (ODE) presented in Equation 3.24 is solved to determine the damage (S) as a function of time under the specified conditions. This solution provides insights into how damage evolves over time during the monotonic test, reflecting the material's response to applied stress.

$$\frac{dS}{d\xi} = \left(-\frac{1}{2} \frac{dC}{dS} (\varepsilon^R)^2 \right)^\alpha \quad (3.24)$$

Where,

S = damage.

ξ = reduced time (s);

C = material integrity, Equation 3.18 ;

εR = pseudo-strain; and

α = damage evolution rate parameter Equation 3.17.

To address this ODE, an initial material integrity value is essential. In the current iteration of FlexMAT™, four specific values are employed to effectively characterize the DPSE behavior: $C_0 = 0.99$, $C_0 = 0.90$, $C_0 = 0.80$, and $C_0 = 0.70$ [12]. These values represent different states of material integrity, allowing the model to evaluate how damage progresses over time under each condition. By incorporating these initial integrity levels, the analysis can capture a range of scenarios, providing valuable insights into the material's performance and its susceptibility to damage as it undergoes loading. This approach enhances the accuracy and applicability of the results in practical settings.

Step 4.4: Determination of the Point of Maximum Stress. In this step, the point of maximum stress is calculated using Equation 3.24. This equation helps identify the peak stress experienced by the material during the simulated test, which is critical for understanding the material's failure mechanisms and overall performance under load.

$$\sigma(\xi) = C(\xi)\varepsilon^R(\xi) \quad (3.25)$$

Where,

σ = stress (kPa);

C = material integrity.

εR = pseudo-strain.

Step 4.5 Calculation of DPSE.

In this phase, the dissipated pseudo strain energy (DPSE) is computed using Equation 3.5.2 for every strain rate outlined in Step 4.1 and for each initial integrity value specified in Step 4.3. By systematically calculating DPSE across these varying conditions, this step enables a thorough analysis of how the material responds to different strain rates and levels of initial integrity [27]. This comprehensive approach not only reveals the material's energy dissipation characteristics but also highlights the relationship between strain rate and integrity, offering insights into the material's performance under various loading scenarios. The resulting data can be invaluable for predicting failure modes and enhancing the material design process.

$$DPSE = \int_0^{\xi_{\max}} (\varepsilon^R(\xi))^2 (1 - C(\xi)) d\xi \quad (3.26)$$

Where,

DPSE = Dissipated Pseudo-Strain Energy;

$\xi_{\max}$ = reduced time for peak-stress (s);

εR = pseudo-strain; and

C = integrity of the material

Step 4.6: Fitting Results to a Power-Law Function.

In this step, the results obtained from Equation for each initial integrity level are fitted to a power-law function, as described in Equation 3.26. This fitting process helps to establish a mathematical relationship that captures the behavior of DPSE across the defined strain rates for a specific initial integrity. By utilizing a power-law model, this step provides a simplified representation of the complex relationship between dissipated pseudo strain energy and strain rate. This enables easier interpretation of the data and facilitates comparisons across different conditions, enhancing the understanding of material behavior under various loading scenarios.

$$DPSE = a(k)^b \quad (3.27)$$

Where,

DPSE = Dissipated Pseudo-Strain Energy;

a, b = fitting coefficients; and

k = reduced strain rate (s-1).

Figure 3.11 provides an illustrative example of typical DPSE characterization results in FlexMAT™. This example highlights how the DPSE varies as a function of reduced strain rate and initial material integrity, offering a clear visual representation of the trends and relationships derived from the characterization process[27]. The graph serves as a reference for interpreting the material's response under different testing conditions, showcasing the effectiveness of the FlexMAT™ framework in modeling and analyzing material behavior

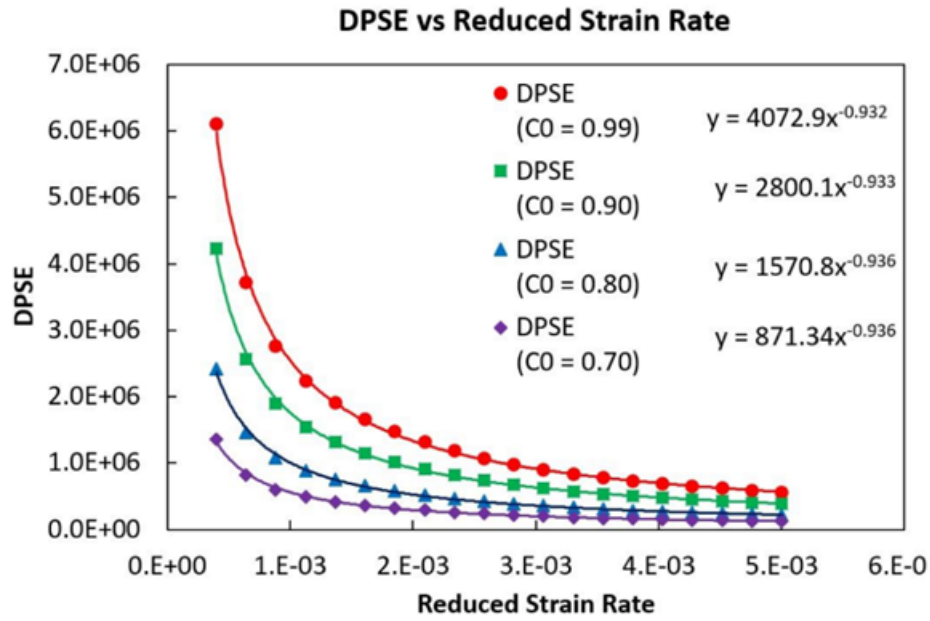


Figure 3.11: Dissipated pseudo strain energy rate of asphalt mixtures[27]

Step 5: Apparent Damage Capacity Index. The apparent damage capacity, denoted as S_{app} , serves as the fatigue cracking index and is calculated using equation 3.27. This index provides a quantitative measure of the material's capacity to resist fatigue-induced damage under repeated loading conditions. S_{app} plays a critical role in assessing the durability and long-term performance of the material, helping predict its susceptibility to cracking and failure over time [91, 89].

$$S_{app} = 1000^{\frac{\alpha}{2}-1} \frac{a_T^{\frac{1}{\alpha+1}} \left(\frac{D^R}{C_{11}} \right)^{\frac{1}{C_{12}}}}{\left| E^* \right|_{LVE}^{\frac{\alpha}{4}}} \quad (3.28)$$

Where,

S_{app} = apparent damage capacity.

a_T = shift-factor for project location temperature.

C_{11}, C_{12} = damage characteristic curve fitting coefficients,

D^R = failure criterion,

α = damage evolution rate parameter,

$|E * |LVE$ = dynamic modulus at target temperature and 10 Hz (kPa).

3.5.2.1 Methods to compute the Sapp

To compute the Sapp index in FlexMAT™, users are provided with two primary methods for inputting temperature data. The first method involves selecting a specific State and City within the U.S. by entering values in FlexMAT cracking. Once the location is chosen, FlexMAT™ calculates the corresponding target temperature in the background, based on regional climate data, and uses this temperature for Sapp calculations [141, 142]. Alternatively, for users requiring more control over the temperature selection, the second method allows for direct input. By selecting "Other" in the state selection users can manually input the desired temperature through the direct climatic Performance Grade (PG) selection. The chosen temperature is then calculated through Equation 3.28, offering flexibility for custom climate settings or regions outside the predefined state/city options. This dual input approach ensures that Sapp calculations are tailored to the specific environmental conditions or requirements of the analysis [143].

$$T_{app} = \frac{T_H + T_L}{2} - 3 \quad (3.29)$$

Where,

TSapp = Sapp calculation temperature (° C);

TH = high climatic PG temperature (° C);

TL = high climatic PG temperature (° C).

3.5.3 Coefficient of Thermal Contraction (CTC) model

This model quantifies how much the material contracts in response to temperature changes, playing a critical role in understanding how thermal stresses affect the mixture's overall performance and durability [144]. The CTC analysis can be done using the following equation 3.30.

$$CTC = CTC_g + (CTC_L - CTC_g) \frac{e^{\frac{T-T_g}{R}}}{1 + e^{\frac{T-T_g}{R}}} \quad (3.30)$$

Where,

CTC = coefficient of thermal contraction of the mixture (°C-1)

CTC_g = coefficient of thermal contraction of mixture, below glassy temperature (°C-1)

CTC_L = coefficient of thermal contraction of mixture, above glassy temperature (°C-1)

T = material temperature (°C)

T_g = glassy temperature (°C)

R = regression constant related to volume change near glassy transition temperature.

Although in FlexMAT there are three levels to calculate CTC of the asphalt mixtures, but this study chooses level three to avoid any extensive laboratory testing. The level three for CTC model can be explained as Level 3 follows a similar methodology to Level 2, with one key difference: instead of using measured binder CTC (Coefficient of Thermal Contraction) coefficients as inputs, these are estimated based on the binder's low performance grade (PG). The inputs required for Level 3 include the following:

- VMA (Voids in Mineral Aggregate)
- Gsb (Bulk Specific Gravity of the aggregate)
- CTC of the aggregate
- Low PG of the binder

This approach allows for estimating the mixture's thermal contraction properties when measured binder data is unavailable, relying instead on the binder's performance grade to infer CTC values. The approximation of the binder's CTC behavior relies on assuming the constants outlined in Table 3.11. These constants are derived based on empirical data and are used to estimate the thermal contraction properties of the binder when direct measurements are not feasible. By incorporating these constants, the model can effectively simulate the binder's response to temperature changes, despite the absence of measured CTC coefficients. This approach enhances the flexibility of the analysis, allowing users to predict the thermal behavior of asphalt mixtures under varying conditions. It simplifies the process while maintaining accuracy, ensuring that reliable estimates of the binder's performance can be obtained using readily available information, such as the low performance grade (PG) [13].

Table 3.11: CTC coefficient as an input in FlexMATTM Cracking [13]

CTC coefficient	Values for level 3
CTC _g	0.003
CTC _L	0.006
R	6
T _g	-40°C for low PG = -40 -25°C for -34 ≤ low PG ≤ -22

3.5.4 Shift model for permanent deformation

The theoretical background about the shift model has already been provided in chapter 2. This section aims to explain the steps of calculation in determining the permanent strain using FlexMAT rutting tool[15]. Determining the coefficients for the shift model using the elements displayed on the FlexMAT rutting a series of the following steps:

Step 1

Compute the reference temperature (T_{ref}) by taking the average of the tested temperatures for the high temperature measurement, TH.

Step 2

Determine the optimal values for ϵ_0 , N_1 , and β in equation 3.31 by minimizing the cumulative squared error between the observed vertical permanent strain (averaged across specimens for each cycle) and the predicted vertical permanent strain values [14]. This is done using equation 3.31 for the high temperature assessment and the initial 200 cycles (intermediate stress level) of the test.

$$\epsilon_{vp} = \frac{\epsilon_0 \times N_{red}}{(N_1 + N_{red})^\beta} \quad (3.31)$$

ϵ_0 , N_1 , β = coefficients of the incremental model;

N_{red} = reduced number of cycles at reference loading conditions;

$$Error^2 = \sum_{i=1}^{200} (\epsilon_{vp,i} - \hat{\epsilon}_{vp,i})^2 \quad (3.32)$$

ϵ_{vp} = observed vertical permanent strain; and

$\hat{\epsilon}_{vp}$ = predicted vertical permanent strain.

The preliminary values chosen for the adjustment process are outlined in Table 3.12. It is crucial to highlight that this situation, incorporating elevated temperature and moderate stress levels, will act as the benchmark condition for the shift model in the forthcoming computations.

Table 3.12: Fitting coefficients for shift model[14]

Fitting coefficient	Initial Values for shift model
ϵ_0	0.002
N_1	2
β	0.75

Step 3

Determine the final reduced cycle, N_{red} , for the additional stress levels and temperatures. This is achieved by utilizing the same standard Microsoft Excel Solver setup as before to minimize the squared error between the observed and predicted vertical permanent strain for the cycles and temperature levels listed in Table 3.13. The initial values from the "Initial Value" column are used while fitting N_{red} in equation 3.31, along with the previously calibrated ϵ_0 , N_1 , and β .

Note that the objective function in Step 2 considers the cumulative squared error outlined in Equation 3.31. In Step 3, however, the objective function focuses on the squared error of an individual point, specifically the final cycle of each stress level. It is important to recognize that for the reference condition (high temperature level and intermediate stress), N_{red} is set to 200.

Step 4

Table 3.13: Initial values for reduce number of cycles [14]

No of Cycles	Level of the temperature	Initial Value for N_{red}
400	High	100
600	High	100
200	Low	0.5
400	Low	0.1
600	Low	0.1

Compute the shift factor for each stress condition at both temperatures by employing Equation 3.32.

$$a_{\sigma_y, n} = \begin{cases} \log \left(\frac{N_{red, n}}{200} \right), & \text{for } n = 1 \\ \log \left(\frac{N_{red, n} - N_{red, n-1}}{200} \right), & \text{for } n = 2 \text{ or } 3 \end{cases}$$

a_{σ_y} = shift factor for vertical stress.

N_{red} = number of cycles reduced under reference loading conditions; and

n = stress level associated with the chosen temperature level.

Step 5

Entails the determination of the adjusted load duration shift factors and the adjusted load durations for each temperature level, as indicated by equations 3.33 and 3.34, respectively.

$$a_{\xi_p} = a_{\sigma_y, \text{intermediate}} \quad (3.33)$$

$$\xi_p = \frac{0.4}{10^{(0.000642 \times (T^2 - T_H^2) + 0.15649 \times (T - T_H))}} \quad (3.34)$$

Where:

- a_{ξ_p} = shift factor for reduced load duration.
- $a_{\sigma_y, \text{intermediate}}$ = vertical stress shift factor for the intermediate stress condition.
- ξ_p = duration of reduced load.
- T = temperature being tested of interest ($^{\circ}\text{C}$); and
- TH = tested elevated temperature ($^{\circ}\text{C}$).

It is important to note that if the temperature of interest is the tested elevated temperature: (1) a_{ξ_p} and a_{σ_y} are

equal to 0 (2) ξ_p is 0.4 seconds, representing the pulse duration, thereby confirming the reference condition. For further information on pulse and rest durations, refer to the work of Kim et al., 2017 [67].

Step 6

Determines p_1 and p_2 through Equation 3.35 and d_1 and d_2 via Equation 3.36, based on the shift factors obtained in Step 5 for each temperature.

$$a_{\xi_p} = p_1 \log(\xi_p) + p_2 \quad (3.35)$$

$$a_{\sigma_v} = d_1 \log\left(\frac{\sigma_v}{P_a}\right) + d_2 \quad (3.36)$$

Where:

- a_{ξ_p} = factor for adjusted load duration shift.
- p_1, p_2 = coefficients of the adjusted load duration shift factor.
- ξ_p = duration of adjusted load.
- a_{σ_v} = factor for vertical stress shift.
- d_1, d_2 = coefficients of the vertical stress shift factor.
- σ_v = vertical stress (kPa).
- P_a = atmospheric pressure for stress adjustment (kPa); and
- T = temperature ($^{\circ}\text{C}$).

3.5.4.1 Calculation of rutting strain index (RSI) through FlexMAT™ rutting

Rutting Strain Index (RSI) functions as an alternative output from FlexMAT™, as it does not constitute a material characteristic. Rather, it serves as an index that delivers important insights for evaluating the performance of the characterized mixture under specific climatic conditions and a designated structural position of the mixture (i.e., surface, intermediate, or base layer). RSI is defined as the simulated average permanent deformation over a period of 20 years of traffic, involving 30 million equivalent single-axle loads applied at a design velocity of 96 km/h. For additional information on the computation of RSI, readers are advised to refer to the work of Ghanbari et al. Temperature is recognized as a significant factor influencing pavement stiffness [145]. To calculate RSI and account for realistic climatic conditions, FlexMAT™ utilizes a consolidated database of temperature profiles sourced from 2,798 stations across the United States [25]. However, in order to select the specified temperature, for example, of the Sicily region, it is supposed to be matched with the state having the same pavement temperature. Temperature is recognized as a significant factor influencing pavement stiffness. To calculate RSI and account

for realistic climatic conditions, FlexMAT™ utilizes a consolidated database of temperature profiles sourced from 2,798 stations across the United States [25]. However, in order to select the specified temperature, for example of Sicily region, it is supposed to be matched with the state having same pavement temperature [133].

3.5.5 Outputs of FlexMAT cracking and rutting

As the PASSFlex System is a kind of sequential system that needs outputs from AMPT as an input in FlexMAT cracking and rutting. For a pavement simulation program by FlexPAVE, input from FlexMAT cracking and rutting [12, 15]. Before proceeding towards the FlexPAVE simulation program the following outputs summarized in Table 3.14 needs to be imported to FlexPAVE for further analysis.

Table 3.14: Summary of FlexMAT cracking and rutting outputs[12, 15]

Source analysis	Characterization type	Coefficients	Symbols of coefficients
Dynamic Modulus Test	Time-Temperature Superposition	4	T_{ref}, a_1, a_2, a_3
	2S2P1D	7	$\delta, k, h, \beta, E_{00}, E_0, \log(E)$
	Prony series	34	$T_i, E_i (i = 1, 2, 3, \dots, 17)$
Dynamic Modulus Test	Damage Evolution Rate Factor	1	α
Cyclic Test	fatigue Damage Characteristic Curve	2	C_{11}, C_{12}
Cyclic Test	fatigue Failure Criterion	1	D_R
FlexMAT cracking	Damage Capacity Index	1	S_{app}
Cyclic test	fatigue Dissipated Pseudo-Strain Energy	8	$a_i, b_i (i = 1, 2, 3, 4)$
CTC	CTC Model	4	CTC_g, CTC_L, T_g, R
Aging	PAM and AMAC	3	$c, \log G^* _{STA}, M$
SSR	Shift model	8	$\varepsilon_0, N1, \beta, p1, p2, d1, d2, T_{ref}$
FlexMAT rutting	Rutting strain index	1	RSI

3.6 Pavement performance analysis and design

The pavement performance and its design can be analyzed using the FlexPAVE simulation program. The FlexPAVE program allows for testing and comparing multiple pavement designs. It evaluates the expected performance of these designs and enables pavement engineers to decide on the appropriate number of layers. FlexPAVE also offers guidance on material selection and placement within the pavement structure while assessing the effects of various traffic and environmental factors [12, 15]. The program incorporates S-VECD and shift models, which are highly

efficient for testing, and predicts how materials will perform under diverse loading and environmental conditions. This gives FlexPAVE several advantages over other advanced pavement design programs [145, 146]. It applies VECD principles to improve the accuracy of its predictions compared to elastic analysis tools. Additionally, FlexPAVE considers the material properties of all asphalt layers, determining how each one impacts cracking and rutting using mechanistic principles [25]. This approach eliminates the need for prior assumptions about where cracks will start or how they will evolve. This ability to analyze top-down cracking without such assumptions is key to FlexPave's effectiveness in evaluating complex pavement structures. Its flexible modeling approach allows cracks to develop and spread based on the material's fundamental properties, leading to a more realistic and precise simulation of top-down cracking than was previously achievable [146]. To begin with the FlexPAVE it requires the following inputs for the performance analysis

1. General information on project
2. Structural layout
3. Climate and Traffic data.

As it has been already overviewed in previous sections of this chapter that the whole PASSFlex system is a sequential process. And in order to proceed for the performance and design analysis of the pavements the outputs from FlexMAT cracking and rutting summarized in table 3.14 needs to be imported in FlexPAVE. This research specifically used the FlexPAVE v 1.1 MATLAB based software, developed by the North Carolina state university USA to analyze the conventional and alternative paving materials. This section specifically highlights the inputs required for the FlexPAVE program.

3.6.1 General information of the project

In the general information tab of the simulation program, the pavement type has to be selected for the analysis of each mixture. To analyze the mixtures, the pavement performance and response analysis option is available [12]. The option has to be selected based on the outputs required for the project. For this research work, the details about pavement performance analysis have been discussed in the basic information section. Before proceeding towards the design structure information, some basic information needed by the FlexPAVE has been discussed in the below section.

3.6.1.1 Basic Information

The general information tab contains essential details about the pavement, including its type, location, traffic data, and optional descriptions. In order to analyze the conventional and APMs mixed, the information was entered for each mixture separately.

3.6.1.2 Pavement type

FlexPAVE currently supports two pavement types: new pavement and AC-on-AC overlay rehabilitation. However, the rehabilitation option is not yet available and may be included in future updates [25]. In this research for conventional and APMs mixtures, the new pavement type option was selected for each mixture.

3.6.1.3 Pavement Location

Pavement location is defined using latitude and longitude to retrieve climate data. In this version of FlexPAVE, this feature is not available. However, for the analysis of conventional and APMs mixtures, the climate data was entered manually for each mixture in the climate data section of the project.

3.6.1.4 Traffic

This section allows users to select the type of traffic data used for analysis. At present, FlexPAVE only supports design Vehicle analysis. A Traffic Spectrum option will be introduced in future versions [12]. For conventional and APMs mixtures, the details for traffic data have been provided in the traffic data section. Please note that for each mixture the traffic data was entered manually considering the heavy traffic for the worst condition in each case of the mixture.

3.6.1.5 Analysis option for conventional and APMs mixtures

FlexPAVE offers two types of analysis. **pavement response analysis:** This analysis provides stress, strain, and displacement results. **pavement performance analysis:** This more comprehensive analysis includes all response data plus additional performance metrics, such as fatigue cracking, thermal stress, and rutting, based on the selected performance criteria. For the analysis of conventional, APMs this option has been selected for further analysis.

3.6.1.6 Design structure details for conventional and APMs mixtures

The design structure tab in the FlexPAVE simulation program is divided into three main sections:

3.6.1.7 Structure general information

The pavement structure was named accordingly with each mixture's code. For conventional mixtures the structure name was coded as AC10 (control) while for the rubberized mixtures the structure names were coded as AC8-Rd and AC10-Rh, respectively. The RAP mixtures were coded as AC10-RA50, AC10-RdRA50, and AC10-RhRA50.

Pavement Schematic: This section provides a visual representation of the pavement structure for conventional and APMs, allowing to review the design [12].

Layer Properties: FlexPAVE supports pavements with any number of material layers [12]. Users can add, remove, or reposition layers using the Add Layer, Remove Layer, and Move Layer options. In this research, the layers as shown in Figure 3.12 have been selected for Italian pavement design for a period of 10 years. According to the Italian pavement design, the top three layers are made up of asphalt concrete materials [28, 17]. Please note that this research only focused on wearing surface "USURA B" for conventional and APMs [2]

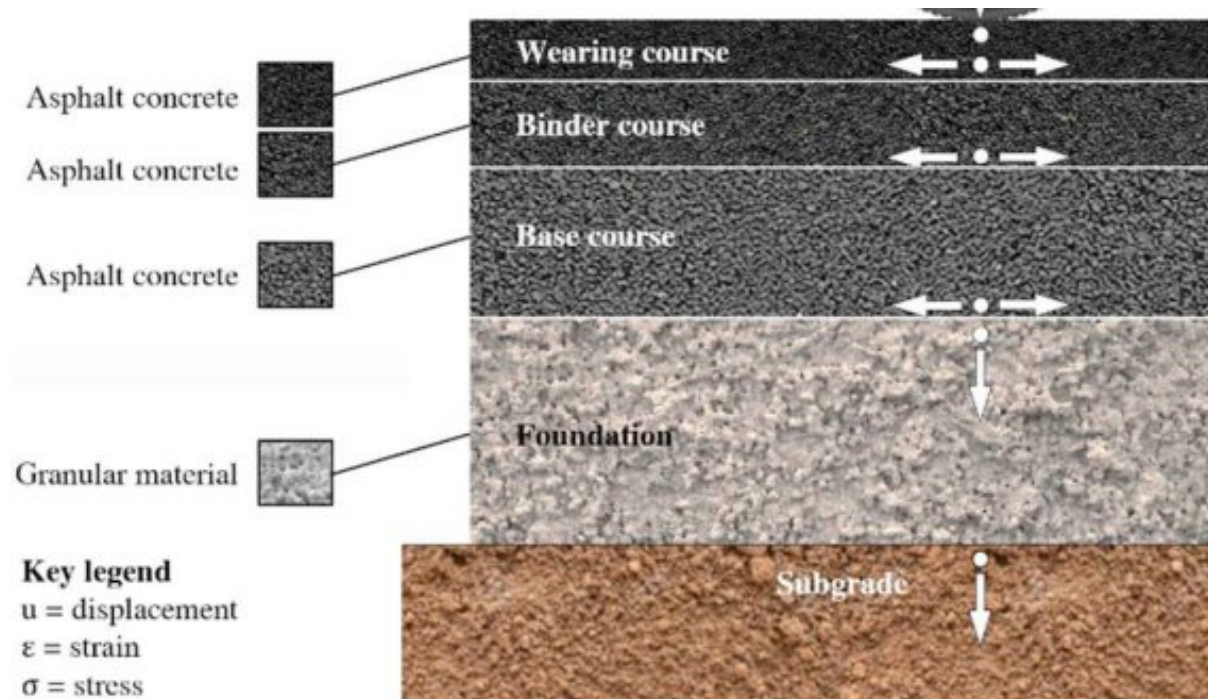


Figure 3.12: Addition of layers according to the Italian pavement design in FlexPAVE simulation program [28, 17]

3.6.1.8 Climate data

To simulate the temperature conditions for the Sicily region, the manual input option in the temperature profile was entered. The pavement temperature was considered according to the hot climate conditions of the Sicily region [8].

3.6.1.9 Traffic Data

For the analysis of conventional and APMs in FlexPAVE, single-axle analysis (ESAL analysis) is used instead of single-wheel analysis. As a result, single-wheel and dual tires are selected. The axle load is set to 40 kN (9,000 lb), and the AADTT (Annual Average Daily Truck Traffic) is 700 with a linear growth rate [12].

3.6.1.10 Output and analysis options for conventional and APMs mixtures

In single-wheel dual tire analysis, the tire placement is determined by the dual tire spacing of 30.47 cm, meaning the right wheel is located at +15.23 cm (half of 30.47 cm). The left wheel is positioned at -15.23 cm (negative half of 30.47 cm). To ensure accurate pavement response assessment for conventional and APMs mixtures, the evaluation points tool automatically generates key measurement points at both wheel positions [12]. These points enable detailed analysis of pavement behavior across various asphalt pavement depths, capturing essential data such as stress, strain, and displacement for performance evaluation.

3.6.1.11 Results from FlexPAVE for conventional and APMs mixtures

. FlexPAVE provides a comprehensive analysis of pavement performance for conventional and APMs, allowing to examine various outputs, including fatigue cracking, tracking damage development over time rutting behavior, and rut depth progression. These outputs enable a detailed evaluation of pavement performance under different conditions. The outputs from the FLEXPAVE are discussed in details

3.6.1.12 Fatigue cracking

FlexPAVE allows to analyze fatigue damage for conventional and APMs by plotting damage (C), which represents the modulus after damage [147]. This can be visualized as:

Contour plots: It shows damage distribution at a specific point in time.

History plots: It tracks damage progression at a predefined node [148, 149]. The damage factor distribution, defined as N/N_f [150] where: N = Current number of load cycles N_f = Number of cycles to failure. When the damage factor reaches 1.0, the asphalt element is considered fully cracked.

3.6.1.13 Damage Evolution

FlexPAVE enables, to track damage evolution over the entire pavement lifespan for the conventional and APMs mixtures. This is visualized through plots that display how damage progresses over time. It allows percent damage evolution as a function of time, providing insights into the long-term deterioration of the pavement structure.

3.6.1.14 Rutting analysis

For the rut depth and permanent deformation analysis of conventional and APMs. The shift model incorporated in FlexPAVE allows us to predict the depth of the rut over the design of the paving structure [100].

3.7 Summary of the chapter

This chapter 3 provides a detailed discussion about the improvement of the pavement monitoring by describing the approach currently utilized by the Italian road authorities for their QA/QC testing. Keeping in view a scientific approach towards the improvement of pavement monitoring as shown in figure 1.4. As stated in the research question, how can the current pavement monitoring by the Italian road authorities be improved? The level of research already carried out and background study discussed in Chapter 2 leads to the hypothesis, “Are these current laboratory testing approaches able to discriminate between conventional bituminous and APMs”?

Moving on forward to support the hypothesis of this research study, what should be improved? This research study proposed and explained the laboratory testing approaches related to the damage-related characterization. The basic-level approach includes cracking, rutting, and moisture resistance for the discrimination of conventional and APMs mixtures. While the advanced-level testing approach is based on SVECD theory, damage modeling, and a simulation program that is elaborated in the experimental program of this research study. The main outputs of the advanced-level approach include stiffness, cracking resistance, and permanent deformation of conventional and APMs mixtures.

Finally, the main outcomes of both these testing approaches based on damage-related characterization are proposed to be used as a monitoring tools for more sustainable road pavements.

Chapter 4

Results and analysis of pavement monitoring tools for conventional and alternative paving materials using basic and advanced laboratory testing

4.1 Introduction

This chapter 4 provides detailed results and an analysis of the experimental campaign carried out and explained in Chapter 3. For more comprehensive understanding, this chapter is partially divided into five sections, as shown in Fig 4.1. The section-I in 4.2 elaborates the results on the volumetric and mechanical characteristics of conventional and APMs based on QA/QC by the Italian road authorities. Section-II in 4.3 of this chapter elaborates the basic-level approach using damage-related characterization for conventional and rubberized asphalt mixes based on IDT testing. The damage- related characterization for conventional and reclaimed asphalt mixes using the basic-level approach based on IDT testing is discussed in the section-III elaborated in 4.4. However, section-IV in 4.5 of this chapter is about an advanced-level damage-related characterization approach based on SVECD theory for conventional and rubberized asphalt mixes. The damage-related characterization of conventional and reclaimed asphalt mixes using the advanced-level approach based on SVECD theory has been summarized in section-V 4.6 of this chapter. The assessment of pavement monitoring tools for conventional and alternative paving materials by the proposed approaches have been compared and analyzed as a summary of this chapter.

4.2 Section-I

4.2.1 Quality control and quality assurance (QA/QC) by the Italian road authorities

To have the QA/QC for the wearing surfaces, according to ANAS 2016 Italian specification[2], the volumetric and mechanical characterization specification is carried out for conventional and APMs. The volumetric analysis includes the control of the void content, while the mechanical characterization includes the IDT testing and CTI obtained from IDT testing.

This section summarizes RO-I in section 1.2 about the ability of discrimination between the conventional and

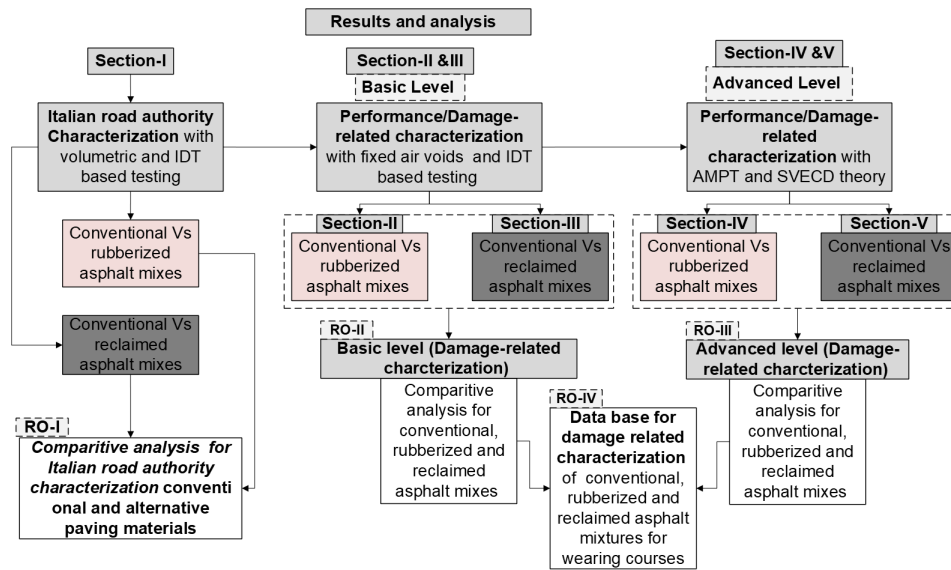


Figure 4.1: Description of results and analysis

APMs using QA/QC including service life by the Italian road authorities. The minimum QA/QC requirements are summarized in table 4.1.

Table 4.1: Thresholds for (QA/QC) by the Italian road authorities[2]

Characterization	Property	Benchmark values	Standards/ Specifications	Reference (s)
Conventional (QA/QC-Italian road authorities)	Void Content check	No of Gyration (%)		UNI EN 12697-5/ANAS
		N10	11-15	
		N120	3-6	
		N210	≥ 2	
	Indirect Tensile Strength at 25.0°C	ITS (MPa)	0.72-1.60	UNI EN 12697-23/ANAS [2]
		ITS (MPa)	≥ 0.6	
	Indirect Tensile Coefficient at 25.0°C	CTI (MPa)	≥ 65	UNI EN 12697-23/ANAS
		CTI (MPa)	50-150	

4.2.1.1 Volumetric characterization of conventional and rubberized asphalt mixes

The volumetric characterization according to the ANAS requirements [2] were carried out using the conventional and rubberized asphalt mixtures. It can be observed from the figure 4.2 that the AC10 (control) has the highest void content at lower gyrations (N10) and is close to the upper limits specified by the ANAS specifications [2]. While AC8-Rd has lower air void content, barely meeting the lower ANAS requirements [2]. At (N120) gyrations, the AC10 (control) is observed to be exactly matching the upper limits. While AC8-Rd and AC10-Rh are observed to be within a specified range with a difference of 0.5 percent. At higher gyrations (N210), the AC10 (control) exceeding the specified minimum of 2 percent significantly. While AC8-Rd and AC10-Rh are within the specified

limits [2]. From figure 4.2, it can be analyzed that all the mixes meet the ANAS requirements for void content at each gyration level. If the comparison between the conventional and rubberized mixes is carried out, then the conventional mix consistently shows higher air void content at each level of gyration compared to the other two mixes. The AC8-Rd mix, with lower void content, especially at N10 gyrations, might suggest that it is more compactable early in the compaction process. AC10-Rh performs similarly to AC10 (control) though with slightly lower air void content, particularly in the higher gyrations, indicating better compaction under pressure [151]. This void content observation summarizes that the control mix is more porous (higher air voids) at lower gyrations, which could influence permeability and durability [152]. While AC8-Rd and AC10-Rh showed an improved compaction, particularly after more gyrations, which might suggest better longevity and resistance to deformation. However, at this stage there is no clear difference between the control and rubberized mixes; further mechanical investigation is required to have a clarity between the performance of each mixture.

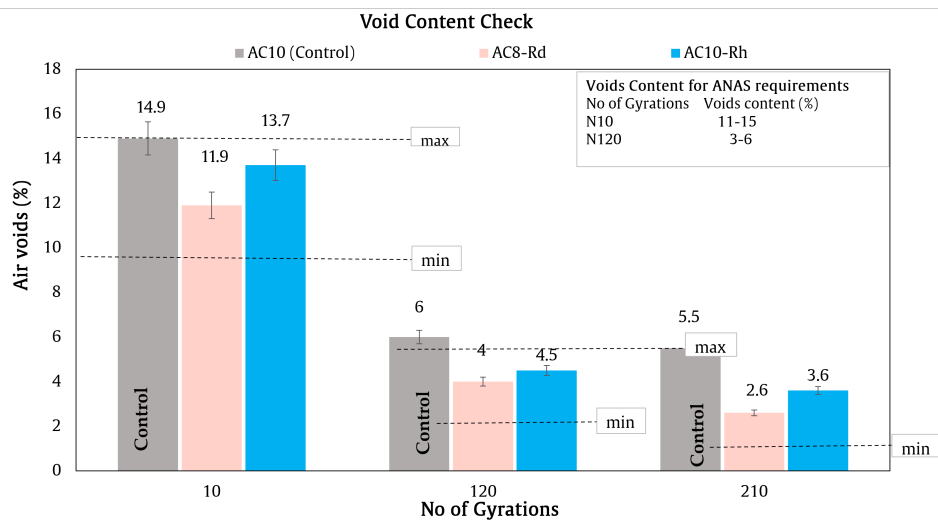


Figure 4.2: Voids content analysis of conventional and rubberized mixes

4.2.1.2 Volumetric characterization of conventional and reclaimed asphalt mixes

The volumetric characterization of conventional and reclaimed asphalt mixes has been summarized in figure 4.3. It can be observed in figure 4.3, that AC10 (control) generally has higher air void content compared to the other modified materials at each gyration level. The control mixture is at the upper limit of the requirement. This may suggest that the mix is relatively resistant to further compaction after initial densification. AC10-RA50 has the lowest air void content implying that adding 50 percent reclaimed asphalt enhances compaction, likely due to a stiffer binder in the recycled material that allows faster consolidation under load [153, 151]. In summary, the 4.3 shows that the reclaimed asphalt mixtures (AC10-RA50, AC10-RdRA50, and AC10-RhRA50) tend to reduce air voids compared to the AC10 (control) mix, which could indicate improved compaction or altered material behavior. All materials comply with ANAS requirements, suggesting that the modifications do not negatively affect performance under the specified void content criteria [2]. In general, excessively low air void content (especially under N210) could make some mixes prone to rutting and moisture damage, whereas higher air void content, as seen in the AC10 (control) could lead to reduced load-bearing capacity and susceptibility to cracking

[154]. The reclaimed asphalt mixtures demonstrate better compaction properties than the conventional asphalt mixtures, with AC10-RA50 performing the best in terms of densification.

However, overcompaction risks should be considered to avoid issues like binder drainage and loss of flexibility [155]. At this stage of volumetric analysis, no clear difference is observed between the conventional and reclaimed asphalt mixes; therefore, further analysis is required in terms of each mixture's mechanical performance.

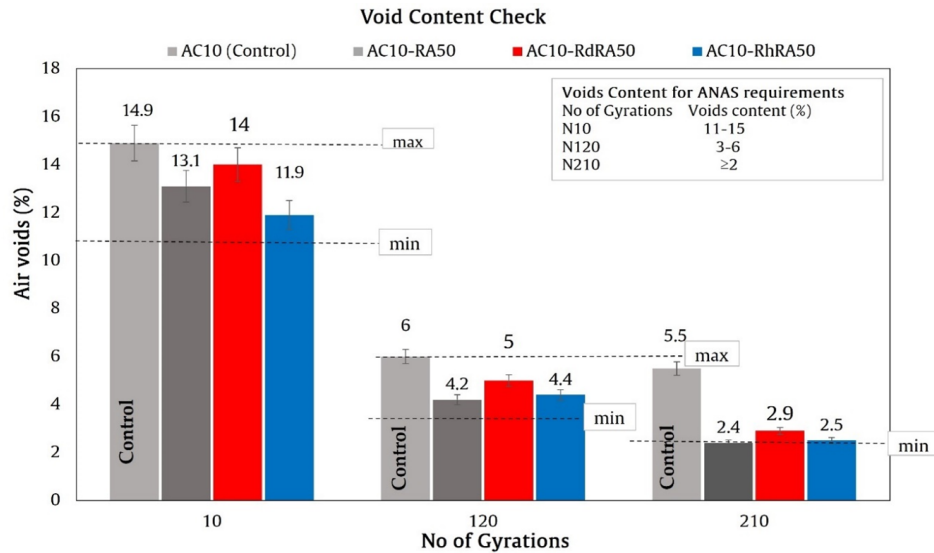


Figure 4.3: Voids content analysis of conventional and reclaimed mixes

4.2.1.3 Volumetric characterization for conventional and alternative paving materials

The summary of the volumetric characterization between the conventional, AC10 (control), and APMs including rubberized and reclaimed asphalt mixes shown in figure 4.4. Although each mixture, including rubberized and reclaimed mixes, is showing lower air void content at each gyration level and meeting the ANAS requirements [2]. The lower void content in rubberized mixes tends to improve the fatigue cracking while the reclaimed mixes tend to improve the rutting performance and stiffness [156]. From figure 4.4 it can be analyzed that modified asphalt mixtures (AC10-RdRA50, and AC10-RhRA50) exhibit better compaction potential compared to the control and rubberized mixtures. However, the lower air void content at high gyrations (especially in AC10-RA50) implies a need for careful balance in the field to avoid over-compaction, which could compromise the mix's durability and performance under traffic loads. The figure 4.4 as a summary clearly indicates a minimal difference between the conventional and alternative paving mixtures. In terms of mechanical performance and damage quantification, there is still a need for further mechanical analysis for more clarity between the performance of each mixture.

4.2.2 IDT analysis for conventional and rubberized asphalt mixes

To analyze the mechanical performance of conventional and rubberized asphalt mixes, the IDT testing conducted at 25.0°C is shown in figure 4.5. The control and rubberized asphalt mixes fulfill the requirements of ANAS and

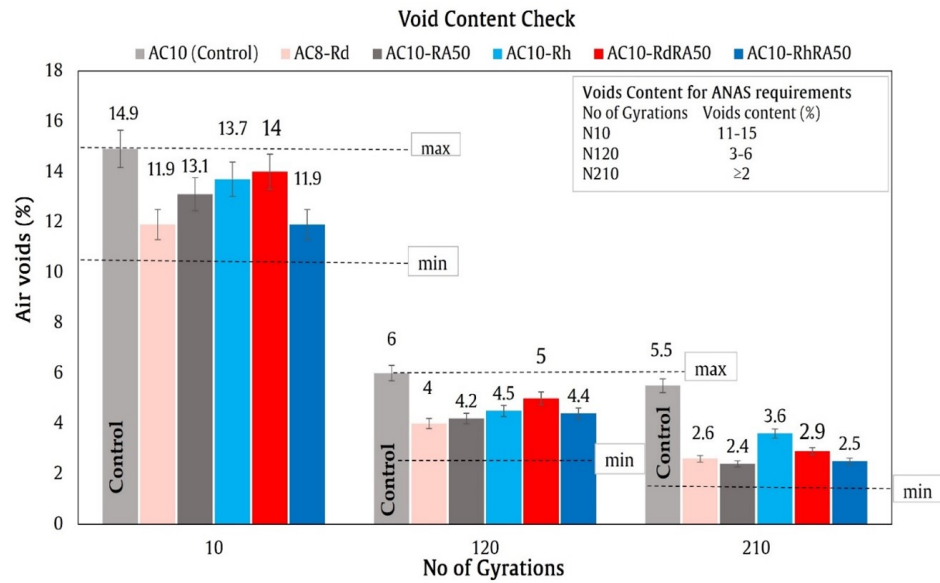


Figure 4.4: Voids content analysis of conventional and alternative paving mixes

ANAS CSA ACUSTICA specification [2, 128]. The AC10-Rh sample achieves the highest *ITS* (1.58 MPa), close to the maximum specified limit, suggesting it has the best tensile strength among the samples tested. This increase in *ITS* of 11.27 % compared to the control mix indicates enhanced performance in tensile strength, potentially implying better durability [157]. Although the better cracking resistance is observed in rubberized asphalt mixes as compared to the control mix, the difference according to the achieved results is quite minimal. The hybrid mix AC10-Rh shows a bit more cracking resistance than rubberized dry mix AC8-Rd. This may be due to the addition of polymers. Both modified samples AC8-Rd and AC10-Rh have higher *ITS* values than the control, with AC10-Rh showing the greatest increase. This suggests that the modifications in AC8-Rd and AC10-Rh contribute to improved tensile strength. Although these results highlighted a minimal difference, the cracking tolerance of each mixture is still questionable.

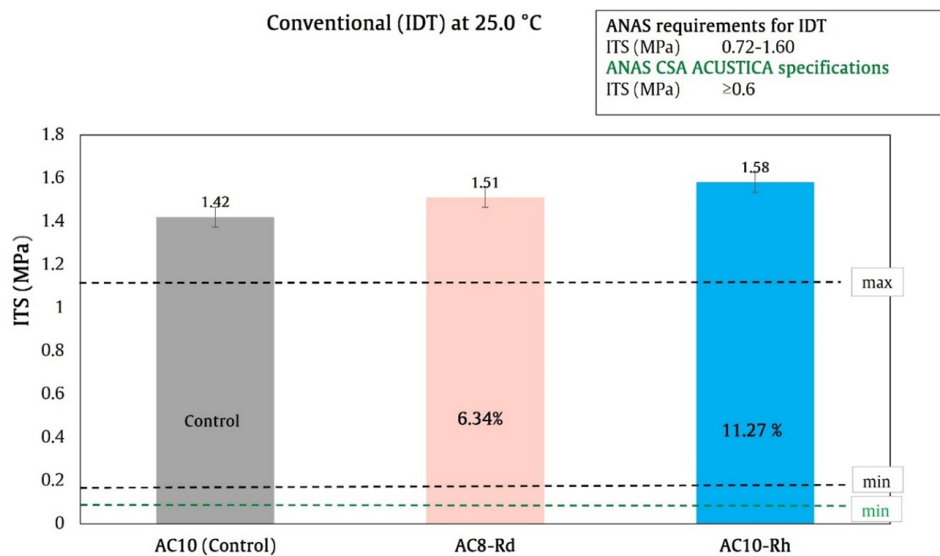


Figure 4.5: Indirect tensile strength analysis of conventional and rubberized mixes

4.2.2.1 CTI analysis for conventional and rubberized asphalt mixes

Cracking is a critical issue in materials exposed to stress, as cracks can lead to structural failure, especially in pavement, concrete, or asphalt [158]. In this context, CTI can be used to estimate how well each material can resist cracking under indirect tensile stresses [2, 128]. The CTI for the conventional and rubberized asphalt mixes were obtained using the IDT testing. Figure 4.6 shows the comparison between the conventional and rubberized asphalt mixes. The AC10-Rh sample, with its 118 MPa CTI, demonstrates the highest tensile strength among the three. This makes it the most suitable material for applications where tensile resistance is critical, such as in structural applications that experience high indirect tensile stresses. Although AC8-Rd shows a slight improvement over the control, the small increase may not justify its use over the control, unless it has other beneficial properties not reflected in CTI. AC10 (control) still demonstrates an adequate CTI well above the minimum requirement, making it a viable option where incremental tensile strength is not crucial. AC10-Rh stands out as the superior material for tensile strength, with an 8.26 % improvement over the control, suggesting potential use in high-stress environments [159]. AC8-Rd and AC10 (control), while meeting minimum requirements, may be suitable for less demanding applications. AC10 should provide adequate resistance to cracking in typical applications. However, under conditions with high tensile stress or heavy loading, it may be prone to cracking sooner than AC10-Rh. The minor improvement in AC8-Rd suggests slightly better cracking resistance than AC10 (control), but the difference is likely negligible. This material may not significantly reduce cracking in demanding applications.

If other performance metrics (such as rutting, moisture damage) are taken as a consideration, further analysis may be needed to justify choosing AC10-Rh over AC10 (control) [160].

Overall, the same level of the difference is observed as in calculation of ITS parameter between the conventional and rubberized asphalt mixes. Although this coefficient is used a recommendation of the mixtures for a possible construction of road infrastructures. But these results are unable to give a clear difference between conventional and rubberized asphalt mixtures.

4.2.3 IDT analysis for conventional and reclaimed asphalt mixes

Previous research suggested that Using 50 % RAP with rejuvenating agents significantly enhances the tensile strength, reaching the maximum permissible limits for conventional asphalt, which suggests these mixes could offer a strong, durable alternative to traditional asphalt [161, 162]. The figure 4.7 indicates that all the mixes including conventional and reclaimed asphalt mixes meets or exceeds the ITS range for ANAS IDT testing requirements (0.72-1.60 MPa) and ANAS CSA ACUSTICA specifications (≥ 0.6 MPa) [2, 128].

The addition of RAP alone (AC10-RA50) improves the ITS by almost 10 %. Using the rubberized mixes with addition RAP further enhances ITS to 1.66 MPa, a 17 % increase, hitting the upper threshold of ANAS standards

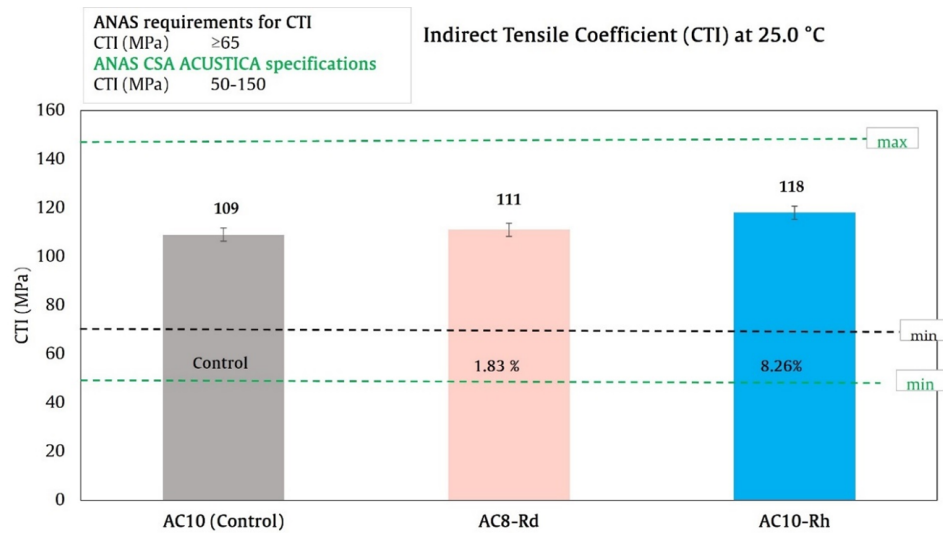


Figure 4.6: Indirect tensile Coefficient analysis of conventional and rubberized mixes

(1.60 MPa) [2]. Both AC10-RdRA50 and AC10-RhRA50 shown in figure 4.7 achieved the same ITS increase of 16.90 % over the control, suggesting similar performance benefits from their respective modifications. The addition of RAP in the samples has significantly improved the ITS, indicating better resistance to cracking under indirect tensile stress.

However, the values for AC10-RdRA50 and AC10-RhRA50 surpass the maximum recommended limit of 1.60 MPa, suggesting the need to balance the RAP content to avoid potential brittleness, which could paradoxically increase cracking under thermal stress or prolonged loading in specific conditions that needs further verification.

In conclusion, while 50 % RAP addition improves cracking resistance in terms of tensile strength, monitoring the upper limits of ITS is crucial to avoid excessive stiffness and potential cracking under specific environmental stresses.

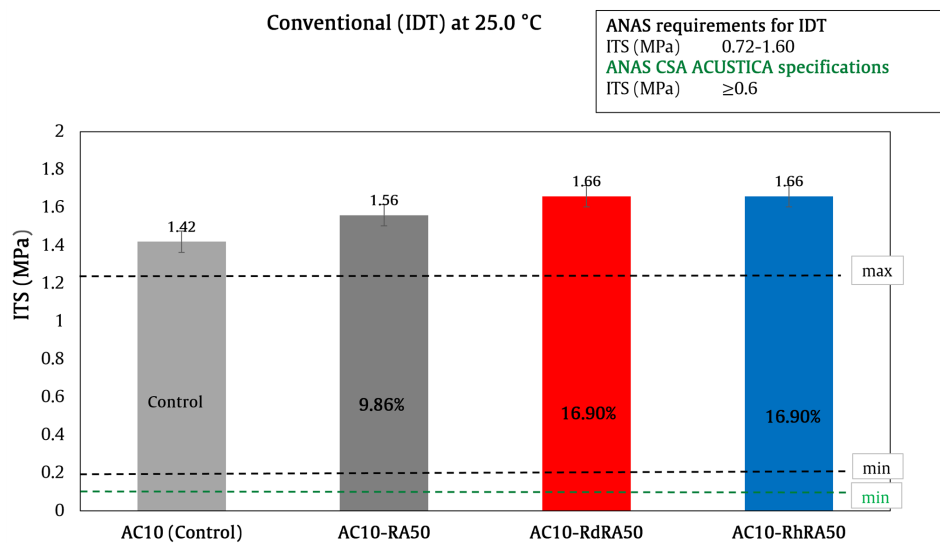


Figure 4.7: Indirect tensile strength analysis of conventional and reclaimed mixes

4.2.3.1 CTI analysis for conventional and reclaimed asphalt mixes

The conventional mixture AC10 (control) without RAP has a CTI value of 109 MPa as shown in figure 4.8. This provides a baseline measure of tensile resistance and cracking performance for comparison with reclaimed mixes [163]. AC10-RA50 (50 % RAP) shows a minimal increase in CTI to 110 MPa (+0.92 %), suggesting a slight enhancement in tensile characteristics but not a substantial improvement in cracking resistance compared to the control. AC10-RdRA50 and AC10-RhRA50 (both with 50 % RAP and additional treatment) show significant improvements, each reaching 120 MPa (10.09 % increase over the control). The substantial CTI improvement in AC10-RdRA50 and AC10-RhRA50 implies that the additional rubber and polymers in these samples greatly enhances tensile strength and cracking resistance beyond what is achieved by RAP addition alone. Adding 50 % RAP alone (without modification) does not significantly enhance CTI or cracking resistance, as evidenced by the 0.92 % increase over the control.

In conclusion, figure 4.8, suggests that RAP, when used alone, provides limited tensile benefit. However, RAP combined with modifications results in significant CTI improvements, enhancing the mix's overall resistance to cracking and meeting performance standards effectively. This highlights the importance of modifying RAP mixtures to achieve optimal performance.

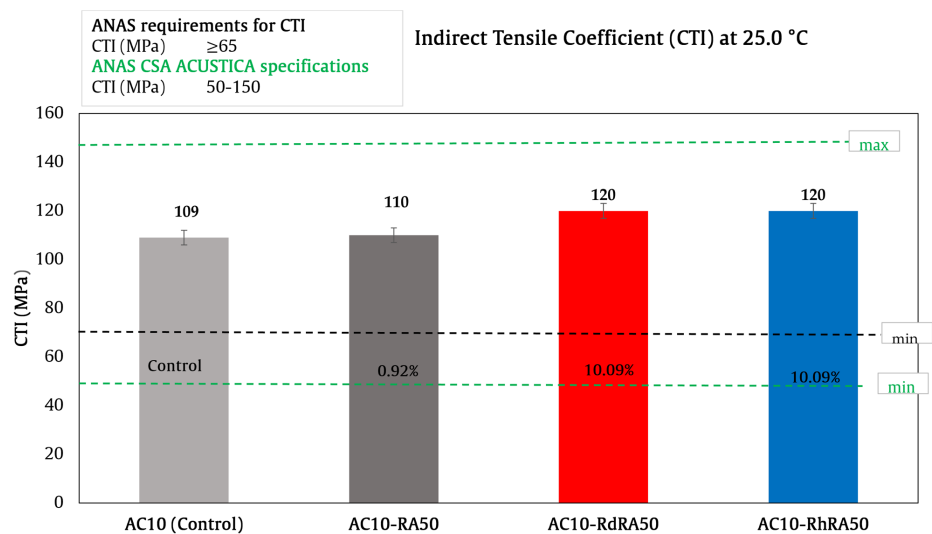


Figure 4.8: Indirect tensile strength analysis of conventional and reclaimed mixes

4.2.4 Mechanical characterization for conventional and alternative paving mixtures

As a summary of conventional and APMs, figure 4.9 indicates that all mixtures meet the minimum requirement of 0.6 Mpa ANAS (CSA Acustica) [2, 128]. The ITS values for all mixtures fall within or just reach the upper limit of the specified range (0.72–1.60 MPa) [2]. The alternative mixtures (AC8-Rd, AC10-RA50, AC10-Rh, AC10-RdRA50, and AC10-RhRA50) show slight increase in ITS compared to the control mixture, indicating a slightly improved tensile strength with these modifications. The alternative paving mixtures (particularly

AC10-RdRA50 and AC10-RhRA50) exhibit a significant improvement in mechanical strength, reaching the upper limit of the ANAS specification for ITS at 1.60 MPa, suggesting these could be viable options for enhancing pavement durability [164, 165, 166]. The data indicate that modifications, especially in samples AC10-RdRA50 and AC10-RhRA50, enhance the tensile strength of asphalt mixtures, making them more durable. These findings are relevant for road construction applications where higher ITS values can contribute to better performance under load. The improvement percentage highlighted for each modified sample (such as 6.34 %, 9.86 %, 11.27 %, and 16.90 %) shows the effectiveness of each modification. In conclusion, the alternative materials show significant improvement in tensile strength over the control, meeting and, in some cases, reaching the upper limits of ANAS standards [2], suggesting these mixtures would perform well under tensile stress conditions. However, the performance-based damage characterization can provide more comprehensive difference about the conventional and alternative paving materials.

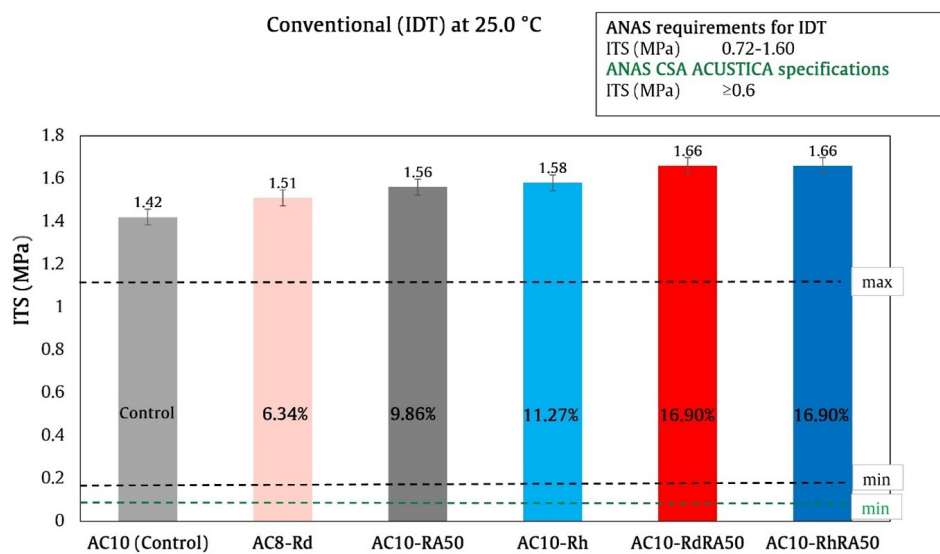


Figure 4.9: Indirect tensile strength analysis of conventional and alternative paving materials

The indirect tensile coefficient of conventional and alternative paving materials is summarized in figure 4.10. As a summary it can be observe that all the mixes including conventional, rubberized and reclaimed are fulfilling the range of ANAS and ANAS CSA ACUSTICA specifications. The rubberized dry mix showed a minimal difference as compared to the conventional mix. The 50 % addition of reclaimed mix AC10-RA50 showed a negligible amount difference in tensile strength. This may be occurred due to the stiffness of mixture. AC10-Rh mixture showed a greater difference as compared to the rubberized mixture. This needs further quantification of damage in terms of cracking resistance. By the combination of rubber with reclaimed mixes showed a remarkable difference as compared to the reclaimed and rubberized mixes. This also needs a further quantification of damage in terms of cracking resistance.

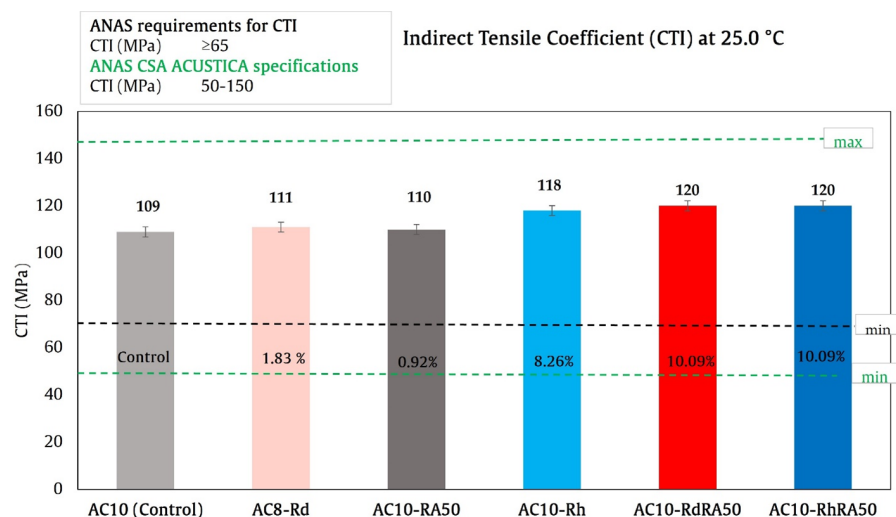


Figure 4.10: Indirect tensile coefficient analysis of conventional and alternative paving materials

4.2.5 Conclusion and recommendations of section-I

This section provides comprehensive conclusions and recommendations on the QA/QC testing approach utilized by Italian road authorities for conventional and APMs. As a response to the research objective RO-I of this research study i-e "To assess whether conventional characterisation of paving materials, according to the current Italian specifications (including both QA/QC and service life), can discriminate between performances of conventional and APMs". From table 4.2, it can be observed that as a comparison between the conventional and APMs in terms of mechanical characterization showed a minimal difference that is less than 20 % for each mixture. Also, the mixtures between their respective category did not show any remarkable difference. Therefore, it can be concluded that, current Italian specifications (including both QA/QC and service life), are unable to discriminate between performances of conventional and APMs. As a recommendation it is suggested that more pragmatic approach in terms of damage-related characterization should be applied for the discrimination of conventional and APMs.

Table 4.2: QA/QC testing for conventional and APM's by the Italian road authorities

Mixture ID	No of gyrations	Void Content (%)	Void Content Range (%)	Diff (%)	ITS at N210 (MPa)	Range (MPa)	Diff (%)	ITS 5 % (AV)	N210 VS 5%	CTI (MPa)	Range (MPa)	Diff (%)
AC10 (Control)	N10	14.9	11-15	Control			Control	1.38	2.8	109		Control
	N120	6	3-6									
	N210	5.5	≥2		1.42							
AC8-Rd	N10	11.9	11-15	20			6.34	1.46	3.3	111		1.83
	N120	4	3-6	16								
	N210	2.6	≥2	18	1.51							
AC10 -Rh	N10	13.7	11-15	8		ANAS	11.27	1.53	3.16	118	ANAS	8.26
	N120	4.5	3-6	25		0.72-1.60					≥65	
	N210	3.6	≥2	34	1.58							
AC10-RA50	N10	13.1	11-15	12		ANAS CSA	9.86	1.51	3.2	110	ANAS CSA	0.92
	N120	4.2	3-6	30		≥0.6					50-150	
	N210	2.4	≥2	56	1.56							
AC10-RdRA50	N10	14	11-15	6			16.9	1.59	4.21	120		10
	N120	5	3-6	9								
	N210	2.9	≥2	11	1.66							
AC10-RhA50	N10	11.9	11-15	20			16.9	1.57	5.4	120		10
	N120	4.4	3-6	27								
	N210	2.5	≥2	4	1.66							

4.3 Section-II

4.3.1 Analysis using basic level approach for damage related characterization

This section starts from the summary shown in table 2.3 that describes an analysis conducted for Italian road authorities using QA/QC approach to evaluate APMs. The findings indicated that the QA/QC lab testing methods by the Italian road authorities were not effective in distinguishing between conventional (traditional) and APMs. To address this limitation, the research proposes a "basic level approach," which is designed based on performance evaluation and damage-related characterization.

4.3.1.1 Key features of the basic-level approach:

The basic-level approach focuses on major principle distresses in flexible pavements. This approach targets primary issues or "distresses" that commonly lead to pavement failures, such as cracking, moisture damage and rutting (depressions formed by traffic) [167, 168, 169]. Recognizing that not all laboratories have the budget or resources for advanced testing, the basic approach is tailored to be both affordable and feasible for labs with limited funding.

4.3.1.2 Testing Methodologies:

The testing methodology utilized to quantify damage-related characterization has been elaborated on in detail in chapter 3. For the sake of understanding the summary of the testing methodology for basic level is discussed below.

4.3.1.3 Volumetric characterization:

(5.0% target air voids study)

As summarized in table 4.2, no clear difference observed in ITS testing between the energy compaction (N210) gyration in QA/QC followed by the Italian road authorities testing [2] and 5.0 % air voids have been observed. Therefore the basic level damage-related characterization approach considers the 5.0 % air voids study [154]. The study examined the air voids within the conventional and APMs aiming for a target of 5.0 % to ensure consistent volume and mixture quality.

4.3.1.4 Cracking Resistance (Ideal-CT testing):

The method assesses the ability of the mixture to resist cracking under stress, especially over time. The conventional and APMs mixtures have been tested to quantify the cracking tolerance using ASTM D8225-19 for Ideal-CT testing [170]. The minimum threshold values for Ideal-CT testing are summarized in table 4.3

4.3.1.5 Rutting Resistance (HT-IDT testing):

This test measures the mixture's resilience against rutting especially at high temperature, which can compromise road safety and performance. The conventional and APMs mixtures have been tested at 54 °C to quantify the rutting resistance using ALDOT-458 specifications for HT-IDT testing. [8, 171]. The minimum threshold values for HT-IDT testing are summarized in table 4.3

4.3.1.6 Moisture resistance (ITSR testing):

This test evaluates how well the mixture can withstand moisture, which is crucial for durability, particularly in wet climates [172]. The conventional and APMs mixtures have been tested at 25 °C to quantify the moisture resistance using EN 12697-12 specifications for ITSR testing

4.3.1.7 Summary of thresholds for damage-related characterization:

The testing methods and the set thresholds for conventional and APMs are organized in a summary table 4.3, allowing for easy comparison of results across the different mixtures. This basic-level approach for damage-related characterization is designed to be practical and effective, especially for labs with budget constraints, providing essential performance evaluations for both conventional and alternative paving materials while remaining accessible and cost-efficient.

Table 4.3: Summary of thresholds for basic-level characterization [8, 16, 7]

Property	Standard(s)	Parameter	Testing conditions	Performance evaluation
Cracking resistance	ASTM D8225-19	IDEAL CT - CT _{index}	Temperature 25°C, Vertical monotonic load 50 mm/min	CT _{index} ≥ 80
	EN 12697-23	Indirect tensile strength - ITS	ITS ≥ 0.9 MPa	
Moisture sensitivity	EN 12697-12	Indirect tensile strength ratio - ITSR	Testing Temperature 25°C. Conditioning in a water bath at (40 ± 1) °C for 72 hours	ITSR ≥ 80
Rutting resistance	ALDOT-458	HT-IDT (MPa)	Temperature 54°C	≥0.44: Excellent 0.32-0.44: Good 0.20-0.32: Fair ≤ 0.20: Poor

4.3.2 Basic-level approach for conventional and rubberized asphalt mixtures

The conventional mix AC10 (control) and the rubberized asphalt mixtures AC8-Rd and AC10-Rh are tested for three key performance criteria using the basic-level approach to quantify damage-related characterization. These criteria include cracking resistance by Ideal-CT testing, which is evaluated using two standard tests, ASTM D8225-19 and EN 12697-23 summarized in 4.3. These tests measure the ability of the conventional and rubberized asphalt mixtures to resist cracking under various loads and temperature fluctuations, which is

critical for long-term durability [170]. The resistance to rutting by HT-IDT testing assessed with ALDOT-458, evaluates the conventional and rubberized mixtures resistance to permanent deformation (rutting) under repeated loading and extreme temperature conditions. Higher rutting resistance indicates better performance under heavy traffic [62]. Finally, the moisture Sensitivity by ITSR testing was evaluated with EN 12697-12. This test assesses how the mixtures respond to moisture, which can cause stripping (loss of adhesion between asphalt binder and aggregate) and degrade the mixture's structure [16].

The purpose of analyzing these results is to determine how conventional and rubberized asphalt mixtures perform under stress, providing insights into their suitability for various environmental and traffic conditions [173]. By comparing the results, the in section 4.3.3 aims to conclude whether rubberized mixtures offer significant improvements over conventional asphalt.

Each performance test provides data on specific damage-related characterization that impact asphalt lifespan, maintenance frequency, and cost-effectiveness. By choosing the optimal mixture based on the results, the road authorities can design pavements that reduce cracking, resist rutting, and withstand moisture exposure, ultimately leading to more resilient and cost-efficient infrastructure. Based on the results from these tests, recommendations for asphalt mixture usage in various climates and traffic conditions could be made. For example, regions with heavy traffic might prioritize rutting resistance, while areas with frequent rainfall might emphasize moisture sensitivity [174, 175].

In summary, this describes a structured and more pragmatic approach testing approach termed as “basic-level” based on damage-related characterization to assess the performance of different asphalt mixtures. It involves analyzing cracking, rutting, and moisture resistance to determine the most effective mix, which could lead to practical recommendations for infrastructure projects based on durability needs. The minimum bench mark values for each test are summarized in table 4.3

4.3.2.1 Cracking resistance analysis for conventional and rubberized mixtures

The figure 4.11 represents the CT-Index for the asphalt mixtures at 5.0 % voids, comparing AC10 (control) with AC8-Rd and AC10-Rh mixtures. These values indicate a progression from AC10 (control) to AC8-Rd, with AC10-Rh having the best performance based on the CT-Index, which suggests improved resistance to cracking. All three mixtures (AC10 (control), AC8-Rd and AC10-Rh) exceed this minimum requirement by a significant margin, indicating that they all meet or exceed the expected performance threshold [7]. The AC10-Rh mixture, with a CT-Index of 181, shows considerably higher resistance to cracking compared to the other mixtures shown in figure 4.11. This implies it would provide superior durability [176, 177].

The AC8-Rd mixture performs moderately well, better than AC10 (control) in terms of CT-Index. The AC8-Rd mixture, with a CT- Index of 139 shown in figure 4.11, provides better fracture energy absorption and higher

post-peak stability than AC10 (control) [173]. This means it is more resilient under repeated loading conditions, such as from vehicles, making it less prone to crack propagation over time.

The AC10-Rh mixture, with a CT-Index of 181, has the highest cracking resistance besides all three mixtures. This is technically significant because a higher CT-Index is indicative of a more ductile material with improved tensile strength [178]. It suggests that AC10-Rh can sustain higher strain energy before initiating cracks and that cracks, once initiated, would propagate more slowly due to better energy dissipation characteristics.

The AC10 (Control) mixture also performs well above the minimum requirement but not as effectively as the other two rubberized mixtures. The minimum threshold of 80, marked by Texas's approach, ensures that each mixture meets a basic level of cracking resistance [179].

In summary, if long-term performance and maintenance reduction are key goals, AC10-Rh would be the optimal choice for construction in terms of cracking resistance. But there is a still need of analysis for further damage characterization in terms of rutting and moisture sensitivity

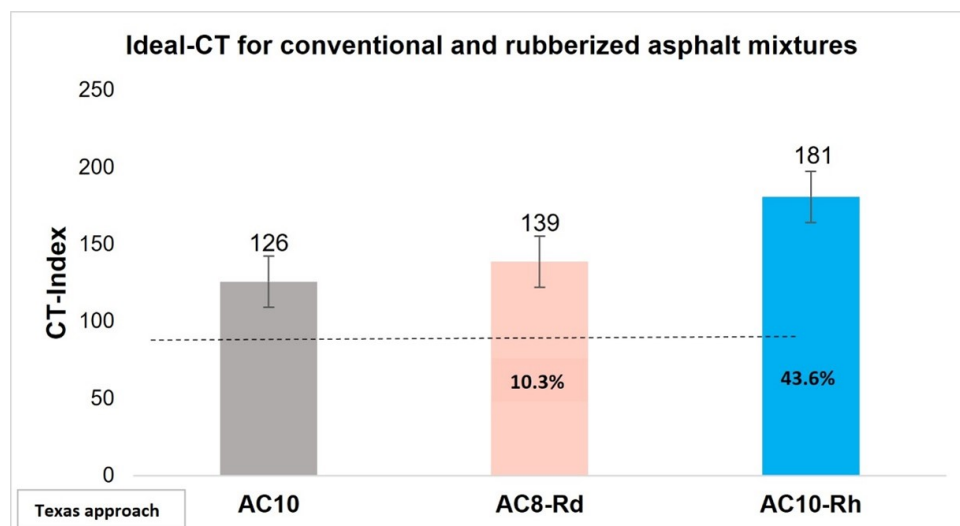


Figure 4.11: Cracking tolerance of conventional and rubberized asphalt mixes

A high CT-Index, such as that of AC10-Rh (181), not only resists the initiation of cracks but also significantly slows the propagation of those cracks once they begin. This attribute is essential for pavement performance because slower crack growth directly translates to less frequent maintenance and longer service life [176]. The AC8-Rd mixture, with a CT-Index of 139, shows improved resistance to both crack initiation and propagation compared to AC10 (control). It is effective in delaying crack growth, making it suitable for moderately demanding conditions. The AC10 (control) mixture has a lower CT-Index (126), suggesting that once cracks form, they may propagate faster compared to the other two mixtures. This faster propagation could lead to accelerated pavement deterioration if not monitored and maintained regularly

4.3.2.2 Rutting resistance analysis for conventional and rubberized mixtures

The figure 4.12, provides insight into the rutting resistance of three asphalt mixtures (AC10 (control), AC8-Rd, and AC10-Rh) at a high temperature of 55 °C, evaluated through their ITS values in MPa [180]. For AC10 (Control) the ITS value is 0.25 MPa, which classifies it as Fair ($0.20 \leq \text{ITS} < 0.32$ MPa). This implies that while it can handle moderate traffic loads (3 to 10 million ESALs), it may experience rutting under high traffic and elevated temperature conditions [8].

AC8-Rd mixture has an ITS of 0.3 MPa, also in the fair category but closer to the good threshold. The slight increase over the control indicates some improvement in rutting resistance. However, this mixture might still not be optimal for heavy-duty roads.

AC10-Rh with an ITS of 0.34 MPa, AC10-Rh achieves a good rating ($0.32 \leq \text{ITS} \leq 0.44$ MPa). This suggests that AC10-Rh can handle higher traffic loads (10 to 30 million ESALs) with better resistance to rutting under stress and elevated temperatures, making it suitable for more demanding applications. AC10-Rh is the only mixture to reach the good category, indicating a higher rutting resistance than AC10 (Control) and AC8-Rd [8]. This mixture is suitable for moderate to high traffic volumes, potentially leading to a longer pavement lifespan in heavy-use areas.

AC10 (control) and AC8-Rd fall within the Fair category, which limits their application to areas with lighter or medium traffic. This lower classification suggests that these mixtures may experience deformation and rutting sooner under repeated loading and high temperatures [8]. For moderate traffic, both AC10 (control) and AC8-Rd might be cost-effective choices for roads with moderate traffic loads. However, if a slight improvement in rutting resistance is needed, AC8-Rd could be preferable over the AC10 (control). For high-traffic areas, AC10-Rh stands out as the best choice due to its good classification, meaning it can handle more substantial loading and resist rutting effectively. Its higher ITS value translates to greater durability and stability under heavy traffic, justifying its potential higher cost.

4.3.2.3 CTI analysis of conventional and rubberized asphalt mixes

The figure 4.13, illustrates the CTI of three mixtures AC10 (control), AC8-Rd, and AC10-Rh measured at 55.0°C. AC10 (control) serves as the baseline measurement for tensile performance without any modifications. As the control, it provides a standard against which the other two mixtures are compared. The 22 % increase in CTI suggests that the AC8-Rd mixture has a more robust tensile property under high-temperature conditions compared to the control. This implies that the rubber in AC8-Rd improve its coating's tensile strength at elevated temperatures [8]. With a CTI of 147 Mpa shown in figure 4.13, AC10-Rh exhibits the highest tensile strength at 55.0°C, surpassing the control by 28 %. This improvement suggests that AC10-Rh's composition is particularly effective in enhancing the tensile strength of the mixture, likely due to a specific modification (addition of rubber and polymers) that

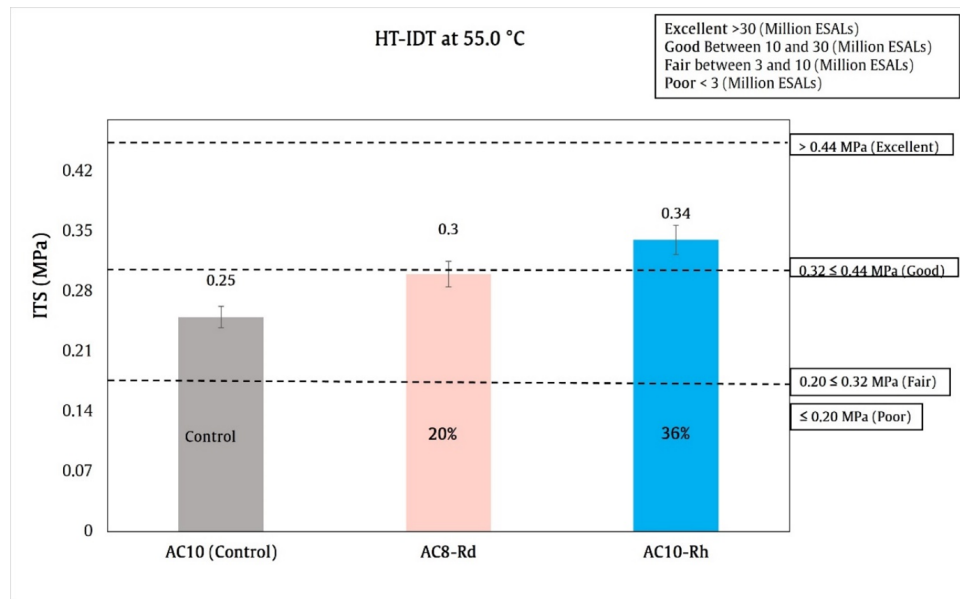


Figure 4.12: Rutting resistance of conventional and rubberized asphalt mixes enhances its structural integrity under heat stress [181].

Both AC8-Rd and AC10-Rh outperform the AC10 (control), indicating that the modifications or additives in these mixtures positively affect CTI at higher temperature. AC10-Rh provides a slightly higher tensile improvement (7 MPa more than AC8-Rd), which could be crucial for applications requiring superior strength at high temperature [8]. The enhanced CTI values suggest that both AC8-Rd and AC10-Rh could be better suited for environments with high-temperature demands, with AC10-Rh potentially offering the best performance

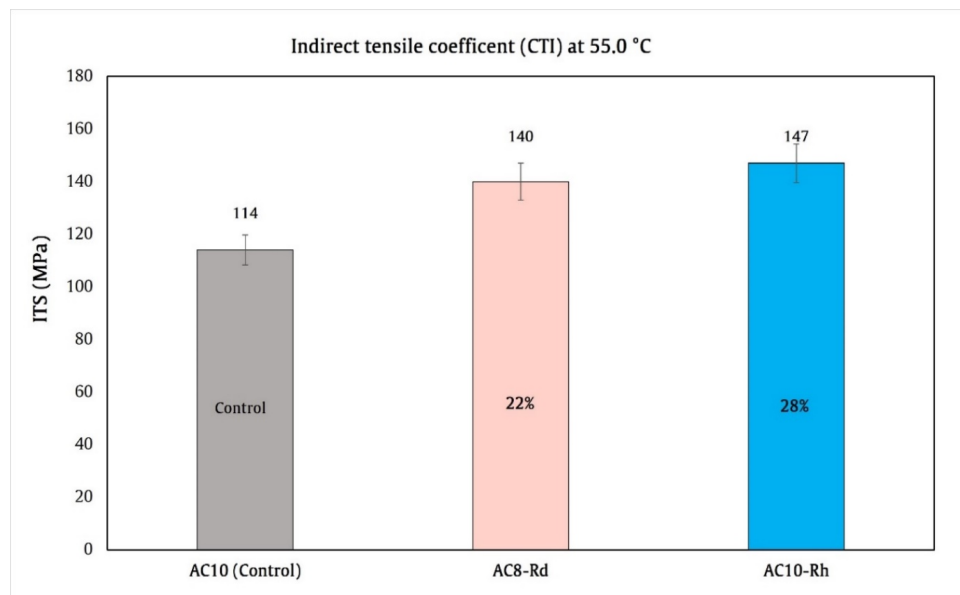


Figure 4.13: Indirect tensile coefficient of conventional and rubberized asphalt mixes

4.3.2.4 Moisture sensitivity analysis for conventional and rubberized mixtures

The figure 4.14 shows the **ITSR** in percentage for three different mixtures, AC10 (control), AC8-Rd, and AC10-Rh. The **ITSR** value for AC10 (Control) is 79.1 %. This value is just below the "Anas limit" black dashed line, which represents a benchmark value for low noise emissions [2]. The AC10 (control) mixture has an **ITSR** slightly lower than the required limit, which may indicate that its moisture resistance is slightly insufficient according to the set standard. This could suggest potential issues with durability under wet conditions.

The AC8-Rd is slightly above the reference line, indicating that the mixture is meeting the minimum requirement for low noise emissions [2]. The AC8-Rd mixture has satisfactory moisture resistance, performing above the threshold. This suggests that it would be more durable under wet conditions compared to AC10 (control), and it complies with the desired moisture sensitivity standard.

The **ITSR** value for AC10-Rh is 86.9 %, the highest among the three mixtures. This value is well above the reference limit [2]. The AC10-Rh mixture has the best moisture resistance performance, indicating high durability in wet conditions. It is significantly above the threshold, making it the most suitable option among the three for areas requiring high moisture resistance and low noise emissions.

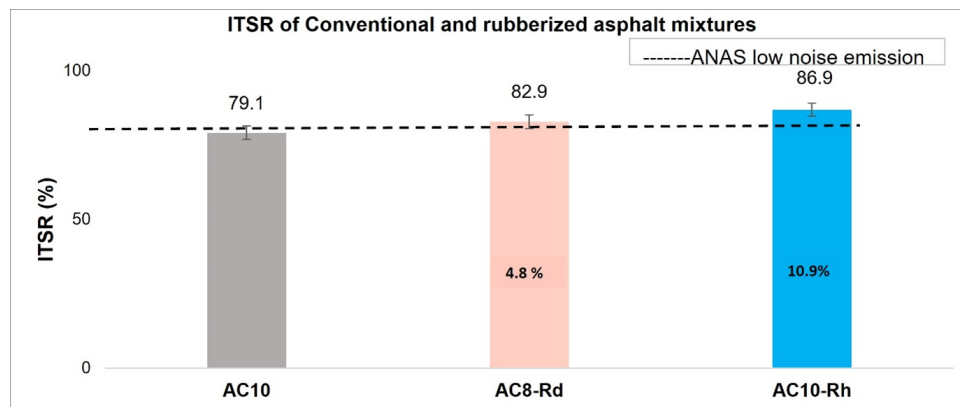


Figure 4.14: Moisture sensitivity of conventional and rubberized asphalt mixes

AC10 (control) falls below the acceptable limit for **ITSR** [2], suggesting that it may not be suitable for applications where moisture resistance and durability are crucial, particularly under wet conditions. AC8-Rd meets the minimum criteria, making it a suitable mixture for general applications. While AC10-Rh performs the best in terms of moisture resistance, well above the threshold. This mixture would be ideal for areas needing better durability, such as roads that experience heavy rainfall or require enhanced resistance to moisture damage.

4.3.3 Conclusions and recommendations of Section-II

From the analysis of cracking, rutting and moisture damage sensitivity and a response to **RO-II** in section 1.2, "To investigate whether a "basic" damage-related characterization of paving materials is able to discriminate between performances of **APMs**.

- Rutting resistance using [HT-IDT](#) testing for the Sicily region.
- Cracking resistance using [Ideal-CT](#) testing.
- Moisture sensitivity using [ITSR](#) testing.

The analysis of these performance indicators reveals that rubberized asphalt mixes, specifically AC10-Rh, consistently outperform the conventional asphalt mix AC10 (control) in all three aspects i.e cracking resistance in terms of CT-index, moisture sensitivity in terms of [ITSR](#), and rutting resistance in terms of HT-ITS. This suggests that incorporating rubber into asphalt significantly enhances its performance, making it a superior choice for road pavements that face varying temperature and moisture conditions and heavy traffic loads.

Rubberized mixes show enhanced resistance to cracking due to improved elasticity. Higher [ITSR](#) values indicate that rubberized mixes are less prone to moisture-induced damage. AC10-Rh demonstrates good resistance to permanent deformation under high temperatures. As a summary shown in table 4.4, of the basic level approach for conventional and rubberized asphalt mixtures AC8-Rd and AC10-Rh exhibit improvement in all performance parameters compared to the conventional mix AC10 (control). AC10-Rh consistently shows the highest percentage of improvement in all categories, highlighting its superior performance in terms of cracking resistance, moisture resistance, and rutting resistance.

This comprehensive performance analysis based on basic damage characterization highlights that rubberized asphalt can provide longer lasting and more resilient road surfaces, justifying its consideration for road before during and after construction projects. This approach also provides a pragmatic experimental program based on conventional and rubberized asphalt mixes that may aid in improved pavement monitoring for more sustainable management of road pavements.

Table 4.4: Basic level approach using damage-related characterization for conventional and rubberized asphalt mixtures

Mixture ID	CT- Index at 5% Voids	Range (MPa)	Diff (%)	ITSR (%)	Range (MPa)	Diff (%)	HT-IDT (MPa)	Range (MPa)	Diff (%)	CTI (MPa)	Diff (%)
AC10 (control)	126	CTindex $\geq$ 80	Control	79.1	ITSR $\geq$ 80	Control	0.25	>0.44: Excellent	Control	114	Control
AC8-Rd	139		10	82.9		4.8	0.30	0.32 $\leq$ 0.44: Good	20	140	22
AC10-Rh	181		43	86.9		10.9	0.34	$\leq$ 0.20: Poor	36	147	28

4.4 Section-III

4.4.1 Basic-level approach for conventional and reclaimed asphalt mixtures

The performance of both conventional asphalt mixture AC10 (control) and various RAP mixes includes (AC10-RA50, AC10-RdRA50, and AC10-RhRA50) taken as APMs were evaluated through a series of standardized tests. In order to investigate RO-II in section 1.2 "To investigate whether a "basic" damage-related characterization of paving materials is able to discriminate between performances of APMs.

- Rutting resistance using HT-IDT testing for the Sicily region.
- Cracking resistance using Ideal-CT testing.
- Moisture sensitivity using ITSr testing.

These mixtures were subjected to cracking resistance assessments as per ASTM D8225-19 and EN 12697-23 standards, rutting resistance testing in accordance with ALDOT-458, and moisture sensitivity evaluation following EN 12697-12 guidelines [182, 183, 184]. The subsequent section presents a detailed analysis of the results obtained from these tests, comparing the conventional asphalt mixture to those incorporating RAP mixtures. The threshold values for each test followed the summarized values in table 4.3. This comprehensive evaluation aims to highlight the strengths and weaknesses of reclaimed materials in terms of cracking, deformation, and moisture durability, offering insights into their suitability for practical application.

4.4.1.1 Cracking resistance analysis for conventional and reclaimed mixtures

The cracking resistance for the conventional and RAP mixtures based on the Ideal-CT test has been summarized in figure 4.15. The CT-Index value for AC10 (control) is 126, which is the highest among all the mixtures [185]. This value serves as the benchmark for comparing cracking resistance. The CT-Index value for AC10-RA50 is 112, showing an 11 % reduction in cracking resistance compared to the conventional mixture. This reduction suggests that the use of 50 % reclaimed asphalt without any performance-enhancing additives compromises cracking resistance to some extent [185]. The CT-Index for AC10-RdRA50 is 121, representing a 4 % decrease in cracking resistance compared to the control.

This suggests that the addition of rubber helps mitigate the loss in cracking resistance and maintains performance relatively close to the conventional mixture. The CT-Index for AC10-RhRA50 is 117, indicating a 7 % decrease in cracking resistance relative to the control mixture. The combination of rubber and polymer improves the cracking resistance compared to AC10-RA50, but it is slightly less effective than using rubber alone (AC10-RdRA50). The conventional mixture (AC10 Control) provides the highest cracking resistance (CT-Index = 126). Among the reclaimed mixtures, AC10-RdRA50 (with rubber) shows the closest performance to the control, with only a 4 % reduction in cracking resistance, indicating that rubber is effective in enhancing cracking resistance. The

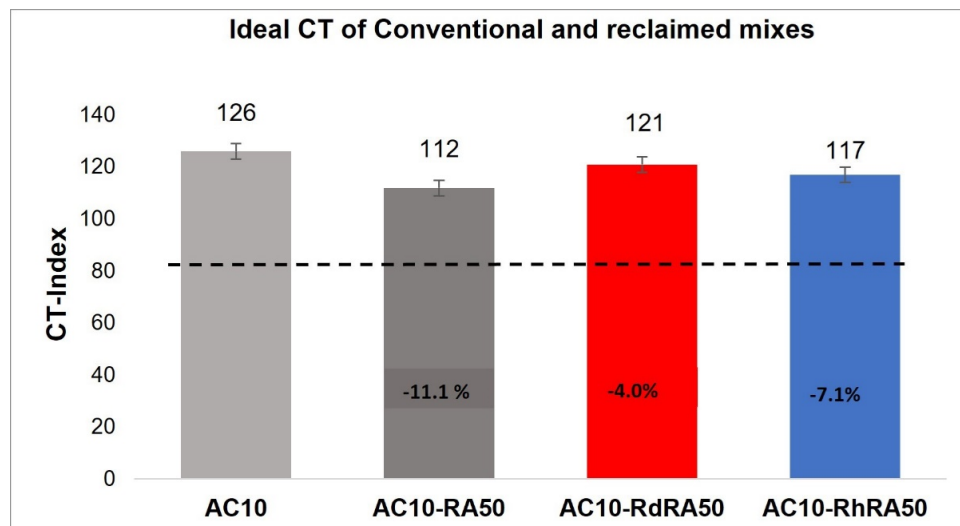


Figure 4.15: Cracking resistance of conventional and reclaimed asphalt mixes

AC10-RhRA50 mixture (with rubber and polymer) has a 7 % reduction in cracking resistance compared to the control, which is better than the unmodified reclaimed mixture but slightly worse than the rubber-only mixture [24]. The AC10-RA50 mixture, containing only reclaimed asphalt without modifiers, shows the greatest reduction in cracking resistance, with an 11 % decrease.

Overall, the addition of rubber or rubber combined with polymer effectively improves the cracking resistance of reclaimed asphalt mixtures, with rubber alone performing slightly better

4.4.1.2 Rutting resistance analysis for conventional and reclaimed mixtures

The figure 4.16 summarizes the rutting resistance for conventional and RAP mixes using HT-IDT testing. AC10 (control) has the ITS value of 0.25 MPa, which falls in the fair range [186]. This indicates that the control mix has moderate resistance to rutting. AC10-RA50 mixture has an ITS value of 0.28 MPa, still in the fair range but showing an improvement (12 %) compared to the AC10 (control) mix. This suggests that adding RAP slightly enhances rutting resistance [164]. AC10-RdRA50 has an ITS value of 0.33 MPa, which falls in the good range [164]. This mixture demonstrates a significant improvement (32 %) compared to the AC10 (control) mix, indicating better rutting resistance. AC10-RhRA50 has an ITS value of 0.37 MPa, also in the good range, with the highest improvement (48 %). This mixture has the highest ITS value among the four, suggesting it provides the best rutting resistance [164].

AC10-RdRA50 and AC10-RhRA50 show substantial improvements in high temperature ITS, which indicates better performance in resisting rutting under high temperature. The addition of RAP with rubber, along with rubber and polymers have progressively improved the performance, with AC10-RhRA50 achieving the highest improvement.

This could mean that the specific additives or modifiers used in AC10-RhRA50 are more effective in enhancing the resistance to deformation. None of the mixtures achieved an **ITS** value high enough to be classified as excellent (≥ 0.44 MPa). The AC10-RhRA50 and AC10-RdRA50 mixtures fall in the good range, which suggests they are suitable for applications where moderate to high rutting resistance is required [8].

AC10-RhRA50 is the most promising mixture among those tested, as it exhibits the highest **ITS** value and falls in the good category [8].

As a conclusion for conventional and **RAP** mixes, the high traffic areas or regions with significant temperature variations, the AC10-RhRA50 mixture would likely perform better in terms of rutting resistance.

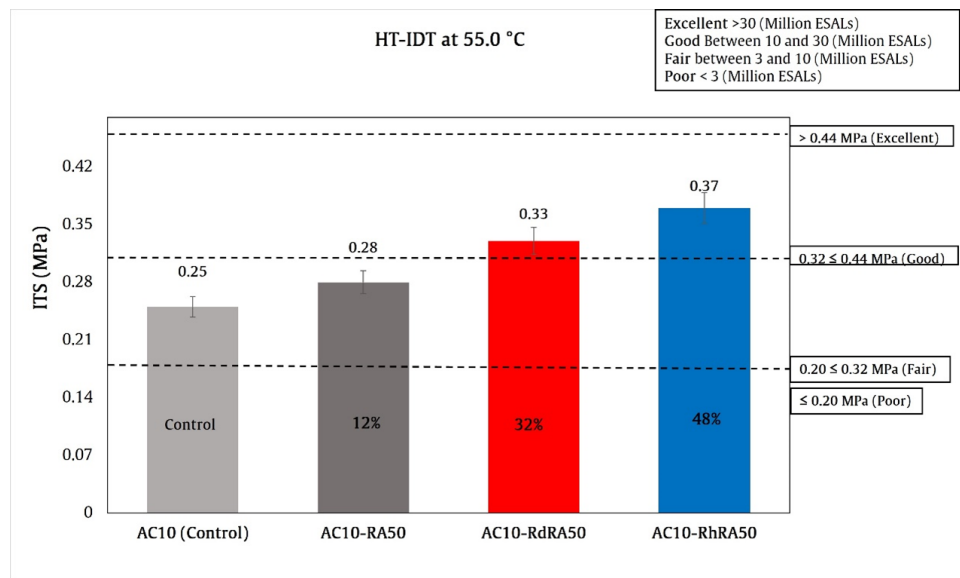


Figure 4.16: Rutting resistance of conventional and reclaimed asphalt mixes

4.4.1.3 Indirect Tensile Coefficient analysis of conventional and reclaimed mixes

The **CTI** measures the tensile strength of conventional and **RAP** under high temperature **ITS** conditions, indicating how well it can resist permanent deformation [8].

A higher **CTI** value suggests greater resistance to tensile failure, which is particularly important for reclaimed mixes as it indicates their viability compared to conventional asphalt [187]. The figure 4.17 compares the **CTI** of conventional and **RAP** mixes. From figure 4.17 it can be noted that the AC10 (control) has **CTI** value of 114 MPa, representing the baseline performance of a conventional asphalt mixes [8]. While AC10-RA50 **CTI** value of 132 MPa, which shows an increase of 15 % compared to the control mix.

This improvement suggests that incorporating 50 % **RAP** has a positive effect on the mixture's tensile strength, making it more resistant to deformation. AC10-RdRA50, **CTI** value of 150 MPa, with an improvement of 31 % over the control mix. This indicates that the addition of rubber with **RAP** significantly improves the tensile properties beyond that of just reclaimed asphalt mixes. For AC10-RhRA50 the **CTI** value of 170 MPa, the highest among all the mixes, with a 49 % increase compared to the control mixes.

The incorporation of hybrid materials (rubber and polymers) with RAP leads to superior performance, suggesting it is the most promising modification for enhancing the tensile strength of RAP mixes. AC10-RA50 already outperforms the AC10 (control) mix, proving that RAP materials can offer viable strength when processed correctly. AC10-RdRA50 and AC10-RhRA50 show progressive increases in CTI, indicating that RAP with modifiers are crucial for optimizing performance and ensuring the reclaimed mix matches or exceeds the capabilities of conventional asphalt.

As a conclusion for rutting resistance of conventional and RAP mixes, the figure 4.17 indicates that adding RAP and further enhancing the mix with specific additives like (rubber and polymer contents) can significantly improve the tensile strength properties of the mixtures. AC10-RhRA50 is the top performer, suggesting that this mix has the highest tensile resistance and is a suitable replacement for conventional mixtures, especially in applications requiring high durability and cracking resistance [188]. The use of reclaimed materials is validated here, as it shows that recycled content, when properly treated and enhanced, can provide comparable or superior performance to traditional asphalt.

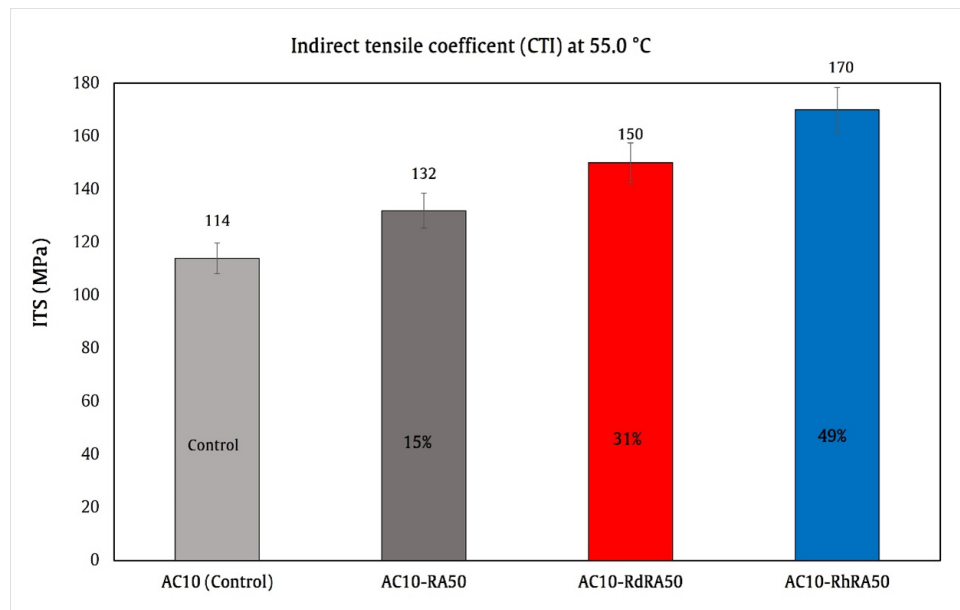


Figure 4.17: Indirect Tensile coefficient of conventional and reclaimed asphalt mixes

4.4.1.4 Moisture sensitivity analysis for conventional and reclaimed mixtures

The moisture sensitivity analysis for conventional and RAP mixes has been summarized in figure 4.18. *ITSR* is an important measure of the moisture susceptibility of asphalt mixtures. A higher *ITSR* value indicates better resistance to moisture damage, meaning the asphalt retains more of its original strength when exposed to moisture [189]. The ANAS Italian road specifications limit of 80 % acts as a benchmark for minimum acceptable moisture resistance [2]. Mixtures that exceed this limit are considered acceptable in terms of resistance to moisture-induced

damage.

As it is evident from in figure 4.18 that AC10 (control), has an **ITSR** value of 79.1 %, which falls below the **ANAS** limit of 80 % [2]. This suggests that the control mixture has marginal moisture resistance and does not meet the required standard for acceptable performance. For AC10-RA50 **ITSR** value of 78 %, which is also below the **ANAS** limit [2]. This means that incorporating 50 % **RAP** mixture has further reduced the moisture resistance compared to the control mix. The improvement percentage is 1.39 %, which represents a decrease from the control mixture. This indicates that **RAP** without any modification does not provide sufficient moisture resistance.

For AC10-RdRA50 **ITSR** value of 81.8 %, which is above the **ANAS** limit [2]. This mixture shows a 3.40 % improvement over the control mix, indicating that the addition of rubber with **RAP** has effectively enhanced the moisture resistance, making it suitable for practical use. For AC10-RhRA50 **ITSR** value of 82.5 %, which is also above the **ANAS** limit. This indicates the best performance among the mixtures, with a 4.30 % improvement. The hybrid addition with **RAP** additive clearly has the most significant effect in improving moisture resistance, making this mixture the most effective option [190]. AC10 (control) and AC10-RA50 both fall below the **ANAS** limit of 80 %, indicating that neither the conventional mix nor the **RAP** mix without modification meets the minimum moisture resistance requirements [2]. AC10-RdRA50 and AC10-RhRA50 both exceed the **ANAS** limit, meaning that the addition of specific modifiers makes these modified **RAP** mixtures suitable for practical use.

This comparison highlights the need for modification when using **RAP** to ensure that the mixture meets moisture resistance standards. The figure 4.18 demonstrates that adding rubber and rubber with polymers to the mixtures containing **RAP** significantly improves **ITSR**.

The analysis highlights the necessity of rubberized and polymers additives when using **RAP** to meet durability and moisture resistance standards.

4.4.2 Conclusions and recommendations of Section-III

As a summary of basic-level approach for damage-related characterization of conventional and **RAP** mixes with rubber and polymer modifiers as **APMs** in table 4.5, it can be concluded that the rubberized hybrid mixture with **RAP** addition showed significantly better resistance to rutting and minimal improvement in moisture sensitivity compared to both the performance of the conventional and **RAP** only mixes. However there is a reduction in cracking resistance may be due to the **RAP** addition that stiffen the mixtures. As a response to **RO-II**, in section 1.2 "To investigate whether a "basic" damage-related characterization of paving materials is able to discriminate between performances of **APMs**".

- Rutting resistance using **HT-IDT** testing for Sicily region.
- Cracking resistance using **Ideal-CT** testing.

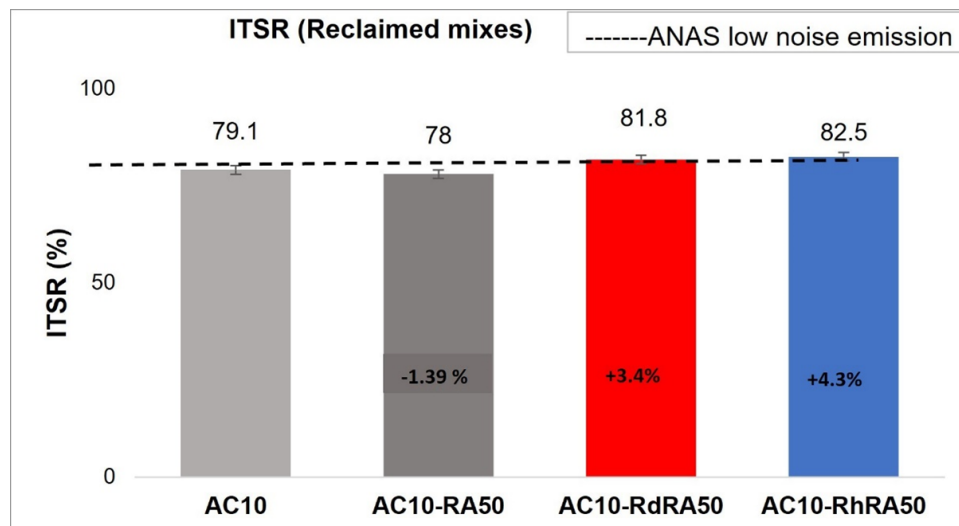


Figure 4.18: Moisture sensitivity analysis of conventional and reclaimed mixes

- Moisture sensitivity using **ITSR** testing.

The results from the section 4.4 for the conventional and **RAP** mixes with hybrid addition of (rubber+polymers) demonstrate key differences in rutting behavior and minimal difference in moisture sensitivity across the different mixtures.

However, the cracking resistance for all the **RAP** mixtures including the rubber and polymers addition did not enhance the flexibility of the mixtures. Thus there is a decrease in improving its resistance to crack initiation and propagation.

The rubberized hybrid with **RAP** mixes exhibited superior rutting resistance in comparison to the conventional mixtures. The addition of **RAP** with rubber and polymer particles contributed to increased stiffness, effectively reducing permanent deformation under load. The rubberized and rubberized hybrid mix with **RAP** addition demonstrated lower improvement in moisture sensitivity. The addition of rubber appears to provide slight better resistance to moisture penetration, likely due to less improved adhesion and binder elasticity because of **RAP** addition. As a result, the less reduction of the mix's vulnerability to stripping is observed.

As a recommendation, both rubberized and hybrid rubberized mixes with addition of **RAP** offer significant performance advantages over the conventional mix, particularly in terms of rutting. However a slight improvement for both the mixtures is also observed in terms of moisture sensitivity. However, there is a decrease in performance in terms of cracking resistance.

The basic-level testing approach for the damage-related characterization can discriminate between conventional and **APMs**. However, to optimize long-term performance, it is also recommended to further investigate these mixes for cracking and moisture resistance for their widespread adoption .

As a final recommendation to the practitioners for the utilization of Basic-Level approach for damage-related characterization for conventional and **RAP** mixes with rubber and polymer additives It is recommended to use the

rubberized hybrid mix with reclaimed mix in regions or projects where rutting related issues are prominent. The improved performance characteristics make it an ideal choice for extending pavement life and reducing maintenance needs.

Table 4.5: Basic level approach using damage-related characterization for conventional and reclaimed asphalt mixtures

Mixture ID	CT Index at 5% Voids	Range (MPa)	Difference (%)	ITSR (%)	Range (MPa)	Difference (%)	HT-IDT (MPa)	Range (MPa)	Difference (%)	CTI (MPa)	Difference (%)
AC10 (control)	126	CT-index ≥ 80	Control	79.1	ITSR ≥ 80	Control	0.25	>0.44: Excellent 0.32 $\leq$ 0.44: Good $\leq$ 0.20: Poor	Control	114	Control
AC10-RA50	112		-11	78		-1.39	0.28		12	132	15
AC10-RdRA50	121		-4	81.8		3.4	0.33		32	150	31
AC10-RhRA50	117		-7	82.5		4.3	0.37		48	170	49

4.5 Section-IV

4.5.1 Analysis using Advanced level approach for damage-related Characterization

Advanced damage-related characterization of the conventional and was conducted using the PASSFlex system, which involves a systematic testing approach with an [AMPT](#). This methodology was applied to both conventional and [APMs](#) to assess their performance. In section IV the rubberized asphalt mixtures are considered as [APMs](#). The laboratory testing phase consisted of several key tests using an [AMPT](#).

Please note that this section highlights a summary of advanced-level methodology. The complete details of each testing method are provided in chapter 3 of this research work.

4.5.1.1 Dynamic Modulus Test

It evaluates the stiffness of the asphalt mixture under varying temperatures and loading frequencies, providing insight into its viscoelastic properties. The AASHTO T 378-22 was utilized to evaluate the stiffness of conventional and rubberized asphalt mixtures [[191](#), [10](#), [192](#)].

4.5.1.2 Cyclic fatigue test

It determines the fatigue life of the conventional and rubberized asphalt mixtures by simulating repeated loading to measure its resistance to cracking over time. The cyclic fatigue utilizes AASHTO TP 133 and [SVECD](#) theory to quantify the damage in asphalt mixtures [[88](#), [193](#)].

4.5.1.3 Stress Sweep rutting test

The [SSR](#) test assesses the rutting potential of conventional and rubberized asphalt mixtures by examining the permanent deformation of the mixture under cyclic loads [[67](#), [194](#)]. The [SSR](#) test utilizes AASHTO TP 34-19 and the [shift](#) model to quantify the viscoplastic behavior of the conventional and rubberized asphalt mixtures [[194](#)].

4.5.2 Damage modeling

After extensive laboratory testing using an [AMPT](#), the data for each mixture was processed using the excel based [FlexMATTM cracking](#) and [FlexMATTM rutting](#) tools for damage quantification. Several advanced modeling techniques were utilized to analyze the viscoelastoplastic aspects of the conventional and rubberized asphalt mixtures [[134](#)].

4.5.2.1 2s2p1D Model

To assess the stiffness characteristics of conventional and rubberized asphalt mixtures, accounted for viscoelastic properties, the 2s2p1D model coded in [FlexMATTM cracking](#) was utilized [10].

4.5.2.2 SVECD Model

The [SVECD](#) model employed in the excel based [FlexMATTM cracking](#) to evaluate fatigue cracking of conventional and rubberized asphalt mixes by considering the material's damage response under repeated stress [88, 89].

4.5.2.3 CTC Model

The [CTC](#) model analyzed thermal of the conventional and rubberized asphalt mixture's contraction to determine the material's behavior in response to temperature changes and thermal-induced stress [13].

4.5.2.4 Shift model

For rutting evaluation, the [shift](#) model was analyzed by FlexMAT rutting tool helping to determine how the material's deformation characteristics change over its lifespan [194]. Finally, after successful data analysis, the results were used in a pavement simulation step, conducted through the FlexPAVE simulation program [145, 133]. This stage enabled the performance assessment and structural design of the pavement, providing insights for both current and future performance under various conditions. Overall, this section provides a comprehensive analysis of an advanced methodology for damage-related characterization, comparing both conventional and alternative paving mixtures by leveraging sophisticated modeling and simulation tools.

4.5.3 Data quality indicators for dynamic modulus analysis

The dynamic modulus testing conducted on both conventional and alternative paving mixtures was subject to specific Data Quality Indicators Data Quality Indicators ([DQI's](#)), as outlined in AASHTO T 378-22 [192]. These DQIs are essential for ensuring the reliability and validity of the test results, as they help identify any inconsistencies or errors in the data before proceeding with further analysis [195]. Before generating the master curve using the FlexMAT cracking tool, it is crucial to carefully assess the limitations associated with each tested sample. Each sample, which represents the paving mixture, must be evaluated for adherence to quality standards, considering factors such as variability in properties, potential inconsistencies in material composition, and any deviations that could affect the overall accuracy of the master curve analysis [10, 192]. By adhering to these data quality indicators, it is possible to ensure that the resulting master curve accurately represents the performance characteristics of the

paving mixtures under various conditions, ultimately leading to a more reliable assessment of their durability and suitability for use in pavement design and construction.

4.5.3.1 Load standard error analysis

Figure 4.19 shows the Load Standard error (%) for different mixtures tested at varying frequencies (10 Hz, 1.0 Hz, 0.1 Hz) and temperatures (4.0°C, 20.0°C, 40.0°C) [192]. Higher temperatures generally lead to higher error values across most mixtures, which is typical as the material behavior can become less predictable. Most mixtures meet the 10 % error limit, but there are some points where error values are relatively high (e.g., 7.7 % for AC10-RhRA50 at 40.0°C). The AC10-Rh and AC10-RA50 mixtures show some higher errors at lower temperatures, indicating possible variability in the material or testing conditions. Since DQI's suggests an acceptable threshold of 10 %, samples with values nearing this limit should be examined closely for any deviations or anomalies in the testing procedure or material quality [192]. This analysis provides insights into the data reliability for each mixture under different testing conditions, enabling better decision-making for further processing and pavement simulation.

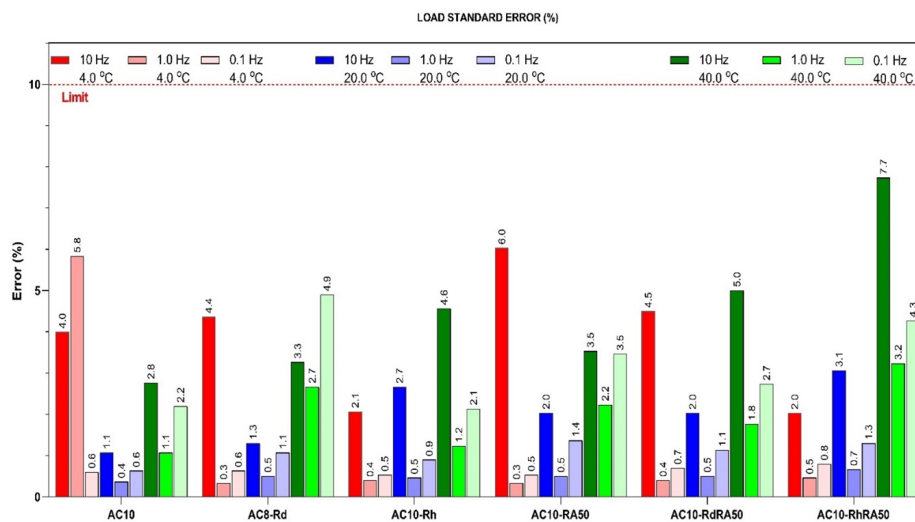


Figure 4.19: Load standard error analysis of conventional and alternative paving materials

4.5.3.2 Deformation standard error analysis

Figure 4.20 summarizes the deformation standard error (%) for conventional and alternative paving materials tested at varying frequencies (10 Hz, 1.0 Hz, 0.1 Hz) and temperatures (4.0°C, 20.0°C, 40.0°C) [192]. The deformation standard error is generally associated with positioning of the sample inside the chamber of AMPT. The main cause as per AASHTO T 378-22 standard can be the attachment of loose gauge points, damaged LVDTs or a non-sinusoidal load. Most mixtures meet the 10 % error limit, but there are some points where error values are relatively high (e.g., 9.9 % for AC10-RhRA50 at 40.0°C). The AC10, AC10-RdRA50 and AC10-RhRA50 mixtures show some higher errors at lower temperatures, indicating possible variability in the material or testing conditions. Since DQI's suggests an acceptable threshold of 10 %, samples with values nearing this limit should be examined

closely for any deviations or anomalies in the testing procedure or material quality [192]. This analysis provides insights into the data reliability for each mixture under different testing conditions, enabling better decision-making for further processing and pavement simulation.

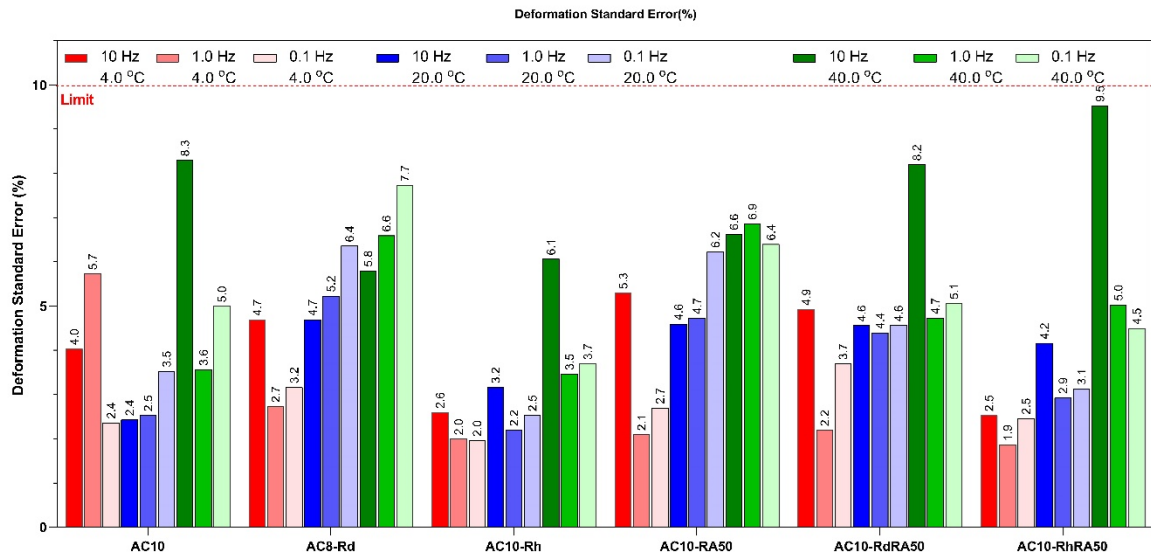


Figure 4.20: Deformation standard error analysis of conventional and alternative paving materials

4.5.3.3 Deformation uniformity analysis

The deformation uniformity is represented for different conventional and alternative paving materials shown in figure 4.21. The limit for acceptable deformation uniformity appears to be around 30 %, which is marked by the red dashed line. The deformation uniformity percentages vary across different frequencies (10 Hz, 1 Hz, 0.1 Hz) and temperatures (4°C, 20°C, 40°C) [192]. The highest deformation uniformity values are observed for AC10 and AC10-RhRA50, especially at 10 Hz and 4°C. The values here are close to or exceed 20 %, which suggests that eccentric loading may have affected these samples, causing uneven stress distribution. Eccentric loading often results in higher deformation non-uniformity. AC10-RhRA50 and AC10-RA50 show higher deformation uniformity values at lower frequencies (0.1 Hz) compared to other mixes, especially at 4°C and 20°C [10, 192].

This inconsistency may suggest the possibility of loose gauge points, leading to less precise strain measurements. Inconsistent deformation uniformity across temperatures and frequencies for AC10 and AC10-RA50 might also indicate issues with sample preparation, such as non-parallel ends. This could result in uneven force application during the dynamic modulus test. The variation in deformation uniformity across different mixes and test conditions might also indicate inaccurate or inconsistent gauge point placement, particularly for AC10-Rh and AC10-RhRA50. Poor placement can cause significant measurement errors, contributing to higher deformation uniformity percentages. The AC10 and AC10-RhRA50 mixes tend to have higher deformation uniformity at higher frequencies (10 Hz), which could also be a sign of nonuniform air void distribution. Nonuniform voids can lead to localized deformation, contributing to higher overall deformation uniformity values. AC8-Rd shows relatively

lower deformation uniformity values compared to other mixes, indicating better performance. This suggests more uniform deformation, likely due to better sample preparation and consistent material properties. AC10-RdRA50 also shows lower deformation uniformity at various conditions, which might indicate better consistency in sample preparation and lower susceptibility to eccentric loading or poor gauge placement. AC10 and AC10-RhRA50 have higher deformation uniformity percentages, suggesting issues with eccentric loading, gauge placement, or non-parallel sample ends. AC8-Rd and AC10-RdRA50 perform relatively well, indicating better control over sample preparation and uniformity in deformation. The highest deformation uniformity values observed at 10 Hz and 4.0°C for several mixes may point toward test condition sensitivity, particularly to eccentric loading and nonuniform sample properties.

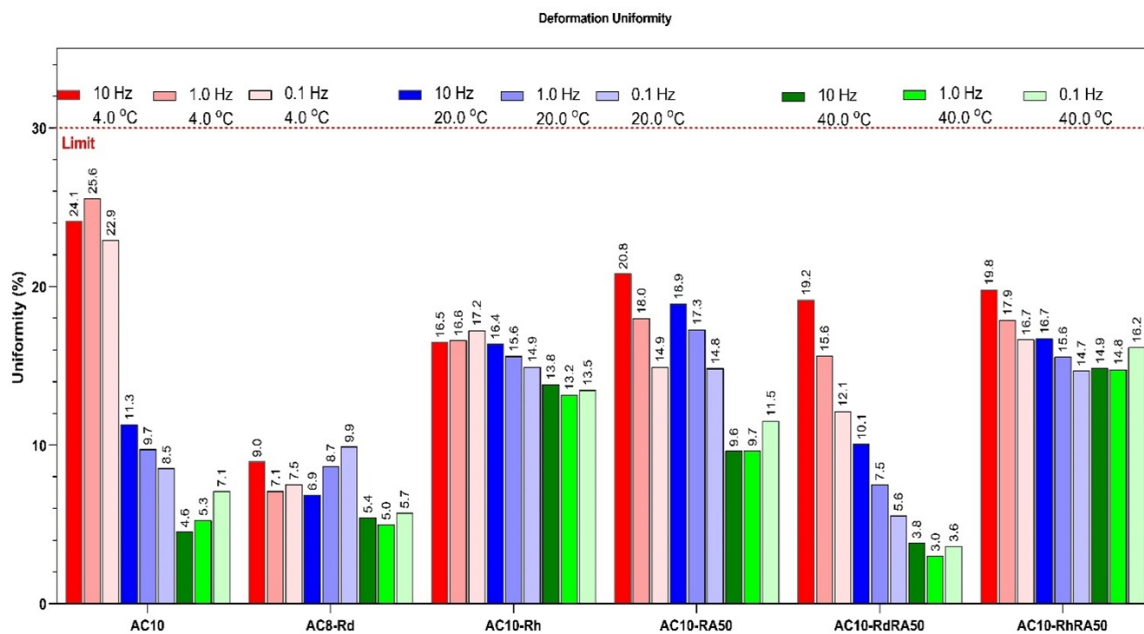


Figure 4.21: Deformation uniformity analysis of conventional and alternative paving materials

4.5.3.4 Phase uniformity analysis

Figure 4.22 represents phase uniformity (in degrees) for different asphalt concrete (AC) mixes under various frequencies (10 Hz, 1 Hz, 0.1 Hz) and temperatures (4°C, 20°C, 40°C) [10, 192]. The limit for acceptable phase uniformity appears to be 3 degrees, as indicated by the red dashed line. Most phase uniformity values across all mixes remain well below this limit, indicating generally acceptable performance. AC8-Rd and AC10-RA50 show slightly elevated phase uniformity at 40°C and 0.1 Hz (values of 1.2 and 1.3, respectively). This may indicate some influence of eccentric loading, particularly at higher temperatures and low frequencies, leading to non-uniform phase response. Eccentric loading can cause uneven stress, resulting in higher phase uniformity variation. AC8-Rd also shows higher phase uniformity values at 20°C and 0.1 Hz (value of 0.7), which suggests the possibility of loose gauge points affecting the measurement precision. Loose gauge points can result in inconsistent readings during testing, causing increased variation in phase uniformity. Variability in phase uniformity for AC10-RhRA50 at 40°C and 0.1 Hz (value of 0.9) suggests gauge point placement issues may be affecting this mix. Improper placement

of gauge points can lead to inaccuracies in strain measurement, causing higher phase differences between different parts of the sample [10, 192]. In some cases, such as AC10-RA50 and AC8-Rd, there are inconsistencies in phase uniformity across temperatures and frequencies (e.g., 1.3 degrees at 40°C and 0.1 Hz for AC10-RA50). This could indicate a damaged LVDT (Linear Variable Differential Transformer), which would contribute to inconsistent or inaccurate displacement measurements.

However, since most of the phase uniformity values are consistently low across the graph, it suggests that LVDT issues are not widespread but may be isolated to specific conditions or samples. AC8-Rd and AC10-RA50 show slightly elevated phase uniformity values, especially at higher temperatures and lower frequencies, suggesting potential issues with eccentric loading or damaged LVDTs. The variation across different frequencies and temperatures for AC10-RhRA50 and AC8-Rd may be due to poor gauge point placement or loose gauge points affecting measurement accuracy. Overall, the phase uniformity values are below the 3-degree limit, indicating acceptable uniformity for most test conditions.

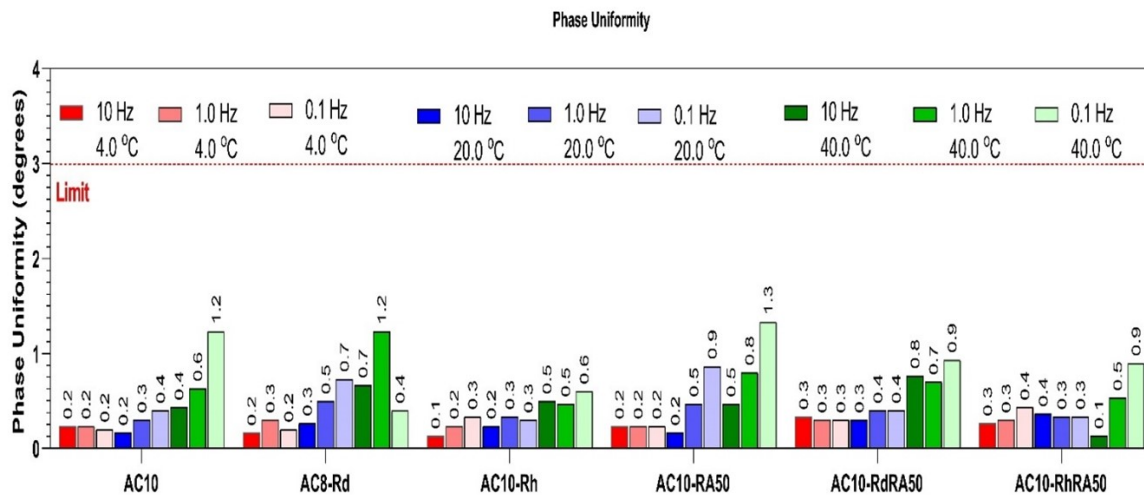


Figure 4.22: Phase uniformity analysis of conventional and alternative paving materials

4.5.4 Stiffness analysis using 2s2p1D model for conventional and rubberized asphalt mixes

The figure 4.23 illustrates the dynamic modulus master Curve for three types of asphalt mixes including conventional mix AC10 (control), rubberized hybrid asphalt (AC10-Rh), and rubberized with ECR (AC8-Rd). The dynamic modulus $|E^*|$ is plotted against reduced frequency, representing a combination of frequency and temperature to show stiffness behavior across different temperature and frequency conditions. The stiffness increases with frequency for all mixtures, reflecting the general behavior of asphalt where higher loading rates or lower temperatures lead to greater modulus [196]. At very low frequencies, AC8-Rd (red triangles) exhibits a lower modulus than the other two mixes, which implies it is softer at lower frequencies or higher temperatures, likely due to the inclusion of engineered crumb rubber, which can enhance flexibility. AC10 (control) mix falls between the

other two mixtures. It shows a consistent increase in modulus with frequency but does not reach the higher stiffness of AC10-Rh in the mid to high range. AC10-Rh a hybrid mix exhibits a higher modulus at low to mid frequencies compared to the conventional AC10 (control) and rubberized AC8-Rd. The polymer component enhances stiffness, especially at higher loading rates, while the ECR improves flexibility at lower frequencies (reflecting slower or higher temperature conditions) [197]. AC8-Rd being a mix with ECR alone, this mix has the lowest modulus at lower frequencies. It indicates more flexibility at slower loading rates, which is generally beneficial for fatigue resistance but may reduce rutting resistance under heavy, slow-moving loads. AC10-Rh, with its higher stiffness at mid to high frequencies, suggests an improved ability to resist rutting. The polymer enhances load-bearing capacity, making this mix ideal for high-traffic applications where resistance to deformation under repeated loading is essential [141]. At lower frequencies, AC8-Rd, which is softer and more flexible, may have a better capacity to absorb strains and avoid fatigue cracking. AC10-Rh, while stiffer, may still provide moderate fatigue resistance due to the elasticity added by the ECR, which helps dissipate energy without forming cracks as quickly as conventional mixtures might not perform well [198].

As a conclusion, the combination of polymer and ECR in AC10-Rh provides a strategic balance between stiffness and elasticity, optimizing it for high-traffic applications that demand both rutting and cracking resistance. This hybrid mix is likely to outperform both AC10 (control) and AC8-Rd in environments requiring durability against deformation and fatigue, making it a robust choice for mixed loading conditions.

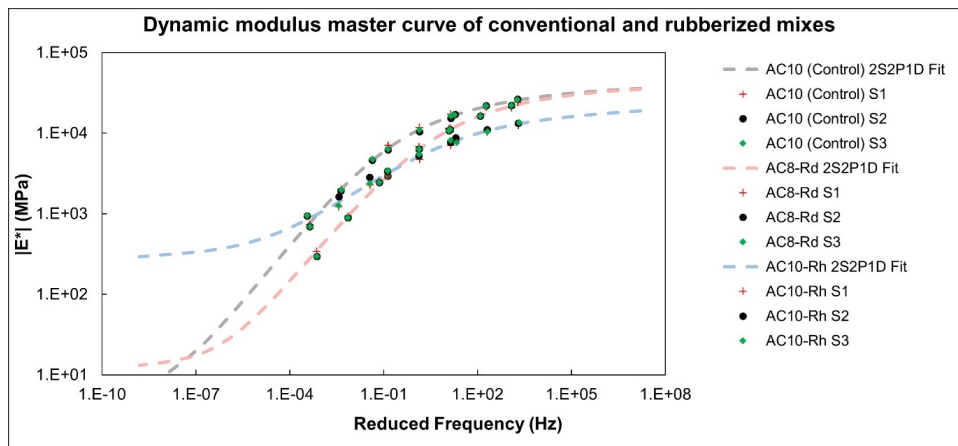


Figure 4.23: Stiffness analysis for conventional and rubberized asphalt mixtures

Figure 4.24 represents the dynamic modulus $|E^*|$ as a function of reduced frequency (Hz) for the conventional and rubberized asphalt mixtures: AC10 (control), AC8-Rd, and AC10-Rh. The AC10-Rh exhibits the highest dynamic modulus values across all frequencies. This indicates superior stiffness and ability to resist deformation under load compared to the other two mixtures [134]. The AC8-Rd shows intermediate modulus values, higher than AC10-Rh at moderate frequencies but lower than AC10 (control). Rubberized mixtures often exhibit flexibility while maintaining good resistance to deformation, as reflected here in figure 4.24.

The conventional AC10 (control) mixture shows the lowest dynamic modulus in most frequencies, indicating increased flexibility and potentially enhanced fatigue resistance [199].

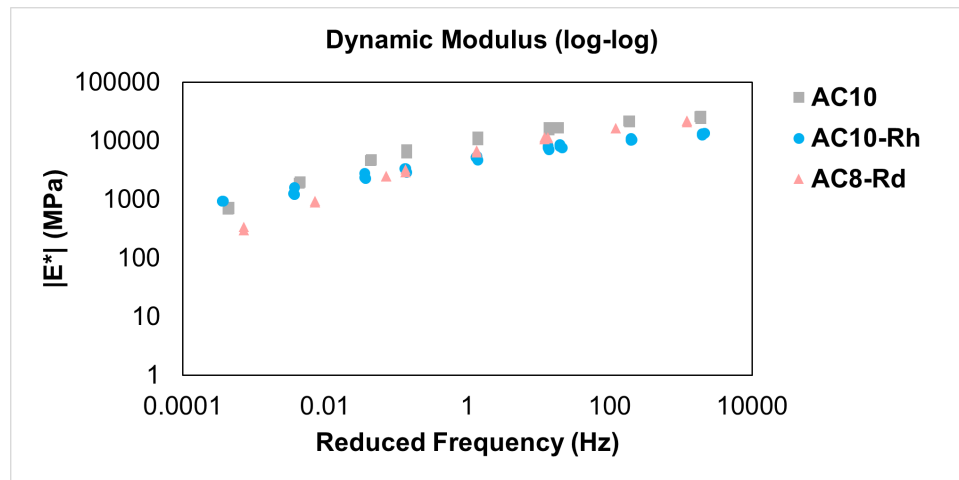


Figure 4.24: Dynamic modulus on log-log scale for conventional and rubberized asphalt mixtures

In summary, all mixtures show relatively low dynamic modulus values, reflecting viscoelastic relaxation behavior. The mixtures become softer under slow loading conditions. The stiffness of all mixtures increases significantly as asphalt becomes more elastic and less time-dependent under rapid loading. The AC10-Rh rubberized hybrid mixture maintains a higher modulus than AC8-Rd rubberized dry across a wide frequency range. This suggests that the addition of rubber with polymer has better stiffness properties while still incorporating rubber's flexibility benefits. The conventional asphalt mixture AC10 (control) is rarely suitable for applications requiring high stiffness, such as highways with heavy traffic loads, where rutting resistance is critical. While AC8-Rd mix is a balance between stiffness and flexibility, making it appropriate for urban roads or climates with temperature variations where moderate rutting and cracking resistance are required. While AC10-Rh, provides greater flexibility, making it ideal for low-volume roads or regions requiring high fatigue cracking resistance.

4.5.5 Phase angle analysis using 2s2p1D model for conventional and rubberized asphalt mixes

Figure 4.25 shows the phase angle (degrees) of three asphalt mixtures: AC10 (control), AC8-Rd, and AC10-Rh as a function of reduced frequency (Hz). Generally, the phase angle decreases as the reduced frequency increases [197]. This trend reflects the transition from viscoelastic behavior (dominated by delayed response) at low frequencies to elastic behavior (immediate response) at high frequencies. All mixtures exhibit this behavior, but their rates of decrease is different [200]. At low frequencies (0.0001–0.1 Hz), the phase angle is highest for AC8-Rd (rubberized dry mixture), followed by AC10 (control) and AC10-Rh. Higher phase angles indicate more energy dissipation, which reflects the rubber's viscoelastic contribution in AC8-Rd [133]. At high frequencies (≥ 100 Hz), the phase angle differences narrow among the mixtures, converging toward similar values (10° – 20°).

This behavior reflects the predominance of elastic behavior under rapid loading. AC10 (control) exhibits moderate phase angles across all frequencies, indicative of a balanced viscoelastic response. It shows slightly less damping compared to AC8-Rd but more than AC10-Rh at low frequencies. AC8-Rd maintains the highest

phase angle across most frequencies, indicating the strongest viscoelastic response and energy dissipation. This aligns with the rubber content, which enhances damping. AC10-Rh exhibits the lowest phase angle across most frequencies, suggesting enhanced elasticity and less energy loss [133]. The polymers likely reduce the material's damping and improve recovery. As a recommendation for higher flexibility AC8-Rd is preferred due to its higher phase angle, indicating better energy absorption and viscoelastic behavior. For the higher elasticity AC10-Rh is better suited for conditions requiring recovery after deformation, such as heavy traffic or repeated loading scenarios.

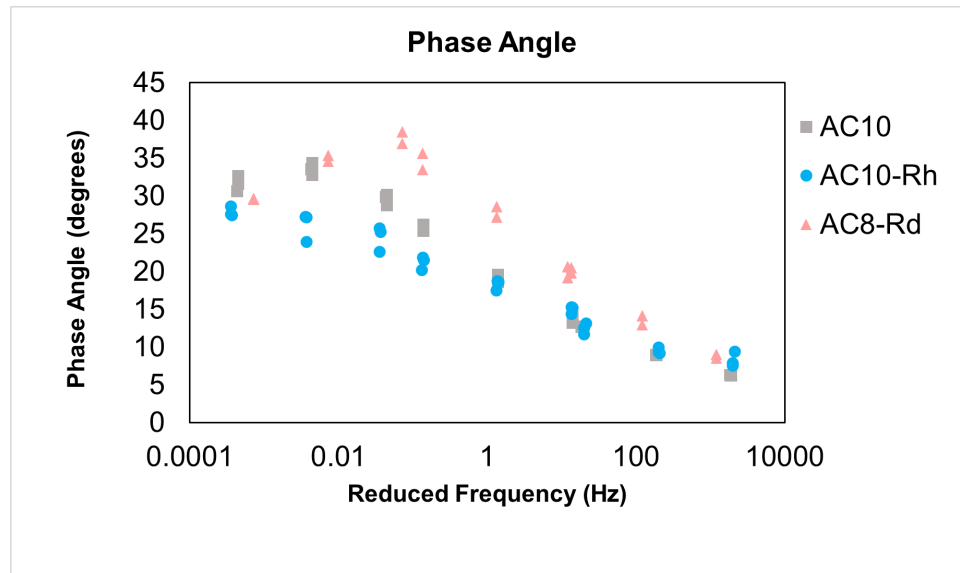


Figure 4.25: Phase angle analysis for conventional and rubberized asphalt mixtures

4.5.5.1 Shift factor analysis using 2s2p1D model for conventional and rubberized asphalt mixes

The shift factor is a measure of how material properties vary with temperature, particularly under loading conditions [201]. A more negative shift factor indicates a significant change in the material's response with increasing temperature [201]. As a general trend, all mixtures show increasingly negative shift factors as the temperature increases from 0°C, 20 °C and 40°C. This reflects reduced stiffness and greater viscoelasticity at higher temperatures, as expected for asphalt mixtures. The magnitude of the shift factor varies among the mixtures, indicating differences in their thermal sensitivity.

Figure 4.26 illustrates the shift factors as a function of temperature (°C) for the three mixtures. AC10(control) mixture behaves intermediate shift factor values at all temperatures. It shows a moderate sensitivity to temperature changes, maintaining a balance between stiffness and flexibility across the range [10]. While AC8-Rd shows the most negative shift factors, particularly at higher temperatures (40°C). This indicates that the rubberized dry mixture is more temperature-sensitive, becoming significantly more flexible at higher temperature [10]. This behavior is due to the ECR particles in the mix, which enhance flexibility and allow for stress relaxation under high thermal conditions [135]. The AC10-Rh displays the least negative shift factors, especially at higher temperatures. This mixture is the least sensitive to temperature increases, maintaining better stiffness due to the stabilizing effect

of polymers. The polymers likely improve high-temperature performance by reducing flow and deformation.

As a summary of the shift factors for the conventional and rubberized asphalt mixtures containing ECR and polymers, although the conventional mixtures are suitable for environments with moderate temperature fluctuations. However, the temperature response ensures that it performs moderately under standard conditions without extreme traffic loading. The AC8-Rd can be a good option for pavements in regions with high temperature variability or where stress relaxation is required (e.g., to minimize cracking under repeated thermal expansion and contraction). However, its high-temperature sensitivity may lead to rutting under heavy loads. AC10-Rh performs best in high-temperature environments or under heavy traffic loads, where stiffness is critical to resist permanent deformation (rutting) [202]. The polymers ensure minimal deformation while still providing some flexibility. Based on shift factors analysis as a final recommendation, the AC10-Rh can be recommended for heavy traffic and high-temperature roads like a Sicily region. While AC8-Rd containing ECR is considered for is considered for regions with high thermal stress variability due to its ability to relax stresses and resist thermal cracking.

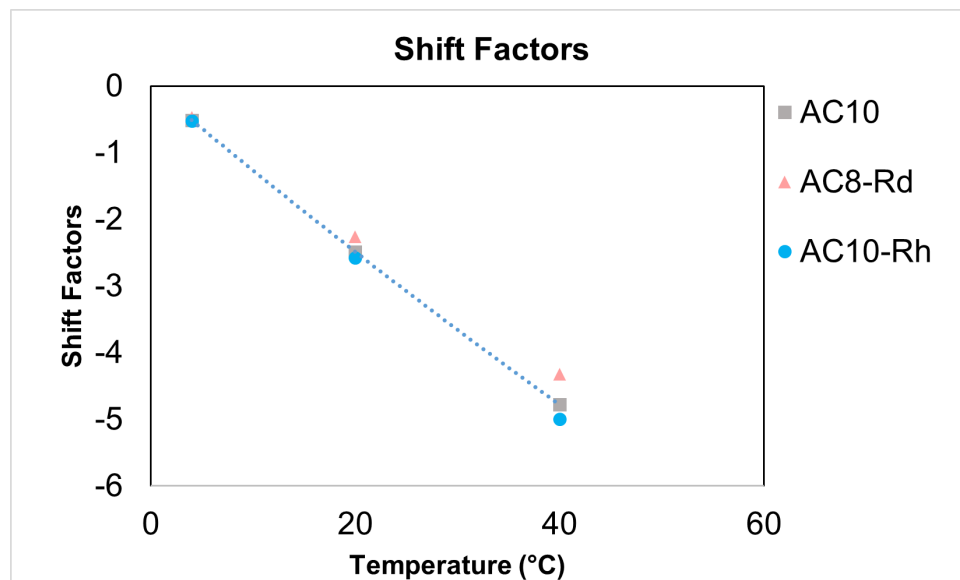


Figure 4.26: Shift factor analysis for conventional and rubberized asphalt mixtures

4.5.5.2 Fatigue cracking analysis using SVECD model for conventional and rubberized asphalt mixes

This section elaborates the understanding of the fatigue performance of conventional and rubberized asphalt mixtures. The pseudo stiffness (C) represents the material's structural integrity, while the damage parameter (S) quantifies the progression of fatigue damage under repeated loading [202]. All mixtures begin with a pseudo stiffness value close to 1.0, indicating similar initial structural integrity before any significant fatigue damage occurs [202]. The early performance of the mixtures aligns with the VEPCD report's expectation that stiffness differences emerge as damage accumulates. Figure 4.27 illustrates that AC10 (control) experiences the fastest stiffness degradation as damage accumulates. This indicates lower fatigue resistance compared to the modified mixtures, consistent with conventional mixtures' limited ability to resist microcrack growth. While AC8-Rd (rubberized dry

mixture) containing ECR declines in stiffness faster than AC10-Rh but more slowly than AC10 (control). Rubber's viscoelastic properties help to dissipate stress, delaying crack propagation and damage accumulation [203]. This suggests a balanced performance, offering good flexibility and fatigue resistance as compared to the conventional mixture. While AC10-Rh (hybrid mixture) exhibits the slowest decline in stiffness with increasing damage. This mixture maintains higher pseudo stiffness values (C) across a wide range of damage (S), indicating excellent fatigue resistance and delayed damage progression [204]. The polymer-rubber combination enhances durability, especially under cyclic loading.

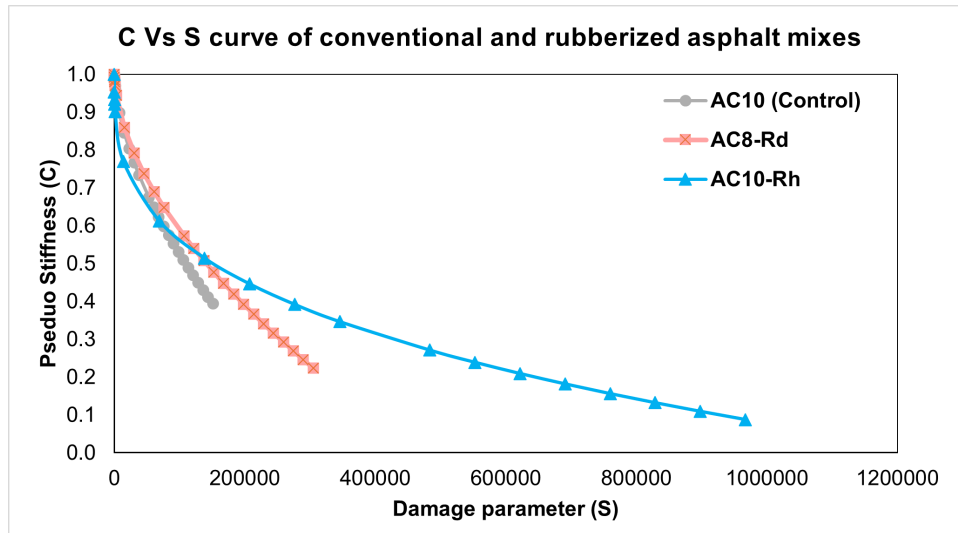


Figure 4.27: C VS S curve for conventional and rubberized asphalt mixtures

As compared to the conventional asphalt mixtures, the slower stiffness degradation in AC8-Rd and AC10-Rh reflects their ability to better resist fatigue cracking and delay macrocrack formation. The SVECD fatigue model described in the report predicts that mixtures with modifiers like rubber and polymers (AC8-Rd and AC10-Rh) demonstrate enhanced fatigue life due to their ability to delay damage accumulation. AC10-Rh clearly benefits from the polymers, as seen in its superior pseudo stiffness retention.

From the C Vs S curve analysis of conventional and rubberized asphalt mixtures, the AC10-Rh is the best option due to its excellent fatigue resistance and structural integrity over time. AC8-Rd provides a balanced solution, leveraging rubber for stress dissipation while maintaining moderate stiffness retention. AC10 (control) is suitable but may require thicker pavement layers to compensate for its faster stiffness loss.

4.5.5.3 Sapp for conventional and rubberized asphalt mixes

Figure 4.28 shows Sapp for conventional AC10 (control) and rubberized asphalt mixtures AC8-Rd, and AC10-Rh—along with corresponding traffic classifications obtained from SVECD report published in 2022 [91, 141]. The Sapp values provide insight into how well each mixture can handle surface-level damage under repeated loading, particularly in different traffic intensity scenarios. The AC10 (control) mixture has a Sapp value of 16, indicating moderate surface damage capacity. This value aligns with its suitability for standard (S) traffic conditions, defined as less than 10 million ESALs (equivalent single-axle loads) [141]. The AC10 (control) conventional composition

limits its ability to withstand heavy or slow-moving traffic, making it ideal for low to moderate traffic environments. The AC8-Rd mixture shows an improved **Sapp** value of 19.5, reflecting a 22 % improvement over the AC10 Control. This enhanced performance can be attributed to the addition of rubber in the dry process, which increases flexibility and resistance to cracking [141, 205].

This mixture is well-suited for heavy (H) traffic conditions, with load capacities between 10 and 30 million ESALs. The rubberized mixture balances durability and flexibility, making it a strong candidate for urban roads or areas with moderate traffic stress. The AC10-Rh mixture demonstrates the highest **Sapp** value of 25, a 56 % improvement over the AC10 (control). This significant enhancement stems from the combined effects of polymers and rubber, which strengthen the mixture's resistance to surface damage while maintaining excellent durability. This mixture is classified for very heavy (V) traffic, making it suitable for conditions exceeding 30 million ESALs [205]. Its superior performance under high loads and slow-moving traffic also qualifies it for extremely heavy (E) traffic conditions, making it ideal for highways, industrial zones, or areas with prolonged vehicle loading.

This analysis highlights the increasing effectiveness of adding rubber and polymer modifiers. The AC8-Rd mixture offers a practical solution for roads requiring moderate damage resistance, while the AC10-Rh stands out as a high-performance option for demanding traffic conditions. The AC10 (control) remains a cost-effective choice for standard applications, but its limitations in high-load scenarios are evident when compared to the modified mixtures.

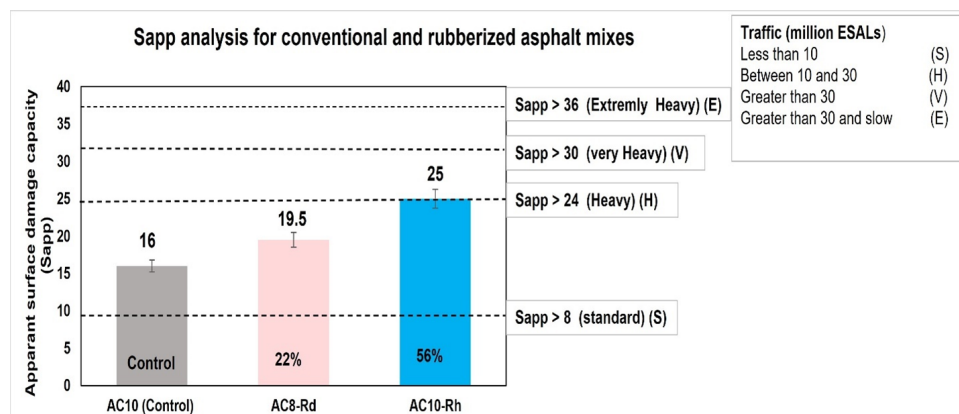


Figure 4.28: Sapp analysis for conventional and rubberized asphalt mixtures

The VEPCD-based **SVECD** model highlight that the incorporation of polymers and rubber significantly enhances fatigue and surface damage resistance, aligning with the observed improvements in **Sapp** for AC8-Rd and AC10-Rh. The AC10-Rh mixture, with its superior **Sapp**, benefits from the healing effects of polymers and enhanced elastic recovery, making it the most durable under repeated loading and high traffic.

4.5.5.4 D^R Failure criterion of conventional and rubberized asphalt mixtures

Figure 4.29 shows a linear relationship between Sum (1-C) and Nf (fatigue life) for the AC10 (control) mixtures [49, 206]. This indicates that the damage accumulation process follows a predictable trend as the number of cycles to failure increases. Sum (1-C) quantifies the cumulative damage in the material over its fatigue life. The higher the value, the more damage has been accumulated, reflecting the progression toward failure [207]. As Nf increases,

the cumulative damage also increases, which is expected as the mixture undergoes repeated cyclic loading. The fatigue life (Nf) represents the number of cycles the material can endure before failure. In this case, the AC10 (control) mixture shows Nf values between 43,000 and 45,000 cycles under the specific testing conditions. The data points align closely with the fitted trend line, suggesting a reliable relationship between cumulative damage and fatigue life for the AC10 (control) mixture. This validates the predictive models used in the analysis. The fatigue life of 43,000–45,000 cycles indicate the AC10 (control) mixture's limitations in withstanding repeated loading compared to modified mixtures (e.g., rubberized or polymer-modified).

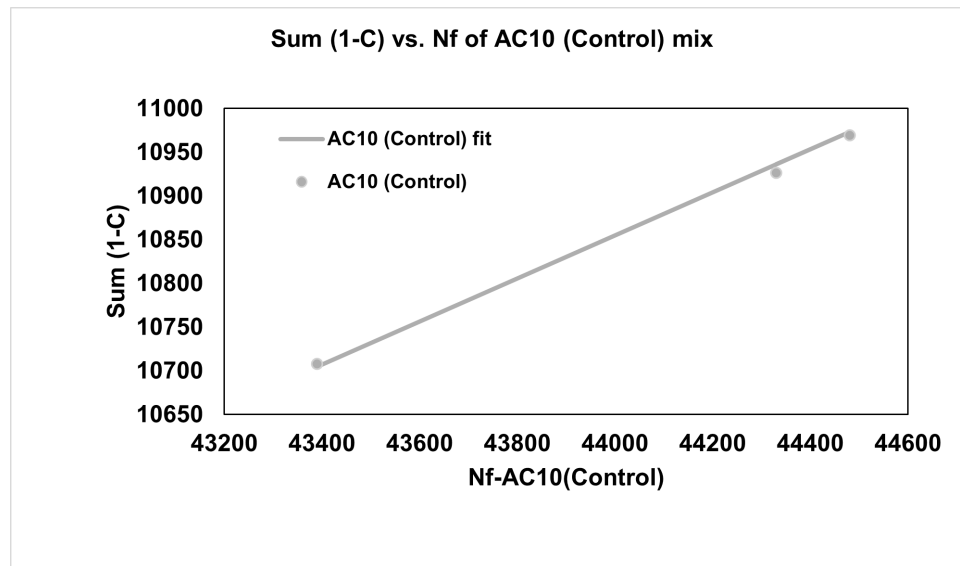


Figure 4.29: Failure analysis for AC10 control asphalt mixtures

The fatigue life (Nf) of the AC8-Rd mixture ranges between 46,000 and 51,000 cycles, shown in figure 4.30, representing an improvement over the AC10 (control) mixture (43,000–45,000 cycles). This demonstrates the enhanced durability of the rubberized mixture due to the stress dissipation capabilities of rubber [178]. The Sum (1-C) values for AC8-Rd range from 20,000 to 25,000, which are significantly higher than those observed for AC10 (control) (10,650–11,000). This suggests that the AC8-Rd mixture can endure more damage before failure, reflecting better fatigue performance. Like AC10 (control), the damage accumulation follows a linear relationship with fatigue life, indicating predictable and steady degradation under cyclic loading. The trend line closely fits the data points, validating the reliability of the [SVECD](#) fatigue model for this mixture. The extended fatigue life (Nf) of AC8-Rd (up to 51,000 cycles) highlights its superior ability to resist fatigue cracking compared to AC10 (control). The inclusion of rubber improves the mixture's flexibility, allowing it to absorb stresses and delay crack initiation and propagation. The higher Sum (1-C) values indicate that AC8-Rd can sustain more cumulative damage before reaching failure, showcasing its enhanced durability over the conventional AC10 (control) mixture.

Rubberized mixtures containing [ECR](#) like AC8-Rd are known for their viscoelastic behavior, which helps dissipate stress and reduces the concentration of microcracks under repeated loading. This property is reflected in the higher fatigue life and cumulative damage values [208]. The enhanced fatigue life of AC8-Rd supports its suitability for roads subjected to moderate traffic loads or thermal stresses. Figure 4.31 shows the relationship between cumulative damage parameter (Sum (1-C)) and fatigue life (Nf) for the hybrid rubberized mixture (AC10-Rh), which combines polymers and rubber. The data points and the trend line reveal the performance characteristics

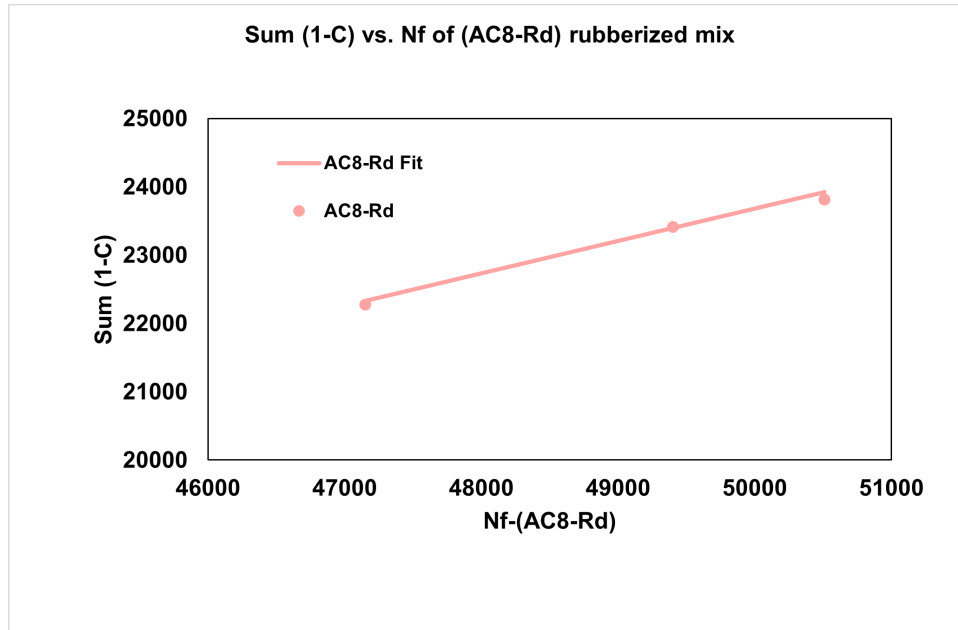


Figure 4.30: Failure analysis for AC8-Rd asphalt mixtures

of this mixture under cyclic loading. The fatigue life (Nf) of the AC10-Rh mixture ranges from 58,000 to 63,000 cycles, significantly exceeding both AC10 (control) (43,000–45,000 cycles) and AC8 Rd (46,000–51,000 cycles). This indicates its superior fatigue resistance, as it endures more load cycles before failure [208]. The Sum (1-C) for AC10-Rh (40,000–50,000) is nearly double that of AC8-Rd (20,000–25,000) and significantly higher than AC10 Control (10,650–11,000). This indicates that AC10-Rh can sustain much greater damage while maintaining its structural integrity [209]. The combination of polymers and rubber in AC10-Rh enhances both flexibility and stiffness, improving its ability to withstand repeated loading and delay damage progression. This hybrid composition clearly outperforms the other mixtures.

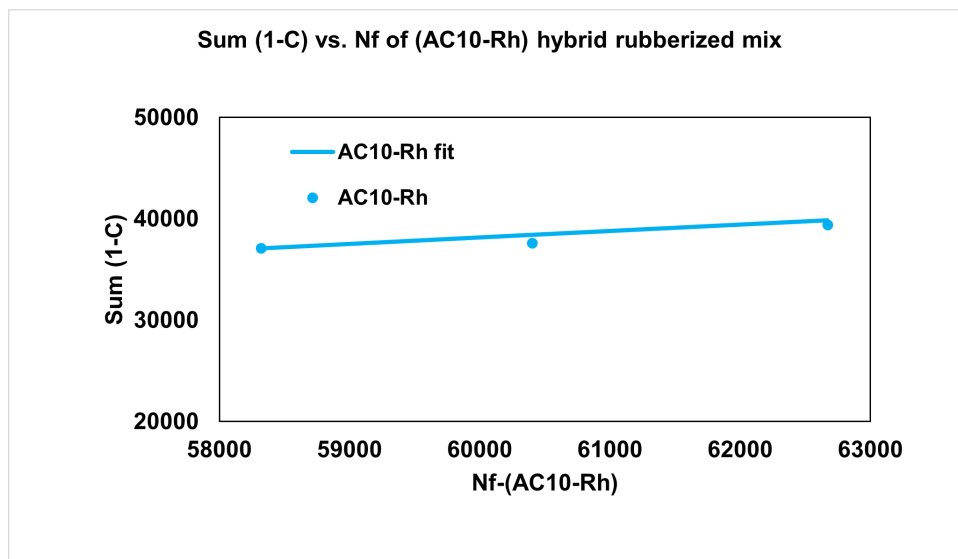


Figure 4.31: Failure analysis for AC10-Rh asphalt mixtures

As a summary of failure analysis from figures 4.29, 4.30, and 4.31, for high-traffic roads AC10-Rh is the ideal choice for highways and roads with heavy or high-speed traffic, as its superior fatigue life ensures long-term

durability. For moderate traffic conditions AC8-Rd is suitable for urban roads or secondary highways where moderate fatigue resistance is sufficient, offering a cost-effective alternative. While for low-traffic applications, AC10 (control) is acceptable for low-traffic or less demanding conditions, but its limited fatigue performance makes it less desirable for high-stress environments. The SVECD model in the report emphasize the role of material modifiers in improving fatigue performance. The exceptional performance of AC10-Rh aligns with these predictions, highlighting the combined benefits of rubber and polymers in enhancing both stiffness and flexibility.

4.5.6 CTC analysis of conventional and rubberized asphalt mixes

Figure 4.32 illustrates the CTC as a function of temperature for conventional AC10 (control), and rubberized AC8-Rd (rubberized dry mixture) containing ECR, and AC10-Rh (hybrid rubberized with polymer mixture). The graph includes both fitted and predicted values for each mixture, providing insights into their thermal behavior across a wide temperature range. AC10 (control) exhibits the lowest CTC values across the temperature range as shown in figure 4.32. This is expected as conventional mixtures lack the thermal enhancement provided by rubber or polymer modifiers [210]. The relatively flat curve at lower temperatures indicates minimal thermal conductivity changes under cold conditions. AC10 Rh's superior CTC makes it ideal for high-temperature regions or heavy-traffic roads, where efficient heat dissipation can reduce the risk of thermal cracking or deformation. AC8-Rd provides moderate performance, offering better heat management than AC10 (control) but falling short of AC10-Rh.

All mixtures show reduced CTC values as the temperature drops. However, AC10-Rh maintains relatively higher conductivity, potentially aiding in temperature distribution and reducing the risk of low-temperature cracking. AC10 (control), with its lowest CTC, might be more prone to localized thermal stresses in cold environments. The steeper slopes for AC8-Rd and AC10-Rh suggest that modified mixtures are more sensitive to temperature changes, which could impact their performance in extreme environments. AC10 (control) mixture flatter curve reflects its thermal stability but at the cost of lower heat conduction [211]. AC10-Rh demonstrates the highest thermal conductivity, making it ideal for high-temperature applications. AC8-Rd shows moderate performance, providing better heat transfer than AC10 (control). AC10 (control) lags, suitable for standard conditions but less effective in extreme thermal environments

4.5.7 Rutting analysis of conventional and rubberized asphalt mixtures

The accumulated permanent deformation (μ_{evp}) as a function of the number of cycles for conventional and rubberized asphalt mixtures at high temperature are shown in figure 4.33. The test was conducted at high temperature of 55.0°C to evaluate rutting resistance under high-temperature conditions. All the mixtures show an increasing trend of permanent deformation with an increasing number of cycles [15]. This is expected as repeated loading induces cumulative damage, particularly at high temperatures [12]. The rate of deformation slows down for all mixtures after the initial cycles, but AC10 (control) continues to accumulate more deformation than the

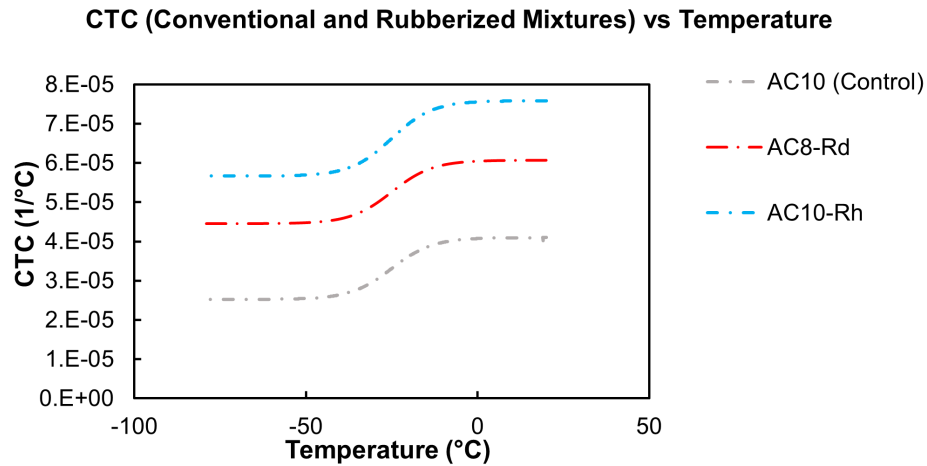


Figure 4.32: CTC analysis for conventional and rubberized asphalt mixtures modified mixtures.

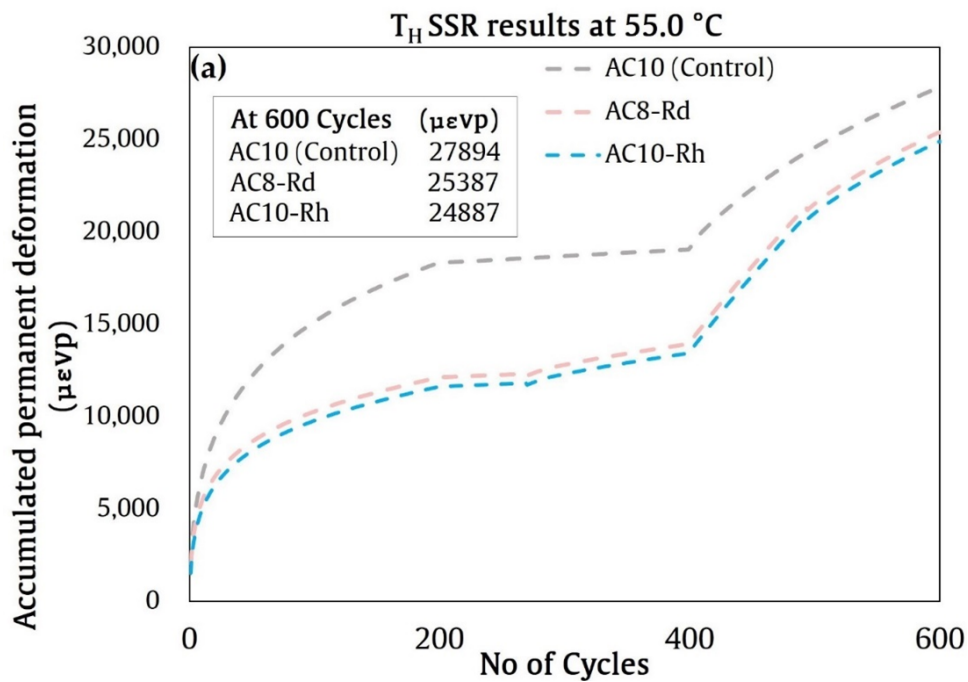


Figure 4.33: Permanent strain analysis at high temperature for conventional and rubberized asphalt mixtures

Around 600 cycles, AC10 (control) has highest permanent deformation at 27,894 ($\mu\epsilon_{vp}$). AC8-Rd has moderate permanent deformation at 25,387 ($\mu\epsilon_{vp}$), reflecting better rutting resistance compared to AC10 Control while AC10 Rh has Lowest permanent deformation at 24,887($\mu\epsilon_{vp}$) indicating the best rutting resistance. AC10-Rh (polymer and rubber) shows the least deformation, highlighting the combined benefits of polymers and rubber in enhancing resistance to permanent deformation under high-temperature and repeated loading conditions [25]. AC8-Rd (rubberized dry) also shows improved performance over AC10 (control), but its rutting resistance is slightly less effective than AC10-Rh due to the absence of polymers.

In summary, AC10-Rh outperforms other mixtures, with the lowest accumulated permanent deformation,

making it ideal for high-temperature and high-load scenarios. AC8-Rd offers improved performance compared to AC10 (control), which exhibits the highest deformation and is less suitable for challenging conditions.

The accumulated permanent deformation ($\mu_{\epsilon vp}$) as a function of the number of cycles for the conventional and rubberized asphalt mixtures at low temperature are shown in figure 4.34. The testing was conducted at a moderate temperature of 20.0°C to evaluate deformation resistance under standard conditions matching the climatic region of Sicily Italy [8]. All mixtures demonstrate a steady increase in permanent deformation as the number of cycles increases, reflecting the progression of the plastic strain under cyclic loading. AC10 (control) accumulates more deformation compared to the rubberized mixtures, indicating a lower resistance to permanent deformation under these conditions [8]. AC10 Control has the highest permanent deformation at 3946.5 ($\mu_{\epsilon vp}$). AC8-Rd shows reduced deformation at 2992.5 ($\mu_{\epsilon vp}$), reflecting improved rutting resistance due to rubber modification. AC10-Rh has the least deformation at 2842 ($\mu_{\epsilon vp}$), showcasing the best performance among the mixtures.

In summary, both at low and high temperature AC10-Rh outperforms other mixtures due to the combined effects of polymers and rubber, which improve both stiffness and flexibility, reducing deformation under repeated loading [212]. AC8-Rd exhibits moderate improvement over AC10 Control, benefiting from the stress-dissipating properties of rubber in the dry mixture. AC10-Rh is the most suitable option for roads with moderate traffic and standard temperature conditions due to its superior deformation resistance. AC8-Rd offers an alternative with moderate improvement over conventional mixtures. While AC10 (control) may be suitable for low-traffic roads but is less reliable under repeated loading.

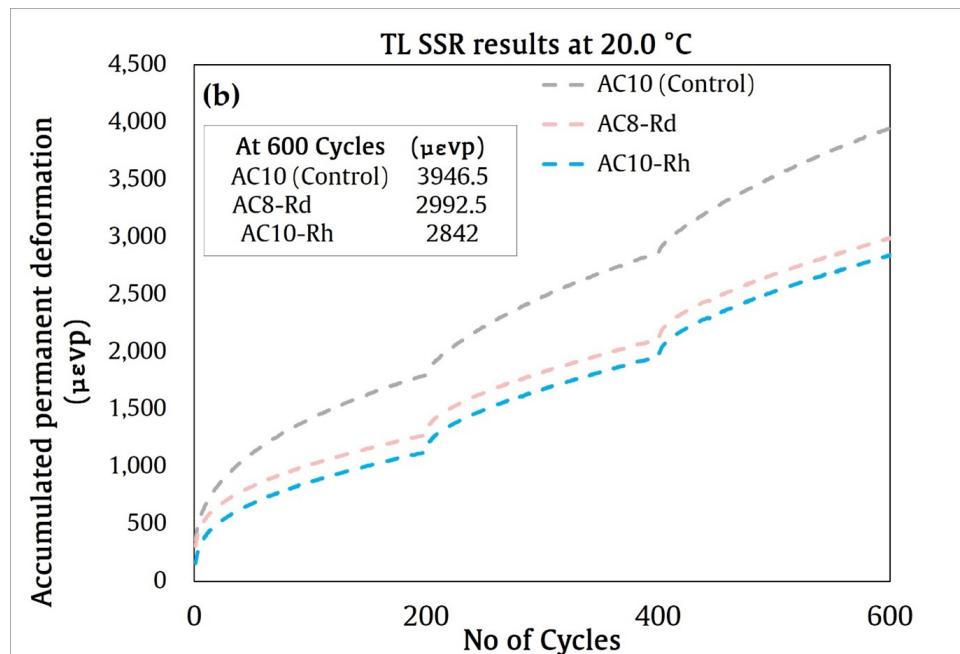


Figure 4.34: Permanent strain analysis at low temperature for conventional and rubberized asphalt mixtures

4.5.7.1 Rutting strain index (RSI) analysis of conventional and rubberized asphalt mixtures

The **RSI** is a metric used to evaluate the rutting performance of asphalt mixtures. It represents the average permanent strain as a percentage, calculated by dividing the permanent deformation of an asphalt layer by its thickness [212]. **RSI** is typically assessed over a 20-year pavement service life, assuming 30 million 18-kip standard axle load repetitions for a standard pavement structure [212]. Figure 4.35 presents the **RSI** values for conventional AC10 (control), and rubberized AC8-Rd (rubberized dry mixture) containing **ECR**, AC10-Rh (hybrid rubberized mixture)—and classifies them according to traffic load categories (Standard, Heavy, Very Heavy, Extremely Heavy). AC10 (control) has the highest **RSI** at 6.53 %, indicating the poorest resistance to rutting among the three mixtures. AC8-Rd shows improved performance with an **RSI** of 3.99 %, reflecting a 40 % reduction in rutting strain compared to AC10 (control). AC10 Rh exhibits the best performance with an **RSI** of 3.54 %, achieving a 45 % reduction in rutting strain compared to AC10 (control). AC10-Rh demonstrates the highest rutting resistance, making it the most reliable option for roads exposed to heavy or high-speed traffic [213]. AC8-Rd also shows considerable improvement over AC10 (control), making it suitable for regions with moderate traffic loads. While AC10 (control) has the least resistance to rutting, making it less suitable for demanding applications.

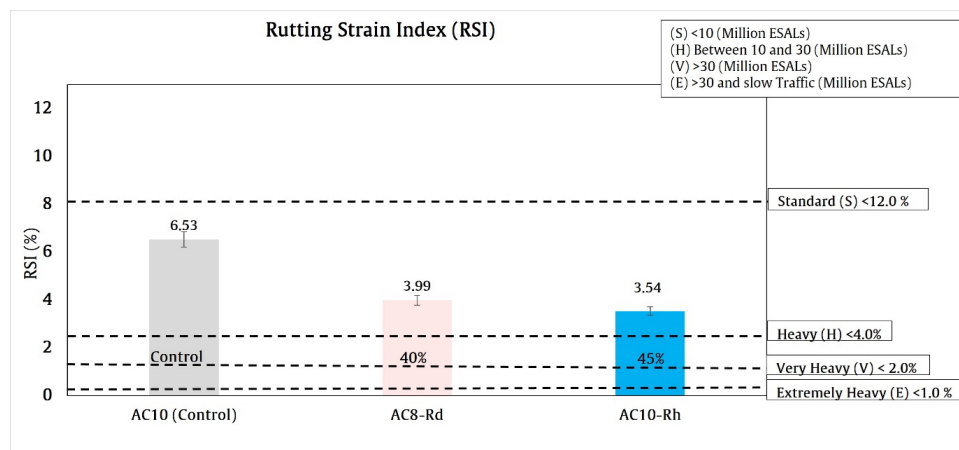


Figure 4.35: Rutting strain index (RSI) analysis of conventional and rubberized asphalt mixtures

4.5.8 Pavement performance analysis of conventional and rubberized asphalt mixes

For predicting pavement performance using a Visco Elastoplastic Continuum Damage (**VEPCD**) framework. Key aspects of the analysis include the material properties of asphalt mixtures, traffic loading, environmental conditions, and their combined effects on pavement durability [211]. Previous sections of this thesis described a **FlexMATTM** cracking **FlexMATTM** rutting tool at the material level that models the behavior of asphalt under cyclic load. It evaluates performance metrics like stiffness, fatigue resistance, and permanent deformation at the material scale. The output of these **FlexMATs** will be further utilized by **FlexPAVETM**, a pavement-level tool that integrates material properties into a larger framework, analyzing multilayered pavement systems under varying traffic and environmental conditions. The following key performance factors will be further analyzed for the performance of conventional and rubberized asphalt mixtures in a pavement structure. Please note that the detailed

methodology for the pavement performance analysis using [FlexPAVETM](#) has been provided in chapter 3 of this research work.

4.5.9 Damage analysis and fatigue cracking

Damage contours and fatigue cracking results from repeated traffic loads, causing microcracks that grow into macro-scale damage of the conventional and rubberized asphalt mixtures [12]. The [SVECD](#) model is employed to predict the initiation and propagation of these cracks [12]. The model quantifies cumulative damage and stiffness reduction, linking them to the material's fatigue resistance.

4.5.10 Rut depth and permanent Deformation

Rutting is modeled using the [shift](#) model, which evaluates the accumulation of permanent deformation under high temperatures and traffic stresses for conventional and rubberized asphalt mixtures. This includes factors like loading time, temperature, and viscoelastic properties of asphalt. Modified mixtures (e.g., those with polymers and rubber) exhibit better resistance to rutting due to their enhanced viscoelastic and viscoplastic behaviors [12].

4.5.11 Healing and Recovery

Healing, a unique property of asphalt mixtures, refers to the material's ability to recover stiffness during rest periods. Please note the version of [FlexPAVETM](#) v 1.1 does not support the healing model. Therefore, this research work do not includes the healing model [12].

4.5.12 Damage analysis of Conventional and rubberized asphalt mixes

The damage factor patterns in Figure 4.36 for AC10 (control) are linked to the [SVECD](#) model outlined in the FlexPAVE system [214, 12, 215]. Consider the definition of damage in the [SVECD](#), that is, $damage = 1 - C$. Where is "C" is the pseudostiffness of the material [12]. The damage factor is the damage accumulated to the D^R failure criterion multiplied by the number of cycles for corresponding mixture considered in cyclic fatigue testing [12]. This figure 4.36 illustrates the damage contours that correspond to the width and depth of the pavement. Red zones indicate areas of significant accumulation of damage (high cracking potential). Blue zones indicate minimal or no damage. The ability of the AC10 (control) mixture to resist fatigue cracking under cyclic traffic load and environmental stresses can be visualized in Figure 4.36. The red zone is the most extensive, both in depth and width, indicating a severe accumulation of damage in the center of the loading zone and extending outward. The damage reaches deeper into the pavement layers, reflecting poor resistance to cracking. Conventional mixtures such as AC10 (control) lack modifiers, leading to lower fatigue resistance. This makes the mixture prone to rapid crack initiation and propagation under cyclic loading.

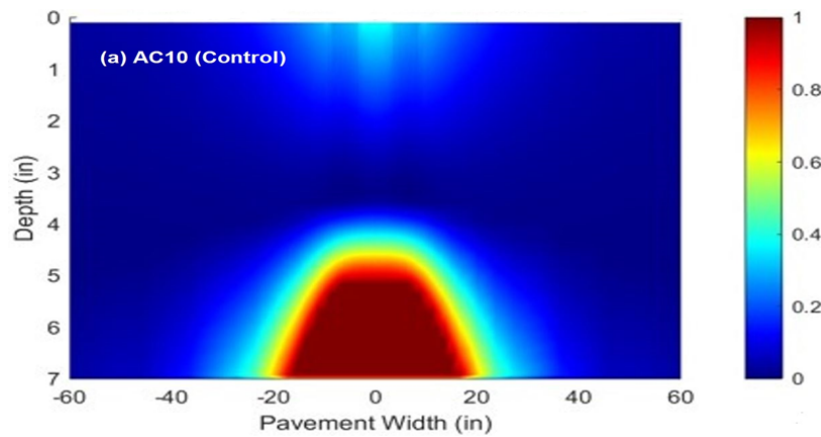


Figure 4.36: Damage analysis of AC10 (Control) mixture

The damage contours of AC8-Rd can be seen in Figure 4.37. The red zone is significantly reduced compared to AC10 (control), with damage concentrated closer to the surface and covering a smaller area. The transition from high to low damage is more gradual, reflecting better stress dissipation. Rubber modification enhances the viscoelastic properties of the mixture, reducing crack growth by dissipating energy and delaying crack propagation [25]. This makes AC8-Rd more durable under repeated loading.

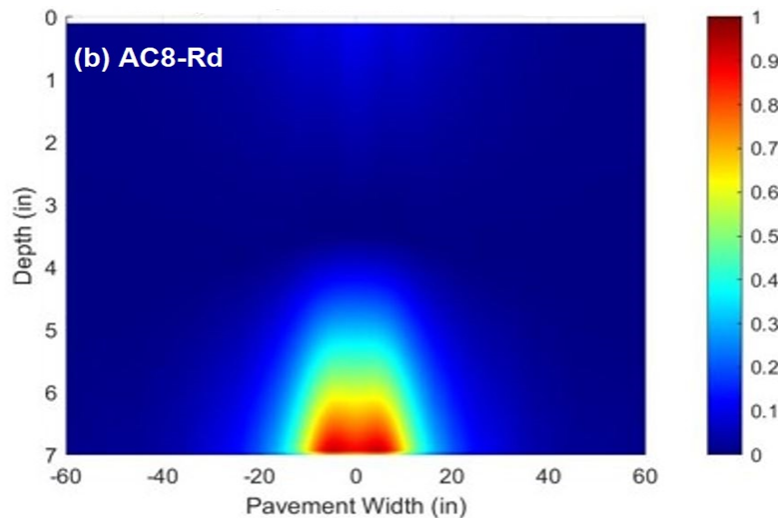


Figure 4.37: Damage analysis of AC8-Rd mixture

The smallest red zone, with damage highly localized and minimal spread, indicates superior resistance to fatigue cracking for AC10-Rh hybrid mixtures shown in figure 4.38. The blue zone dominates, showing minimal damage penetration into the pavement layers. The hybrid modification with both polymers and rubber provides the highest resistance to cracking. Polymers improve stiffness and load recovery, while rubber enhances energy dissipation, making AC10-Rh ideal for high-traffic and high-stress conditions [25].

As a comparative analysis AC10 (control) exhibits the highest damage accumulation, followed by AC8-Rd, with

AC10-Rh showing the least. This trend confirms the superior fatigue performance of modified mixtures compared to conventional ones. Damage extends deeper into the layers for AC10 (control), increasing the likelihood of structural failure. AC8-Rd and AC10-Rh effectively limit damage depth, preserving the integrity of the underlying layers. Rubber modification in AC8-Rd enhances flexibility, reducing crack severity but not as effectively as the hybrid modification in AC10-Rh. AC10-Rh demonstrates the best performance due to the combined benefits of rubber and polymers.

As a summary AC10-Rh is recommended choice for heavy traffic areas due to its minimal damage accumulation, ensuring long-term performance and resistance to fatigue cracking [12, 25]. AC8-Rd provides an option with improved cracking resistance compared to AC10 (control). AC10 (control) may be sufficient, but its susceptibility to cracking necessitates thicker pavement layers or periodic maintenance. The SVECD model quantifies cumulative damage and stiffness reduction, directly correlating to the observations in these heat maps. The model's ability to predict crack initiation and propagation under cyclic loading is validated by the improved performance of rubberized and hybrid mixtures.

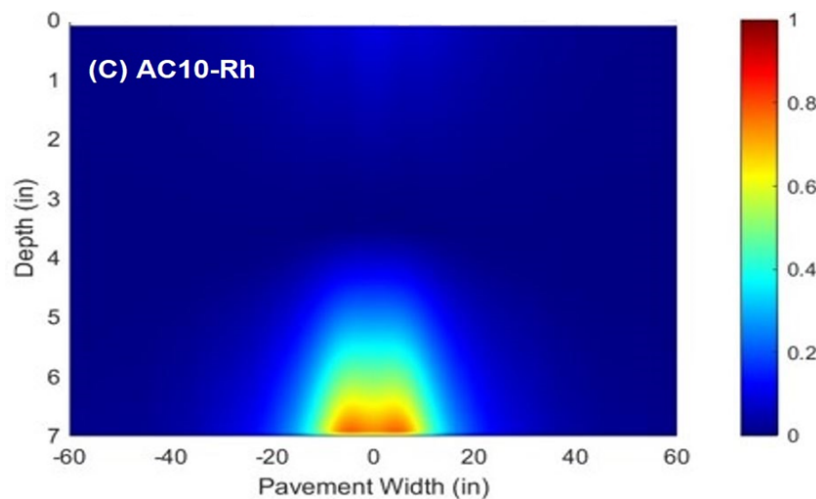


Figure 4.38: Damage analysis of AC10-Rh mixture

4.5.13 Cracking analysis of conventional and rubberized asphalt mixtures

Figure 4.39 represents the cumulative damage percentage over a 10-year pavement life for three asphalt mixtures i-e, AC10 (control), AC8-Rd, and AC10-Rh. The analysis leverages SVECD incorporated in FlexPAVETM v 1.1 simulations to predict crack initiation and propagation under realistic loading and environmental conditions. AC10 (control) exhibits the steepest increase in cumulative damage, reaching approximately 70 % by year 10, indicating its poor resistance to fatigue cracking. AC8-Rd shows improved performance, with a slower rate of damage accumulation, reaching about 60 % damage by year 10. AC10-Rh demonstrates the best cracking resistance, with damage limited to 50 % by year 10, reflecting its superior durability.

During the first 2 years, all three mixtures exhibit relatively low damage (<10 %), indicating that crack initiation

takes time. By year 4, differences in cracking resistance become apparent, with AC10-Rh exhibiting the least damage accumulation, followed by AC8-Rd and then AC10 (control) [67, 12, 129].

By year 6, the gap between the mixtures widens, highlighting the impact of material modifications. AC10 (control) mixtures damage accelerates rapidly, indicating a high susceptibility to fatigue and crack propagation [67, 12, 129].

AC8-Rd maintains a moderate damage accumulation, benefitting from the stress-relief properties of rubber. AC10-Rh continues to outperform, showing the slowest damage accumulation, owing to its hybrid modification with polymers and rubber.

As a summary of the conventional and rubberized asphalt mixtures AC10 (control) experiences the most significant fatigue cracking due to its lack of modifiers. It may be Unsuitable for high-traffic or heavy-load conditions without thicker pavement designs or frequent maintenance. The addition of rubber enhances flexibility, reducing crack initiation and propagation. AC8-Rd performs better than AC10 (control) but accumulates more damage over time compared to AC10-Rh. It can be recommended for moderate traffic. AC10-Rh Combines the stiffness-enhancing properties of polymers with the stress-dissipating effects of rubber, providing the best resistance to cracking. It can be ideal for high-traffic roads, industrial zones, and regions with extreme environmental stresses. AC10-Rh demonstrates the best cracking resistance over a 10-year service life, making it ideal for high-demand applications. AC8-Rd provides moderate resistance, balancing cost and performance. AC10 (control) shows the highest cumulative damage, limiting its suitability to low-stress environment.

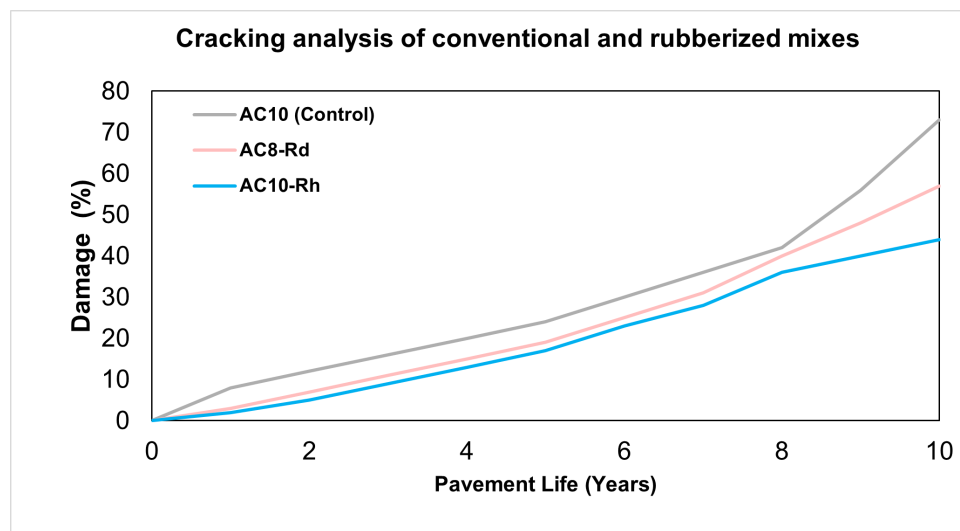


Figure 4.39: Cracking analysis of conventional and rubberized asphalt mixtures

4.5.14 Rut-depth analysis of conventional and rubberized asphalt mixtures

The outputs of RSI and shift model were utilized to perform the simulations in FlexPAVETM v 1.1 for the prediction of rut depth. Figure 4.40 compares the predicted rut depths for conventional and rubberized asphalt mixtures highlighting their performance in resisting permanent deformation under traffic loading. The rut depth is measured in millimeters (mm), with the percentages indicating improvement over the conventional AC10 (control) mixture.

AC10 (control) shown in figure 4.40 has the highest rut depth, reflecting its poor resistance to permanent deformation. The absence of modifiers in AC10 (control) makes it susceptible to rutting under high temperatures and repeated traffic loading [15]. For AC8-Rd, rubber modification improves the viscoelastic properties of the mixture, dissipating stresses more effectively and reducing rut depth [8]. AC8-Rd is suitable for moderate traffic conditions with improved durability over conventional mixtures. The AC10-Rh the combination of polymers and rubber provides the best resistance to rutting, making AC10-Rh the most durable among the three mixtures. The hybrid composition enhances stiffness and reduces deformation, even under heavy traffic or high-temperature conditions. AC10-Rh demonstrates the highest resistance to rutting, with the lowest rut depth among the three mixtures. AC8-Rd also performs significantly better than AC10 (control), though its resistance is slightly lower than AC10-Rh. The inclusion of rubber in AC8-Rd improves stress dissipation and flexibility, leading to reduced rutting. The hybrid modification in AC10-Rh further enhances resistance due to the synergistic effects of polymers and rubber, which provide both flexibility and stiffness [213].

As a summary, AC10-Rh is the most suitable option for highways, industrial zones, and areas experiencing heavy or high-speed traffic due to its superior rutting resistance. AC8-Rd offers a cost-effective alternative for urban streets or secondary highways, providing better performance than conventional mixtures. AC10 (control) can be used for roads with low traffic volumes, but its higher rut depth indicates a need for additional maintenance or pavement thickness to improve durability.

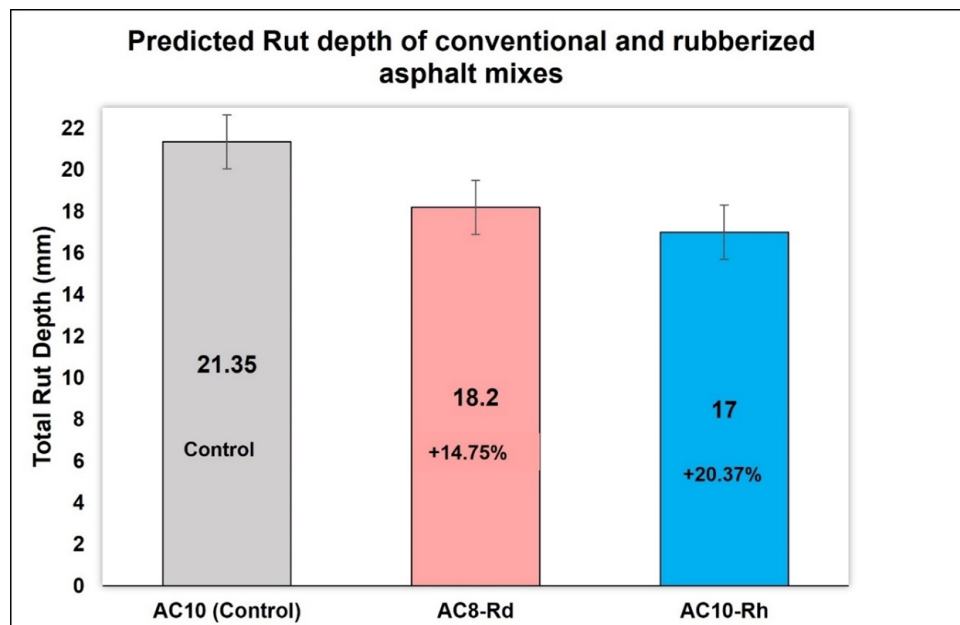


Figure 4.40: Rut-depth analysis of conventional and rubberized asphalt mixtures

4.5.15 Conclusions and recommendations of section IV

As a response to RO-III in section 1.2 "To investigate whether a "advanced-level" SVECD damage-related characterization of paving materials can discriminate between performances conventional and APMs

- Stiffness using AMPT for dynamic modulus testing and 2s2p1D model.

- Rutting resistance using [AMPT](#) for [SSR](#) testing and [shift](#) model.
- Cracking resistance using [AMPT](#) for cyclic fatigue test and [SVECD](#) model.
- Thermal analysis using [CTC](#) model

This section provides a detailed analysis using advanced-level damage-related characterization based on [SVECD](#) approach for conventional and [APMs](#). The [APMs](#) considered in this section are rubberized asphalt mixtures.

The summary of conventional and rubberized asphalt mixtures is described in table [4.6](#), respectively. It can be observed that the Stiffness (E^*) for AC10-Rh demonstrates the highest stiffness, especially at elevated temperatures, due to its polymer and rubber combination. For cracking resistance both AC8-Rd and AC10-Rh perform significantly better in terms of [Sapp](#) than AC10 (control), with AC10-Rh showing the best results in [FlexPAVETM](#) v 1.1 simulations. The rutting performance of AC10-Rh consistently outperforms both AC8-Rd and AC10 (control) in resisting rutting and minimizing rut depths at both moderate (20°C) and high (55°C) temperatures. The [RSI](#) for AC10-Rh exhibits the lowest [RSI](#), qualifying it for heavy and very heavy traffic categories. While in terms of [CTC](#) analysis the AC8-Rd and AC10-Rh outperform the AC10 (control) mixtures. It can be summarized that the advanced damage-related characterization using [SVECD](#) theory provides a comprehensive difference between conventional and rubberized asphalt mixtures, including AC8-Rd (rubberized dry mixture) and AC10-Rh (hybrid rubberized mixture), are engineered to improve the performance of conventional mixtures (e.g., AC10 (control)) by addressing critical issues like cracking, rutting, and thermal sensitivity.

Table 4.6: Advanced-level approach using (SVECD) damage-related characterization for conventional and rubberized asphalt mixtures

Mixture ID	StiffnessDiff (%) (E*) (MPa) at 4.0°C (10 Hz)		StiffnessDiff (%) (E*) (MPa) at 40°C (0.1 Hz)		Sapp (%)	Range	Diff(%) Cracking (%)		Diff (%)	RSI (%)	Diff (%) Range		Rut depth (mm)	Diff (%)
AC10 (control)	26131	Control	611	Control	16	Sapp>36: E	Control	73	Control	6.53	Control	RSI<12:S	21.35	Control
AC8-Rd	22205	-15	641	5	19.5	Sapp>30:V	22	57	22	3.99	40	RSI<4:H	18.2	14.75
AC10-Rh	23304	-10	727	19	25	Sapp>8:S	56	44	40	3.54	45	RSI<1:E	17	20.37

4.6 Section-V

4.6.1 Advanced-level approach for damage-related characterization of conventional and RAP mixtures

As a similar approach to the conventional and rubberized asphalt mixtures, advanced-level damage-related characterization of conventional and RAP mixtures was conducted using the PASSFlex system, which involves a systematic testing approach with the AMPT. These mixtures include AC10 (control) as a conventional mixture, AC10-RA50 mixture containing 50 % RAP, AC10-RdRA50 mixture contains 50 % RAP with dry rubber addition, While AC10-RhRA50 containing hybrid mixture with 50 % RAP materials. Please note that these materials have been discussed in chapter 3 of this thesis. This methodology was applied to both conventional and APMs to assess their performance. In this section the RAP mixtures are considered as APMs. The laboratory testing phase consisted of several key tests using the AMPT.

4.6.1.1 Dynamic Modulus Test

It evaluates the stiffness of the conventional and RAP mixtures under varying temperatures and loading frequencies, providing insight into its viscoelastic properties. The AASHTO T 378-22 was utilized to evaluate the stiffness of conventional and RAP mixtures [216, 10, 192].

4.6.1.2 Cyclic fatigue test

It determines the fatigue life of the conventional and RAP mixtures by simulating repeated loading to measure its resistance to cracking over time. The cyclic fatigue utilizes AASHTO TP 133 and SVECD theory to quantify the damage in asphalt mixtures [217, 193, 196].

4.6.1.3 Stress Sweep rutting test

The SSR test assesses the rutting potential of conventional and RAP mixtures by examining the permanent deformation of the mixture under cyclic loads [218, 194]. The SSR test utilizes AASHTO TP 34-19 and the shift model to quantify the viscoplastic behavior of the conventional and RAP mixtures [194].

4.6.2 Damage modeling

After extensive laboratory testing using an [AMPT](#), the data for each mixture was processed using the excel based [FlexMATTM cracking](#) and [FlexMATTM rutting](#) tools for damage quantification. Several advanced modeling techniques were utilized to analyze the viscoelastoplastic aspects of the conventional and [RAP](#) mixtures [134, 219].

4.6.2.1 2s2p1D Model

To assess the stiffness characteristics of conventional and [RAP](#) mixtures, accounted for viscoelastic properties, the [2s2p1D](#) model coded in [FlexMATTM cracking](#) was utilized [201].

4.6.2.2 SVECD Model

The [SVECD](#) model employed in the excel based [FlexMATTM cracking](#) to evaluate fatigue cracking of conventional and [RAP](#) mixes by considering the material's damage response under repeated stress [220, 88, 89].

4.6.2.3 CTC Model

The [CTC](#) model analyzed thermal of the conventional and [RAP](#) mixture's contraction to determine the material's behavior in response to temperature changes and thermal-induced stress [13].

4.6.2.4 Shift model

For rutting evaluation, the [shift](#) model was analyzed by [FlexMATTM rutting](#) tool helping to determine how the material's deformation characteristics change over its lifespan [194]. Finally, after successful data analysis, the results were used in a pavement simulation step, conducted through the [FlexPAVE](#) simulation program [145, 133]. This stage enabled the performance assessment and structural design of the pavement, providing insights for both current and future performance under various conditions. Overall, this section provides a comprehensive analysis of an advanced methodology for damage-related characterization, comparing both conventional and [APMs](#) by leveraging sophisticated modeling and simulation tools.

Finally, after successful data analysis, the results were used in a pavement simulation step, conducted through the [FlexPAVETM v 1.1](#) simulation program. This stage enabled the performance assessment and structural design of the pavement, providing insights for both current and future performance under various conditions. Overall, this section provides a comprehensive analysis of an advanced methodology for damage-related characterization,

comparing both conventional and **APMs** by leveraging sophisticated modeling and simulation tools.

4.6.3 Stiffness analysis using 2s2p1D model for conventional and RAP mixes

The dynamic modulus $|E^*|$ is a critical indicator of stiffness. Mixtures with **RAP** and additives (especially those with rubber and polymer) show distinct performance compared to the control mixtures. Figure 4.41 illustrates the dynamic modulus master curve for conventional AC10 (control) and **RAP** mixtures (AC10-RA50, AC10-RdRA50, AC10-RhRA50) under various conditions. All reclaimed mixes, including AC10-RA50, AC10-RdRA50, and AC10-RhRA50, exhibit stiffness comparable to the AC10 (control) mixtures. This similarity ensures they retain sufficient strength to resist deformation under heavy loads [216, 221]. AC10 (control) mix has Higher $|E^*|$ values at lower frequencies indicate less stiffness and brittleness under slow-loading conditions. This behavior could make it prone to thermal cracking in cold climatic conditions [216]. AC10-RA50 has slightly increased stiffness compared to AC10 (control), reflecting less flexibility. This may be the addition of **RAP** inside the conventional mixture. AC10-RdRA50 **RAP** with **ECR** addition has a significant reduction in $|E^*|$ compared to the AC10 (control) and AC10-RA50, indicating improved flexibility, likely due to the rubber content [222]. AC10-RhRA50 (**RAP** with rubber and polymer) shows the lowest $|E^*|$ at low frequencies, suggesting the best ability to deform without cracking. This mix is expected to exhibit superior resistance to thermal cracking due to its viscoelastic properties. AC10 (control) has higher stiffness at low frequencies, increasing its likelihood of cracking under thermal stress. While AC10-RdRA50 and AC10-RhRA50 demonstrate significantly lower stiffness at low frequencies, enhancing their ability to resist cracking in colder conditions [223]. AC10-RA50 achieves moderate improvements in flexibility but doesn't outperform the other reclaimed mixes. AC10-RdRA50 and AC10-RhRA50 offer superior flexibility while maintaining comparable stiffness at high frequencies, making them ideal mixtures for high-performance pavements. **ECR** additives (AC10-RdRA50) reduce stiffness at low frequencies, enabling flexibility, while maintaining adequate stiffness for load bearing flexible pavements. Polymer and rubber combinations (AC10-RhRA50) further enhance these properties, showing the most balanced performance across all frequency ranges.

As a conclusion the conventional mix AC10 (control) has marginal flexibility under heavy, fast-loading conditions but more resilient to thermal stresses and long-term fatigue. In terms of reclaimed addition (AC10-RA50) has an improvements in stiffness over the AC10 (control) but lacks the advanced properties of modified mixes. In terms of **RAP** and modifiers 50 % **RAP** and **ECR** (AC10-RdRA50) has substantial flexibility improvement, particularly in thermal cracking resistance, while retaining sufficient load-bearing capacity. 50 % **RAP** + **ECR** + Polymer (AC10-RhRA50) has better performance with enhanced flexibility and high strength at traffic frequencies, making it ideal for areas requiring durability and crack resistance. The log-log representation of the Dynamic Modulus $|E^*|$ for the conventional mix AC10 (control) and the three reclaimed asphalt mixes (AC10-RA50, AC10-RdRA50, AC10-RhRA50) is represented in Figure 4.42. The AC10 (Control) has the lowest $|E^*|$ among the mixes, reflecting less stiffer behavior under slow-loading or thermal stresses. This may reduces the likelihood of cracking under thermal expansion [206]. AC10-RA50 has slight improvement in $|E^*|$ compared to AC10 (control), indicating minor improvements [224]. AC10-RdRA50 has more significant reduction in stiffness,

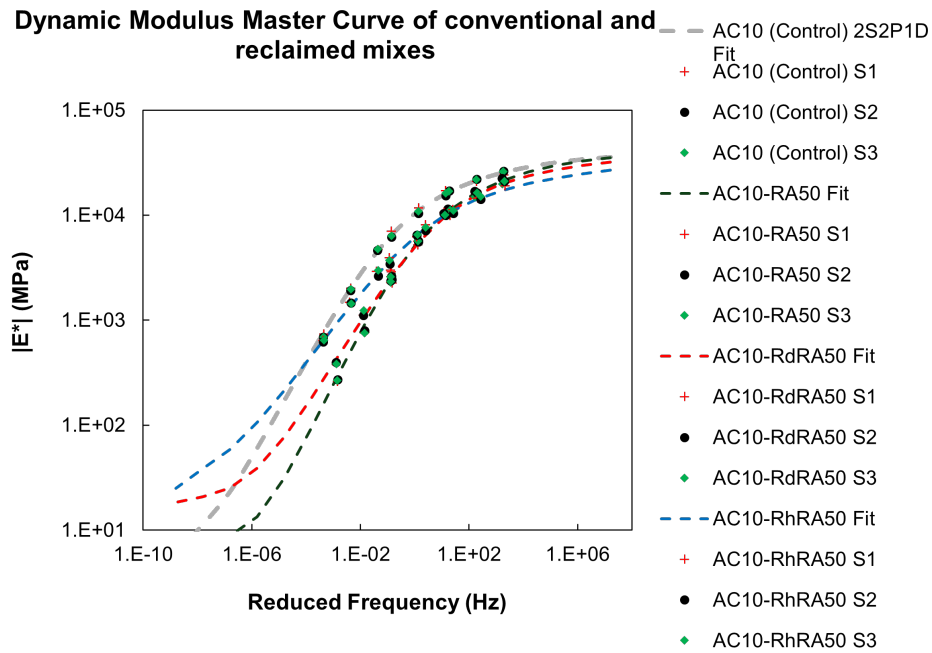


Figure 4.41: Master curve of conventional and reclaimed asphalt mixtures

reflecting enhanced flexibility due to the inclusion of **ECR**. While AC10-RhRA50 has the lowest $|E^*|$ among all mixes, showing superior flexibility due to the combined effects of polymer and rubber additives. As a summary all three modified mixes (AC10-RA50, AC10-RdRA50, AC10-RhRA50) show comparable $|E^*|$ to the AC10 (control) mix. This is crucial for ensuring that the use of modifiers does not compromise structural integrity during fast traffic loads.

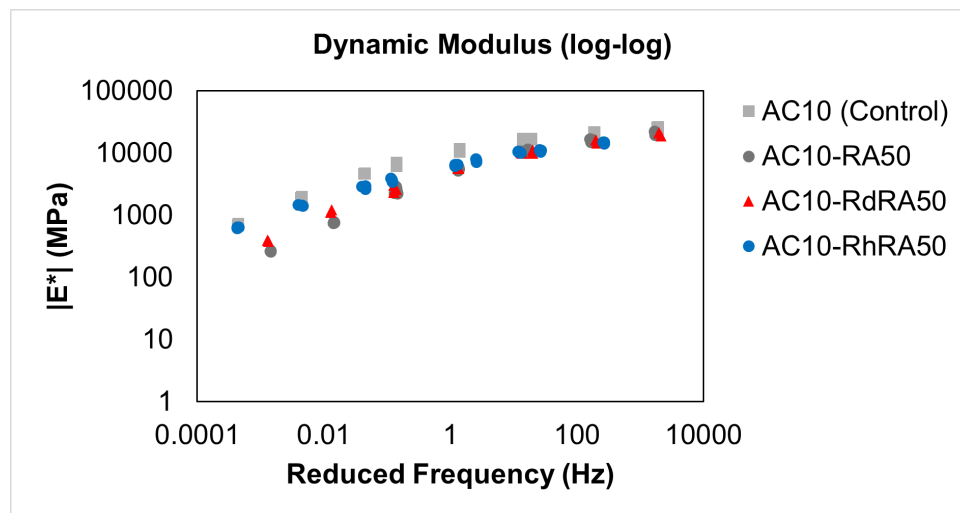


Figure 4.42: Log-Log scale Dynamic modulus analysis for conventional and **RAP** mixtures

4.6.3.1 Phase angle analysis using **2s2p1D** model for conventional and **RAP** mixes

Figure 4.43 shows the phase angle (degrees) as a function of reduced frequency (Hz) for the conventional mix AC10 (control) and the **RAP** mixes (AC10-RA50, AC10-RdRA50, AC10-RhRA50). The phase angle is an essential parameter in evaluating the viscoelastic behavior of asphalt mixtures, representing the delay between stress and

strain under cyclic loading [225]. AC10 (control) mixture displays relatively low phase angles compared to the reclaimed mixes, indicating a more elastic (solid-like) response. This behavior aligns with its higher stiffness and reduced flexibility, which could lead to thermal cracking under slow loading or high-temperature conditions. In reclaimed asphalt mixtures AC10-50 has slightly higher phase angle than the control, reflecting an increased viscous component due to the addition of 50 % RAP. AC10-RdRA50 has significantly higher phase angles at low frequencies shown in figure 4.43, indicating an increased viscous (fluid-like) response from the RAP and ECR modification [12].

The ECR enhances flexibility, making the mix better suited for resisting thermal cracking. AC10-RhRA50 exhibits the highest phase angle among all mixes at low frequencies shown in figure 4.43,, showcasing the most substantial viscous behavior due to the combined effects of rubber and polymer. This mix offers the best flexibility and thermal cracking resistance at lower frequency [12]. At an intermediate frequency all mixes begin to converge in phase angle, indicating a balance of elastic and viscous behavior in this range. AC10-RdRA50 and AC10-RhRA50 consistently exhibit slightly higher phase angles compared to AC10 (control) and AC10-RA50, demonstrating their enhanced viscoelastic properties. While at high reduced frequencies fast loading or low temperatures the AC10 (control) shows a lower phase angle, suggesting a more elastic response suitable for handling heavy, fast-moving traffic loads. The control mix maintains its stiffness and elastic nature under rapid loading. As compared to this AC10-RA50 has Similar phase angle to the control, indicating minimal impact of 50 % RAP on high-frequency performance.

AC10-RdRA50 and AC10-RhRA50 has slightly higher phase angles compared to the control but remain close. This indicates the retained structural capacity of these modified mixes, despite their enhanced flexibility at lower frequencies.

As a conclusion, AC10 (control) is less stiffer, more elastic response but limited flexibility [224, 226]. Suitable for high-load areas but less effective in preventing thermal cracking. AC10-RA50 has slight improvement in flexibility and phase angle balance but lacks the advanced properties of ECR-modified mixes. AC10-RdRA50 with ECR modification improves viscous behavior and flexibility, making it effective for crack resistance while retaining structural integrity. While AC10-RhRA50 is the most balanced performance with superior flexibility at low frequencies and robust elastic behavior at high frequencies. The ideal mixtures for pavements requiring both durability and flexibility.

4.6.3.2 Shift factor analysis using 2s2p1D model for conventional and RAP mixes

Figure 4.44 illustrates the shift factors as a function of temperature for the AC10 (control) mix and the three RAP mixes (AC10-RA50, AC10-RdRA50, and AC10-RhRA50). Shift factors are used in time-temperature superposition principles to construct master curves for viscoelastic materials [227]. These values reflect the

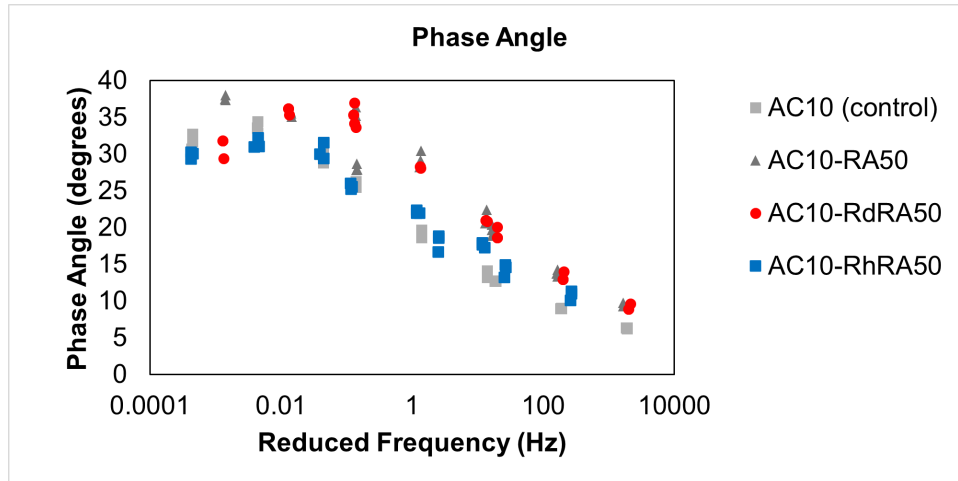


Figure 4.43: Phase angle analysis of conventional and reclaimed asphalt mixes

temperature sensitivity of each mixture's viscoelastic behavior. All mixes show a negative correlation between temperature and shift factors. As temperature increases, the materials become less sensitive to time-dependent deformation due to increased viscous behavior [220].

AC10 (control) exhibits the most negative shift factors across the temperature range. It indicates the highest sensitivity to temperature changes, which can lead to reduced performance in extreme climates (e.g., stiff and brittle in cold temperatures, soft and deformable in hot temperatures) [228]. AC10-RA50 shift factors are slightly less negative than AC10 (control), shown in figure 4.44 indicating slightly improved temperature stability due to the inclusion of RAP [220].

Temperature sensitivity remains relatively high compared to rubber- or polymer-modified mixes. AC10-RdRA50 shift factors are moderately less negative compared to AC10 (control) and AC10-RA50 shown in figure 4.44. ECR and RAP modification enhances temperature stability by adding viscoelastic properties, making this mix less prone to extreme performance shifts across temperature ranges. AC10-RhRA50 shows the least negative shift factors across the range, demonstrating the lowest sensitivity to temperature variations. The combination of rubber and polymer provides the most stable behavior, effectively balancing stiffness and flexibility over a wide temperature range [228].

As a conclusion, to shift factors of conventional and RAP mixes, the AC10 (control) are highly sensitive to temperature variations; likely to perform poorly in extreme climates. AC10-RA50 has marginal improvement in temperature stability, but still relatively sensitive. While AC10-RdRA50 has a substantial enhancement in temperature insensitivity, suitable for varied temperature conditions. AC10-RhRA50 is the most temperature-stable mix, ideal for regions experiencing extreme or fluctuating climates.

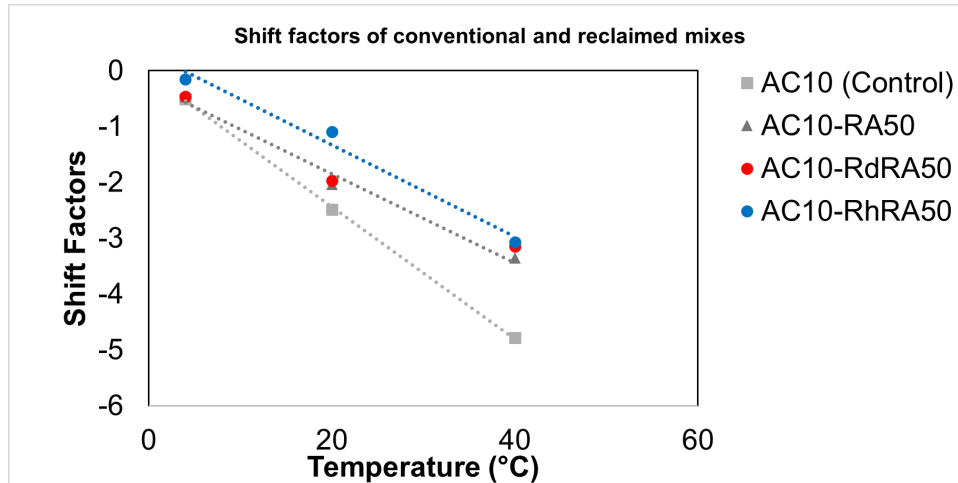


Figure 4.44: Shift factors analysis of conventional and reclaimed asphalt mixtures

4.6.4 Fatigue cracking analysis using SVECD model for conventional and RAP mixes

Figure 4.45 illustrates the C (pseudostiffness) versus S (damage parameter) curves for conventional AC10 (control) and RAP mixes (AC10-RA50, AC10-RdRA50, AC10-RhRA50). The C value decreases as the S value increases, indicating growing fatigue damage [227]. Pseudostiffness C represents the material integrity; a sharp drop means faster damage accumulation [227]. The control mixtures AC10 (control) shows moderate damage progression with relatively higher pseudostiffness at lower S. Its lifespan is shorter compared to modified mixes but performs better than AC10-RA50 [186]. The AC10-RA50 With 50 % RAP content, this mix demonstrates slightly accelerated damage accumulation lower C at a given S. The curve flattens earlier, suggesting a quicker reduction in structural integrity compared to the control mixture [186]. The rubberized dry mix containing ECR with addition of 50 % RAP AC10-RdRA50 mix has higher initial damage progression compared to the control but remains consistent. It demonstrates less long-term resistance to damage, as C values decline sharply with increasing S. This behavior aligns with the findings in the VEPCD theory, which states RAP mixes without sufficient rejuvenation exhibit quicker microcrack propagation [12]. AC10-RhRA50 mix performs significantly better in resisting damage accumulation. The pseudostiffness C decreases more gradually with increasing S, extending fatigue life considerably. The slower degradation aligns with better healing properties and improved load distribution mechanisms, potentially due to the use of an effective rejuvenating agent.

As a summary the SVECD model utilizes the C Vs S curve to capture fatigue damage progression. AC10-RhRA50 shows the most favorable behavior due to slower microcrack growth. RAP mixes (AC10-RA50, AC10-RdRA50) exhibit quicker damage accumulation primarily because aged RAP binder reduces flexibility and healing capacity. The healing effects of AC10-RhRA50 likely incorporates a rejuvenating agent that enhances healing during rest periods, reducing damage rate over time, as reflected in its extended curve [186].

As a conclusion, AC10-RhRA50 outperforms all other mixes in fatigue resistance and damage mitigation, suggesting superior healing properties and long-term durability. AC10-RdRA50 improves slightly over the control

but is less effective than AC10-RhRA50. AC10-RA50 demonstrates poorer performance due to the aged binder's limited flexibility.

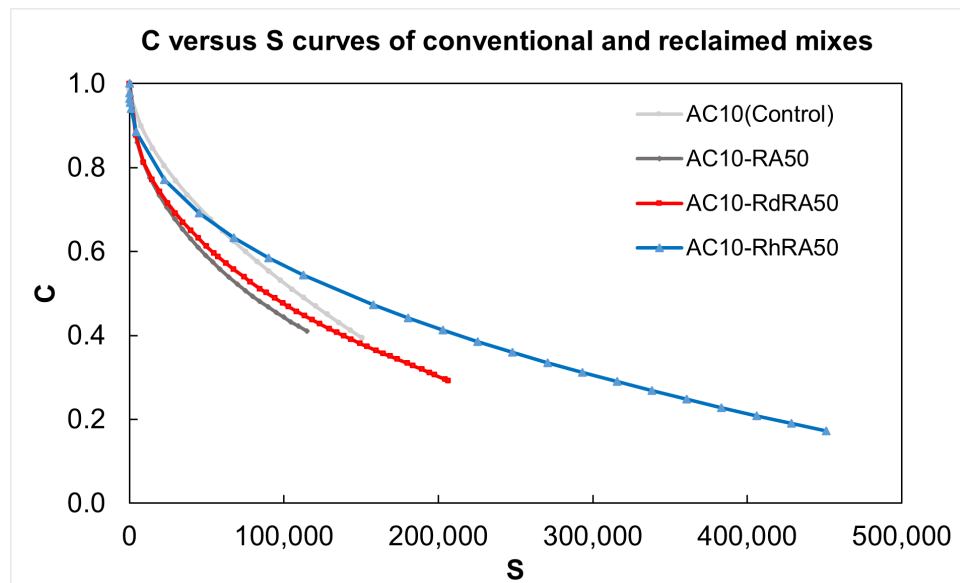


Figure 4.45: C Vs S curve of conventional and reclaimed asphalt mixes

4.6.4.1 Sapp analysis for conventional and RAP mixes

In general, the [Sapp](#) is the application of load Cycles to the corresponding failure of the asphalt material under varying traffic conditions, as defined by ESALs (equivalent single axle loads) [229]. Figure 4.46 shows the [Sapp](#) of conventional and [RAP](#) mixes. AC10 (control) represents the baseline performance for conventional asphalt. It is classified for Standard traffic ([Sapp](#)

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8), indicating suitability for lower ESAL traffic (<10 million ESALs) [229]. It can be seen in figure 4.46 that rubber with polymer significantly improve [RAP](#) mixes' fatigue life, with AC10-RhRA50 performing the best. While only [RAP](#) (AC10-RA50), exhibit reduced fatigue life due to stiff and aged binder from [RAP](#) as compared to the AC10 (control) mix. AC10-RA50 shows a 10 % reduction in load cycles to failure compared to the control, performing worse under traffic conditions [230]. AC10-RhRA50 qualifies for heavy traffic conditions, making it suitable for higher ESAL loads. AC10-RdRA50 demonstrates improved performance but falls short of the heavy traffic threshold. AC10-RA50 remains in the standard category and underperforms compared to the control mix.

As a conclusion, AC10-RhRA50 offers the best balance of strength and durability for heavy traffic conditions ([Sapp](#)

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24). Proper selection of rubber and polymers with [RAP](#) is critical to achieving performance gains in [RAP](#) mixes. [RAP](#) with rubber and polymers addition can be a sustainable alternative for heavily traffic pavements, aligning

with findings from the [SVECD](#) model report regarding healing and durability improvements.

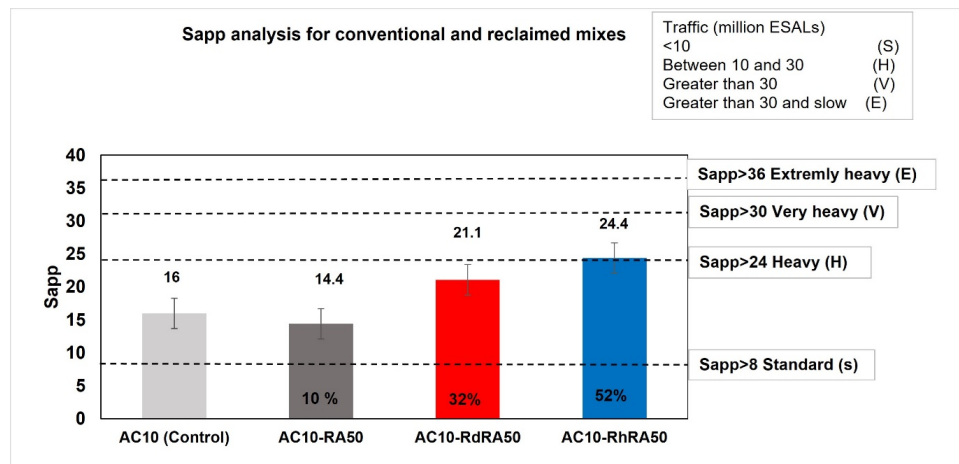


Figure 4.46: Sapp analysis of conventional and reclaimed mixes

4.6.4.2 D^R Failure criterion of Conventional and reclaimed asphalt mixes

Figure 4.47 shows a linear relationship between Sum (1-C) and Nf (fatigue life) for the AC10 (control) mixtures as it has already been discussed in 4.5. The purpose of the figure 4.47 is to show the comparison of failure between the conventional and [RAP](#) mixes. As it indicates that the damage accumulation process follows a predictable trend as the number of cycles to failure increases. Sum (1-C) quantifies the cumulative damage in the material over its fatigue life [231]. The higher the value, the more damage has been accumulated, reflecting the progression toward failure [219]. As Nf increases, the cumulative damage also increases, which is expected as the mixture undergoes repeated cyclic loading.

The fatigue life (Nf) represents the number of cycles the material can endure before failure [232]. In this case, the AC10 (control) mixture shows Nf values between 43,000 and 45,000 cycles under the specific testing conditions. The data points align closely with the fitted trend line, suggesting a reliable relationship between cumulative damage and fatigue life for the AC10 (control) mixture [232]. This validates the predictive models used in the analysis. The fatigue life of 43,000–45,000 cycles indicate the AC10 (control) mixture's limitations in withstanding repeated loading. For the [RAP](#) with conventional mixture, Figure 4.48 evaluates the fatigue damage behavior of AC10-RA50 (50 % [RAP](#) mix), using results from three specimens. All three specimen show similar performance, with moderate Nf and higher cumulative damage [222]. In [RAP](#) mixes appears early damage propagation due to [RAP](#) binder stiffness or localized material variability The trend line shows a clear linear relationship between Sum(1-C) and Nf [196]. This validates the damage characteristic curve from the [SVECD](#) model for AC10-RA50, aligning with the [VEPCD](#) approach for fatigue life prediction.

In summary AC10-RA50, mixed with 50 % [RAP](#), accumulates fatigue damage faster due to the aged binder's limited flexibility and healing capacity. The linear trend between Sum(1-C) and Nf confirms that the fatigue damage

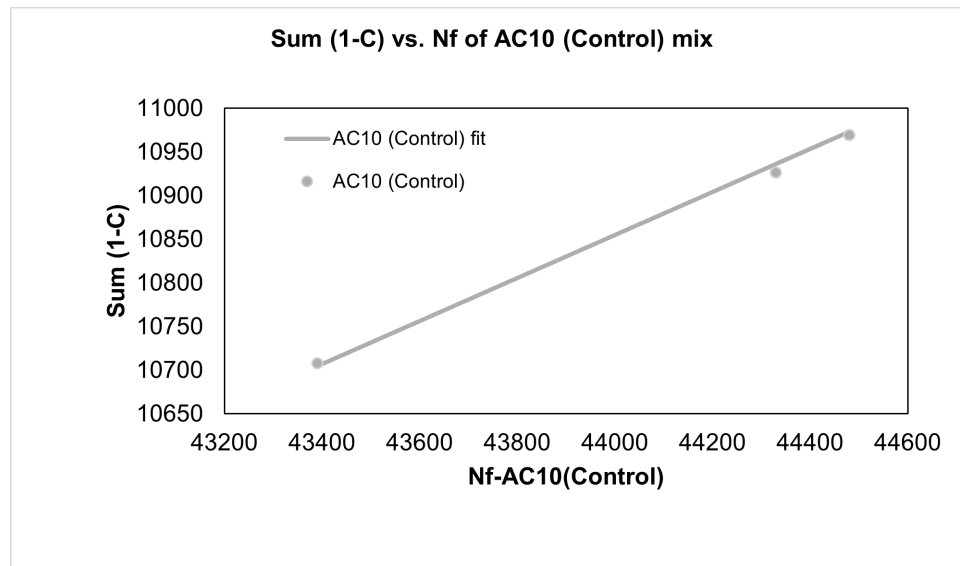


Figure 4.47: Failure analysis of AC10 (control) mixture

progression in AC10-RA50 aligns with the **SVECD** framework. Incorporating 50 % **RAP** in the AC10-RA50 mixture accelerates crack growth, reducing the number of cycles to failure (Nf) compared to modified or hybrid mixes with addition of 50% **RAP**. **RAP** stiffens the mixture, which can improve initial stiffness but reduces fatigue resistance due to the aged binder.

In summary as compared to the AC10 (control) mixes AC10-RA50 specimens (S1, S2, S3) demonstrate earlier damage initiation and faster progression, evidenced by lower Nf values and steeper damage accumulation trends [196]. For improved fatigue performance, introducing strategies like rubberized hybrid additions (as in AC10-RhRA50) would likely mitigate variability and extend the fatigue life of **RAP** mixes [196]. The observed linear trend confirms the applicability of the **SVECD** model for characterizing **RAP** mix fatigue behavior. This relationship allows for more accurate predictions of fatigue life using **FlexPAVETM** v 1.1 pavement simulation program. Generally, sum(1-C) reflects the total damage accumulation as the fatigue load cycles increase. Higher

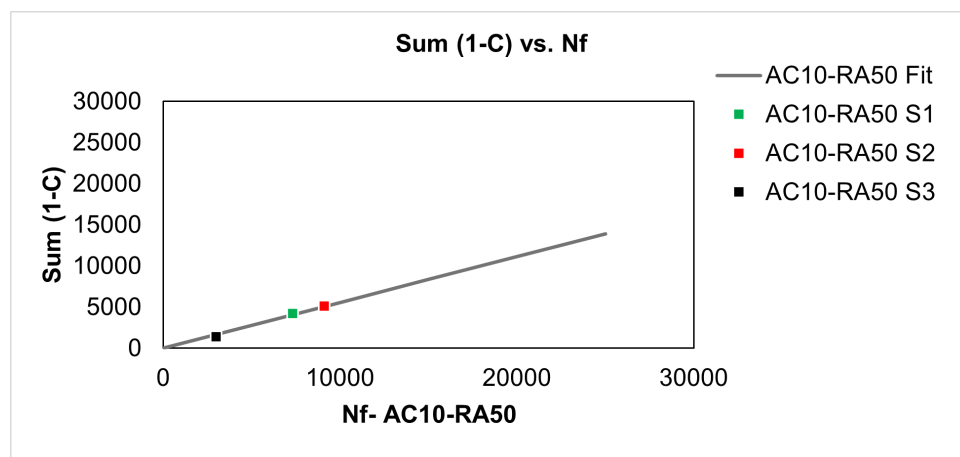


Figure 4.48: Failure analysis of AC10-RA50 mixture

Nf corresponds to greater cumulative damage before failure [220]. Figure 4.49 of AC10-RdRA50 shows a clear

improvement over AC10-RA50. ECR addition with RAP helps reduce early damage progression, resulting in higher Nf values [196]. The linear relationship between Nf and Sum(1-C) confirms the validity of the damage characteristic curve for AC10-RdRA50. AC10-RdRA50 shows enhanced fatigue resistance compared only RAP mixes. The rubberized dry addition with RAP effectively mitigates early damage accumulation and extends mix durability [230].

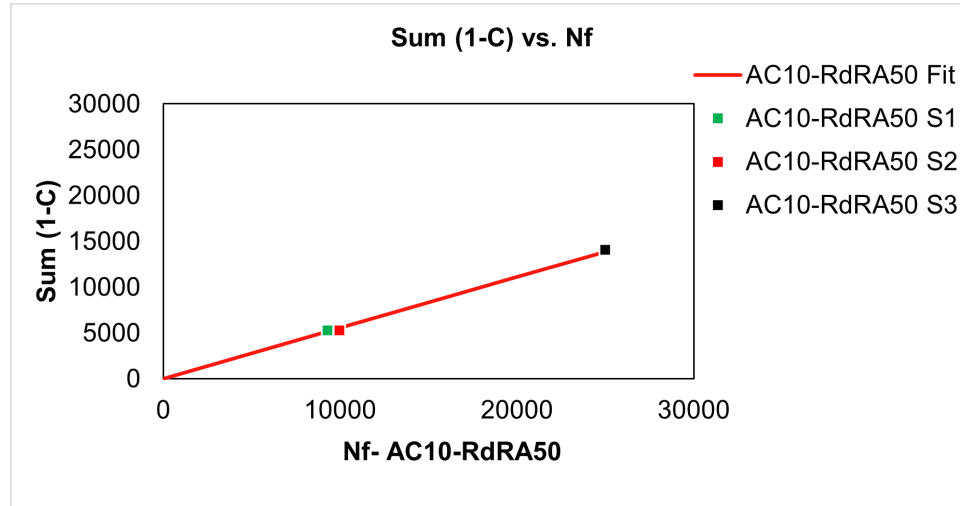


Figure 4.49: Failure analysis of AC10-RdRA50 mixture

For further performance gains, rubber-polymer hybrid modifications with addition of RAP shown in figure 4.50, (AC10-RhRA50) may provide even better fatigue resistance, as previously observed in C vs S curve [206]. It can be seen in figure 4.50 the relationship between Sum (1-C) (cumulative damage indicator) and Nf (number of cycles to failure) for the AC10-RhRA50 mixture. The results are displayed for three specimens (S1, S2, and S3), with a fitted trend line representing the overall behavior of the mixture.

A clear linear relationship is observed between Sum (1-C) and Nf, indicating consistent behavior of the AC10-RhRA50 mixture under cyclic loading. The fitted trend line captures the cumulative damage progression accurately across all three specimens. The AC10-RhRA50 mixture demonstrates significantly higher Nf values compared to AC10-RA50 (50 % RAP only). For example: AC10-RA50 had Nf values ranging from 5,000 to 10,000 cycles. AC10-RhRA50 achieves Nf values between 18,000 to 30,000 cycles. This improvement highlights the beneficial effect of combining rubber and polymers with RAP to improve fatigue resistance [206]. The combination of rubber and polymers with 50 % RAP significantly enhances the fatigue resistance, achieving Nf values nearly 2-3 times higher than AC10-RA50.

The modified binder with polymer and modification of mixture with ECR by dry process with RAP reduces crack initiation and slows crack propagation. AC10-RhRA50 shows lower cumulative damage (Sum 1-C) compared to AC10-RA50, highlighting improved energy dissipation and viscoelastic recovery properties. While AC10-RhRA50 does not match the performance of AC10 Control, it may provide a cost-effective balance between sustainability (use of RAP) and fatigue resistance [12].

As a summary the AC10 (control) has a better fatigue resistance but lacks RAP sustainability benefits. While only the RAP addition AC10-RA50 suffers from low fatigue life and rapid crack progression. The dry addition of rubber with RAP provides better fatigue resistance as compared to the control and only RAP mix. AC10-RhRA50 is a balanced solution with enhanced fatigue resistance compared to AC10-RA50 due to the hybrid modification, making it suitable for moderate to high-traffic applications. Further analysis can be made after utilizing these data in FlexPAVETM v 1.1 simulation program to analyze the fatigue life of these mixtures.

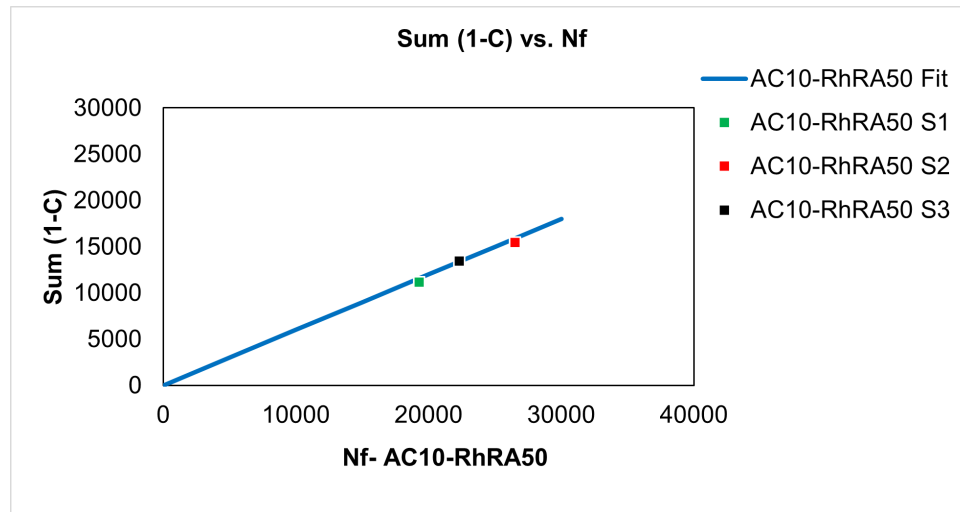


Figure 4.50: Failure analysis of AC10-RhRA50 mixture

4.6.5 CTC analysis of conventional and RAP mixes

The coefficient of the thermal contraction for conventional and RAP mixes can be analyzed in figure 4.51. It shows the variation of CTC with temperature for different asphalt mixtures AC10 (control), AC10-RA50, AC10-RdRA50, and AC10-RhRA50. The CTC value reflects the material's thermal contraction behavior as temperatures decrease, which is critical for evaluating the risk of thermal cracking. All mixtures show an increase in CTC as the temperature drops below 0°C. This behavior indicates greater thermal contraction at colder temperatures, which can lead to cracking in pavements if not managed effectively [211, 233]. AC10 (control) shows lower CTC values (3.5E-05 to 4.5E-05 1/°C) but higher susceptibility to thermal cracking due to its lack of modification. AC10-RA50 has the lowest CTC values (3.0E-05 to 4.0E-05 1/°C), indicating a brittle nature caused by the aged RAP binder. While this reduces thermal contraction slightly, it increases the material's vulnerability to cracking due to stiffness. AC10-RdRA50 displays moderate CTC values (4.5E-05 to 6.0E-05 1/°C).The inclusion of rubber reduces thermal contraction compared to conventional mixtures, improving thermal crack resistance [234]. AC10-RhRA50 exhibits the highest CTC values (6.0E-05 to 7.5E-05 1/°C). The hybrid modification with ECR and RAP improves flexibility and thermal contraction control, minimizing stress buildup during temperature changes.

As a summary ECR and polymer modifiers with RAP addition significantly reduce the risks associated with

thermal contraction by increasing flexibility and stress relaxation. AC10-RhRA50 performs best in managing thermal contraction stresses, making it ideal for cold climates where thermal cracking is prevalent. AC10-RA50, while sustainable, is more prone to thermal cracking due to the aged and stiff RAP binder, highlighting the need for rejuvenators or modifiers.

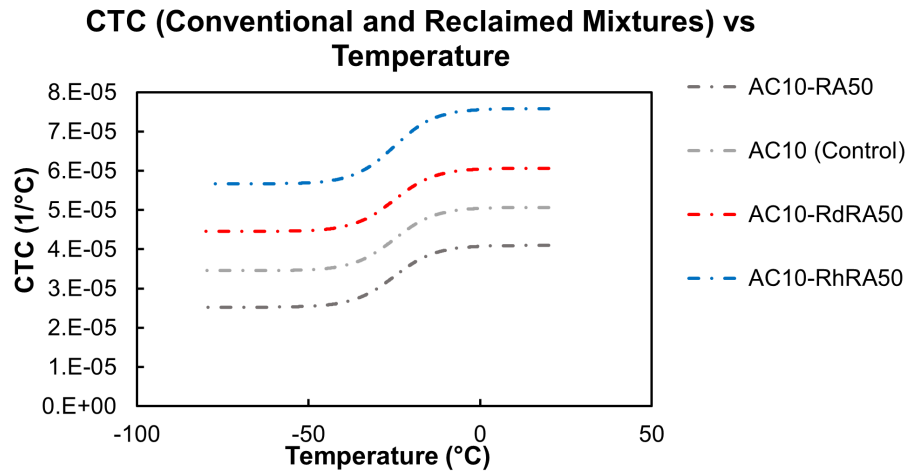


Figure 4.51: CTC analysis of conventional and RAP mixtures

4.6.6 Rutting analysis of conventional and RAP mixes

The accumulated permanent deformation for conventional and RAP mixes investigated for high temperature using the SSR test conducted at 55°C are shown in figure 4.52. This test evaluates the rutting resistance of the mixtures under repeated stress applications over 600 cycles [186]. Permanent deformation increases with stress cycles for all mixtures. The rate and magnitude of deformation depend on the mixture composition [218]. AC10 (control) exhibits the highest accumulated permanent deformation at 27,894 (μ_{evp}), indicating poor resistance to rutting under high temperatures. AC10-RA50 (50 % RAP) shows reduced deformation 17,904 (μ_{evp}) compared to AC10 Control. RAP inclusion increases mixture stiffness, improving rutting resistance but reducing flexibility. AC10-RdRA50 further reduces deformation to 16,502 (μ_{evp}) due to the addition of ECR, which enhances stress distribution and flexibility. AC10-RhRA50 demonstrates the lowest deformation at 11,679 (μ_{evp}) showcasing excellent rutting resistance. The combination of rubber and polymer modifiers provides enhanced stiffness and improved stress dissipation [235].

The conventional mixture AC10 (control) performs poorly under repeated stress, with high accumulated deformation indicating low rutting resistance. Incorporating 50 % RAP improves rutting resistance compared to AC10 (control) due to increased stiffness. However, the aged binder in RAP makes the mixture more brittle, limiting its overall flexibility [236]. Adding ECR to RAP enhances flexibility and stress dissipation, reducing deformation further compared to AC10-RA50. AC10-RhRA50, hybrid modification (combining ECR and polymers) results in the lowest permanent deformation, offering superior rutting resistance. This mixture combines stiffness and flexibility, making it the best-performing option for high-stress and high-temperature conditions [132].

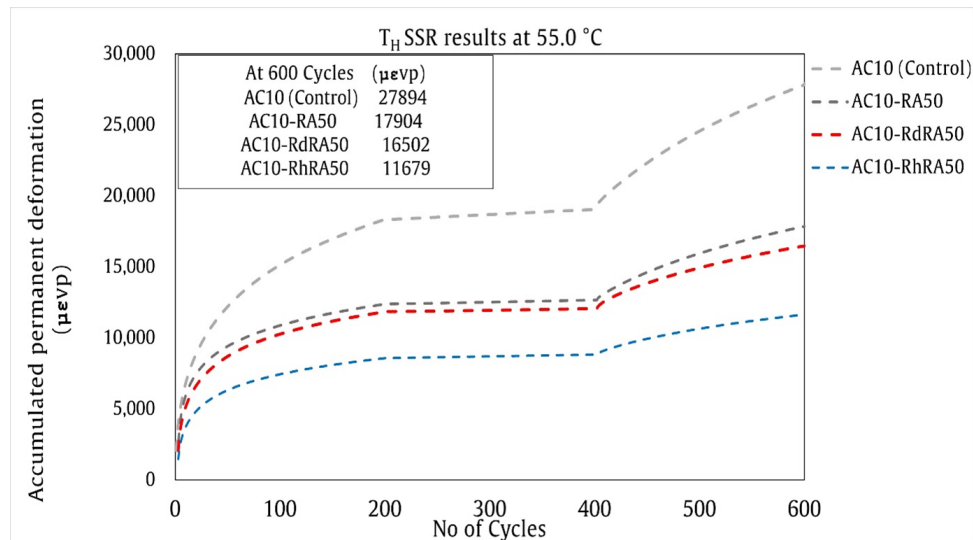


Figure 4.52: High temperature permanent strain analysis of control and reclaimed mixtures

As a conclusion AC10-RhRA50 provides the best rutting resistance, with 58 % less deformation compared to AC10 (control) mixtures. AC10-RdRA50 also shows significant improvement due to ECR modification, while AC10-RA50 provides moderate rutting resistance. AC10 (control) demonstrates the highest deformation, indicating it is less suitable for high-temperature, heavy-load applications.

The accumulated permanent deformation under the SSR test at low temperature (20°C) for conventional and RAP mixtures i-e, AC10 (control), AC10-RA50, AC10-RdRA50, and AC10-RhRA50 has been analyzed in figure 4.53. The deformation is tracked across 600 loading cycles. Permanent deformation accumulates gradually for all mixtures under lower temperature conditions (20°C) [218]. The rate and magnitude of deformation depend on the mixture type and its composition.

AC10 (control) exhibits the highest accumulated deformation at 3946.5 (μ_{evp}), indicating poor rutting resistance under low temperatures [132]. AC10-RA50 (50 % RAP) shows reduced deformation 2792.5 (μ_{evp}) compared to AC10 (control) due to the stiffened aged binder in RAP. Despite improved resistance to rutting, its flexibility remains limited. AC10-RdRA50 ECR and RAP further decreases to 1881 (μ_{evp}), demonstrating enhanced performance from rubber modification, which improves stress dissipation and flexibility. AC10-RhRA50 (Hybrid Rubberized RAP) exhibits the lowest deformation at 1803.5 (μ_{evp}), showcasing superior rutting resistance under moderate stress and temperature. The combination of rubber and polymer modifiers significantly reduces deformation [236].

AC10 (control) displays the poorest rutting resistance due to its conventional binder, which cannot withstand repeated stress effectively at low temperature. AC10-RA50 Incorporating 50 % RAP improves rutting resistance compared to AC10 (control) due to the increased stiffness from the aged binder. However, this stiffness limits flexibility, increasing susceptibility to cracking [235]. AC10-RdRA50 the addition of ECR enhances flexibility and stress distribution, significantly improving rutting resistance. AC10-RhRA50 combines ECR and polymer

modifiers with **RAP**, achieving the lowest permanent deformation. The mixture effectively balances stiffness and flexibility, providing the best rutting resistance under moderate temperature [132].

As a conclusion AC10-RhRA50 offers the lowest accumulated permanent deformation, reducing rutting by 54 % compared to AC10 (control). AC10-RdRA50 also shows substantial improvement, reducing deformation by 52 % relative to AC10 (control). AC10-RA50 performs better than the conventional mixtures but remains less effective than rubber-modified mixtures. AC10 (control) performs the worst under **SSR** test conditions at 20°C, indicating its limitations for moderate climates.

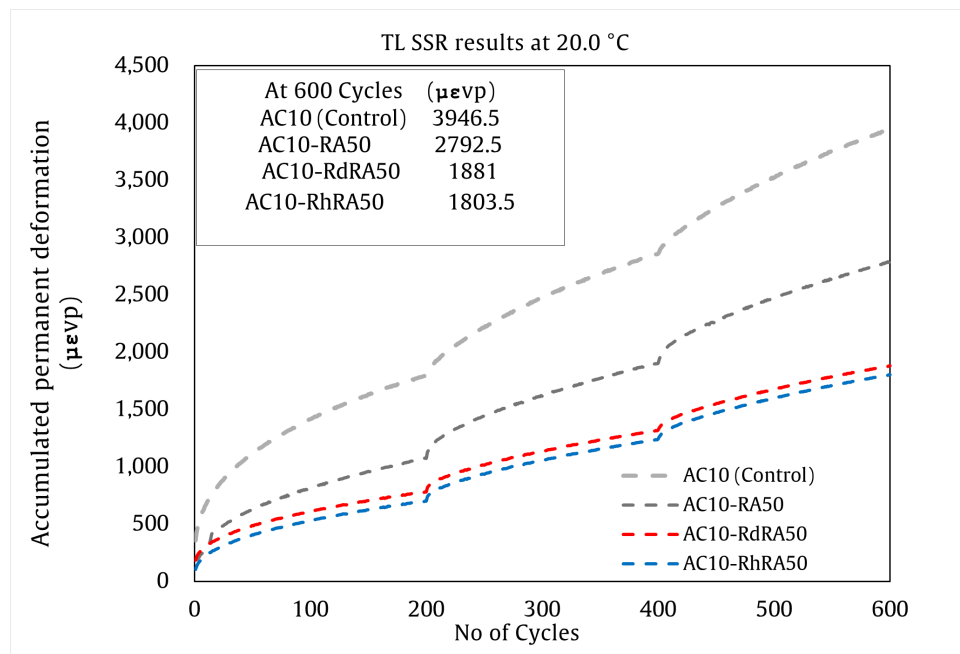


Figure 4.53: Low temperature permanent strain analysis of control and **RAP** mixtures

4.6.6.1 **RSI** analysis of conventional and **RAP** mixtures

The **RSI** results for various asphalt mixtures, conventional AC10 (control), and **RAP** mixtures (AC10-RA50, AC10-RdRA50, and AC10-RhRA50) has been analyzed in figure 4.54. The **RSI** reflects the susceptibility of each mixture to rutting under repeated loading, where lower **RSI** values indicate better resistance. AC10 (control) exhibits the highest **RSI** value of 6.53 %, reflecting poor rutting resistance. It fails to meet the requirements for heavy or very heavy traffic categories, making it the least suitable mixture for high-stress conditions. AC10-RA50 shows significant improvement, with an **RSI** value of 3.8 %, which is 41 % lower than the AC10 (control). The stiffer binder from the inclusion of **RAP** enhances rutting resistance but may reduce flexibility, limiting its performance in high-stress conditions [132]. The addition of **ECR** to **RAP** reduces the **RSI** value to 2.82 %, achieving a 56 % reduction compared to the AC10 (control) [179]. **ECR** modification improves flexibility and stress dissipation, significantly enhancing rutting resistance. AC10-RhRA50 demonstrates the best rutting resistance, with an **RSI** value of 2.37 %. This is a 63 % reduction compared to the AC10 (control). The hybrid modification (combination of **ECR** and polymer) provides optimal balance between stiffness and flexibility, resulting in superior rutting

performance [237].

As a summary, AC10 (control) has the highest RSI, indicating poor rutting resistance. AC10-RA50 shows moderate improvement due to RAP inclusion but does not meet very heavy traffic requirements. AC10-RdRA50 achieves significantly better rutting resistance with rubber modification. AC10-RhRA50 performs the best, with the lowest RSI, making it suitable for very heavy and even extremely heavy traffic conditions.

As a conclusion, rubberized and hybrid-modified mixtures, particularly AC10-RhRA50, offer superior rutting resistance compared to the conventional AC10 (control). The combination of ECR and polymers effectively reduces rutting strain, ensuring the mixture performs well under heavy and extreme traffic conditions.

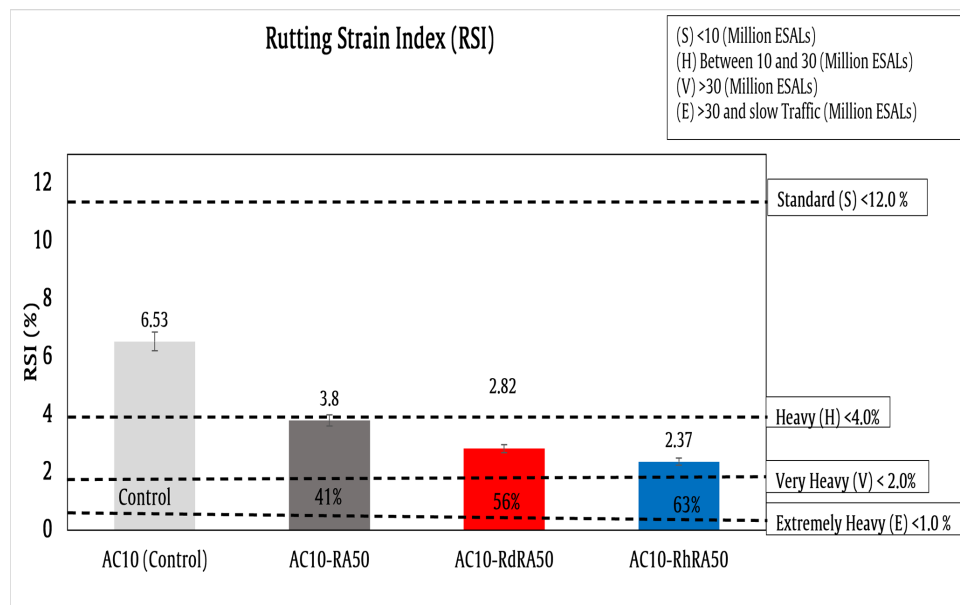


Figure 4.54: Rutting strain index (RSI) analysis of conventional and reclaimed asphalt mixtures

4.6.7 Pavement performance analysis of conventional and RAP mixtures

To predict the performance of conventional and RAP mixtures the FlexPAVETM v 1.1 was utilized by incorporating the data from FlexMAT cracking and rutting evaluating the pavement performance of asphalt mixtures under various loading, temperature, and environmental conditions [220]. The analysis integrates models like VEPCD theory and other performance-based simulations to assess the behavior of conventional and RAP mixtures. Key performance parameters such as rutting depth, cracking percentage, and stiffness are analyzed to quantify the long-term durability and performance of pavement structure over its design life. The following key performance factors will be further analyzed for the performance of the conventional and reclaimed asphalt mixtures in a pavement structure.

4.6.7.1 Fatigue Cracking

Fatigue cracking results from repeated traffic loads, causing microcracks that grow into macro-scale damage. The [SVECD](#) model is employed to predict the initiation and propagation of these cracks [12]. The outputs from [FlexMAT](#) cracking are further utilized in [FlexPAVETM](#) v 1.1 for pavement simulation. The model quantifies cumulative damage and stiffness reduction, linking them to the fatigue resistance of the conventional and [RAP](#) mixtures.

4.6.7.2 Rutting (Permanent Deformation)

Rutting is modeled using the [shift](#) model, which evaluates the accumulation of permanent deformation for conventional and [RAP](#) mixtures under high temperatures and traffic stresses [12]. This includes factors such as loading time, temperature, and viscoelastic properties of asphalt. Modified mixtures (e.g., those with polymers and [ECR](#)) along with [RAP](#) exhibit better resistance to rutting due to their enhanced viscoelastic and viscoplastic behaviors.

4.6.7.3 Healing and Recovery

Healing, a unique property of asphalt mixtures, refers to the material's ability to recover stiffness during rest periods. Please note the version of [FlexPAVETM](#) v 1.1 does not support the healing model. Therefore, this research work does not include the healing model.

4.6.8 Damage analysis of conventional and RAP mixtures

The damage factor provides a quantitative measure of damage severity within the pavement layer and is usually tied to the color intensity in the contour plots (red areas indicate the highest damage factors) [12]. The damage factor distribution for conventional mixture AC10 (control) can be analyzed in figure 4.55. The peak damage factor is concentrated within the red zone, extending from the surface to a depth of 7 inches. Horizontal distribution is widespread (± 30 inches), indicating that the AC10 (control) mixture experiences more damage over a larger area [230]. From the figure 4.55 it can be analyzed that the conventional material lacks confinement, resulting in greater stress dispersion both horizontally and vertically. This leads to higher cumulative damage factors in the pavement, reducing its lifespan. As AC10 (control) exhibits the largest and deepest damage contours. This can be concluded as the red zone transitions gradually to yellow, indicating that the material absorbs and spreads the load, but at the cost of higher overall damage.

The addition of 50 % [RAP](#) with the conventional mixture, can be analyzed in figure 4.56, the red zone still extends to about 6.5–7 inches, like the AC10 (control) mixture. The horizontal spread of damage is reduced to

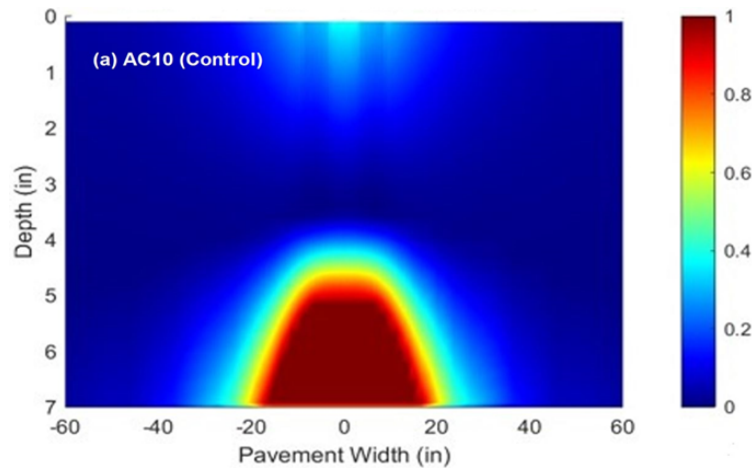


Figure 4.55: Damage analysis of conventional asphalt mixtures

± 20 –25 inches, indicating a more localized load distribution [12, 238]. The concentration of damage is slightly improved compared to the AC10 (control) mixture, with sharper transitions from red to yellow zones. Incorporating 50 % RAP reduces horizontal spread but does not significantly improve vertical damage resistance. AC10-RA50 shows slightly more focused damage zone compared to the control. The depth of damage is still significant (6-7 inches), but the damage is more localized in the central region [239].

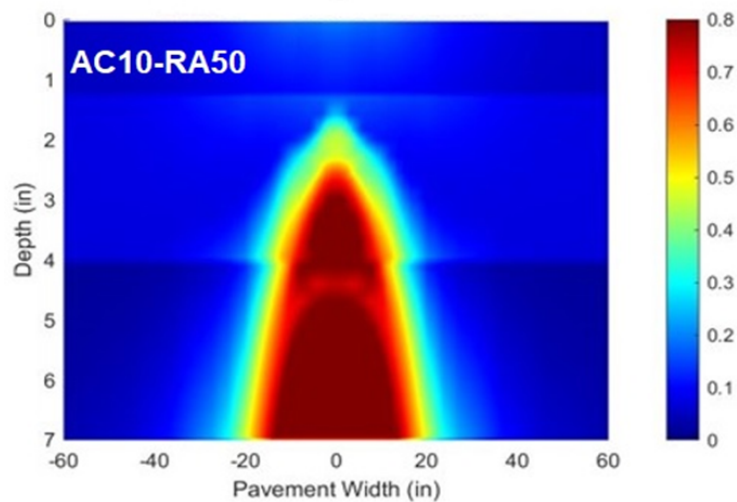


Figure 4.56: Damage analysis of AC10-RA50 mixture

As a conclusion, RAP enhances stiffness, limiting the horizontal spread of damage. However, the stiffness also makes it susceptible to brittle behavior, causing similar depths of peak damage factors compared to the AC10 (control) mixture.

In figure 4.57 for AC10-RdRA50 mixture the damage is concentrated in a much smaller central region. The depth of damage is reduced (5-6 inches). It indicates that the rubberized dry ECR addition with ECR might improve resistance to horizontal spread and depth of damage [240]. From figure 4.57 the damage transitions are steeper (red to yellow occurs rapidly), highlighting an ability to resist deformation beyond the concentrated load

zone. Rubberized dry addition **ECR** improves performance by reducing both vertical and horizontal damage. The addition of **ECR** (dry method) introduces more flexibility and crack resistance [179]. Damage factors decrease with depth, as the material better dissipates stress in the load path. Although the addition of rubber improves elasticity and energy dissipation, reducing both vertical penetration and horizontal stress spread. This mixture resists cracking better than **RAP**-only mixtures.

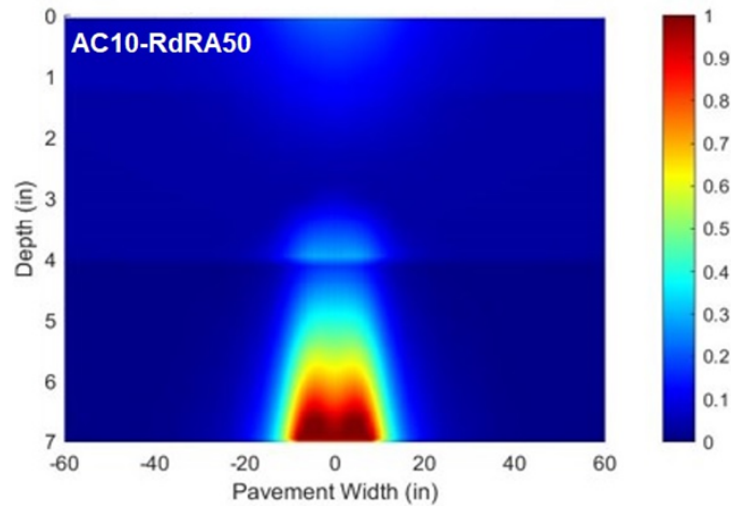


Figure 4.57: Damage analysis of AC10-RdRA50 mixture

The smallest and most concentrated red zone, confined to the immediate load area shown in figure 4.58 for AC10-RhRA50 mixture. Peak damage is limited to 4–5 inches, the least among all mixtures, indicating superior vertical resistance. Horizontal spread is minimal, within ± 10 inches, reflecting excellent lateral confinement. The steepest gradient from red to yellow, indicating exceptional resistance to stress propagation. The combination of **ECR** and polymers enhances both elasticity and stiffness, achieving a balanced response [241]. This mixture resists both vertical and horizontal stress effectively, making it the most durable and damage resistant [242].

As a conclusion AC10 (control) demonstrates the poorest performance due to its inability to localize stress and prevent deep damage penetration. This indicates a need for material modification. 50 % **RAP** mixture (AC10-RA50), Incorporation of **RAP** reduces horizontal spread but does not significantly improve vertical resistance, making it only moderately better than the control mixture. Rubberized dry addition **ECR** (AC10-RdRA50) adding **ECR** using the dry method with **RAP** significantly enhances damage resistance. This mixture confines both vertical and horizontal damage effectively, reflecting a good balance between stiffness and elasticity. rubberized hybrid Addition (AC10-RhRA50) the hybrid combination of rubber and polymer delivers the best performance by confining damage within a highly localized zone, reducing peak damage factors, and enhancing stress resistance. It is the most durable option. Addition of **ECR** with **RAP** enhances elasticity and crack resistance, helping localize stress. While the hybrid modifications further improve the balance between stiffness and flexibility, offering the best performance.

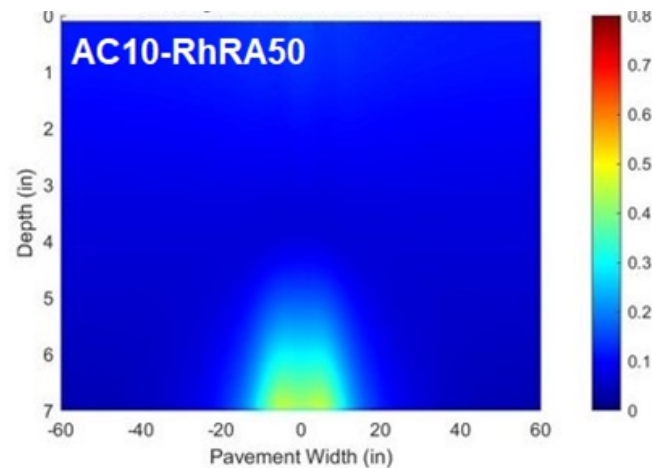


Figure 4.58: Damage analysis of AC10-RhRA50 mixture

4.6.9 Rut depth analysis of conventional and RAP mixes

The rut depths resulted from FlexPAVETM V 1.1 simulations can be visualized in figure 4.59. The figure 4.59 illustrates the predicted rut depth (in mm) for conventional and RAP mixtures under specified loading conditions, emphasizing improvements achieved through RAP, Polymer and ECR modifications. The conventional mixture AC10 (Control) has a rut depth of 21.35 mm (highest among all mixtures). This baseline mixture demonstrates the least resistance to permanent deformation, as evidenced by the highest rut depth [220]. It lacks additives to improve stiffness or elasticity, making it prone to rutting under repeated loads.

AC10-RA50 (50 % RAP) has a rut depth of 17.6 mm (17.56 % reduction compared to the control). The inclusion of 50 % RAP reduces rutting significantly due to increased stiffness [243]. However, the improvement is modest as the RAP content alone does not address elasticity, limiting resistance to permanent deformation under high loads. AC10-RdRA50 (rubberized dry addition of ECR with RAP) has a rut Depth of 16.2 mm (24.12 % reduction compared to the control). ECR with RAP offers a notable improvement in rutting resistance. The rubber enhances elasticity and energy dissipation, effectively reducing the depth of permanent deformation [230].

AC10-RhRA50 (rubberized hybrid addition with RAP) has a rut depth of 15.4 mm (27.87 % reduction compared to the control). The hybrid mixture achieves the best performance with the lowest rut depth. The combination of ECR and polymer with RAP provides a balanced enhancement of stiffness and elasticity, ensuring superior resistance to rutting [244]. Rubberized mixtures (AC10-RdRA50 and AC10-RhRA50) outperform RAP-only mixtures in rutting resistance, showcasing the critical role of rubber in improving elasticity and reducing permanent deformation. The hybrid modification (AC10-RhRA50) yields the most significant improvement due to the synergistic effects of rubber and polymer. Incorporating 50 % RAP improves stiffness, which helps in reducing rut depth. However, RAP alone does not provide sufficient elasticity to effectively prevent deformation, which is evident from the smaller reduction in rut depth compared to rubberized mixtures [25]. The hybrid mix (AC10-RhRA50) provides the best resistance to rutting due to its balanced approach, combining stiffness and

flexibility to dissipate stress and resist permanent deformation.

As a conclusion, rubberized mixtures with RAP are the most effective in improving rutting resistance, with hybrid approaches (rubber + polymer) offering the greatest benefits. RAP-only mixtures provide moderate improvements in stiffness but lack the elasticity needed for optimal rutting resistance. Modifying conventional asphalt with both RAP and rubberized additives is crucial for achieving sustainable and durable pavements.

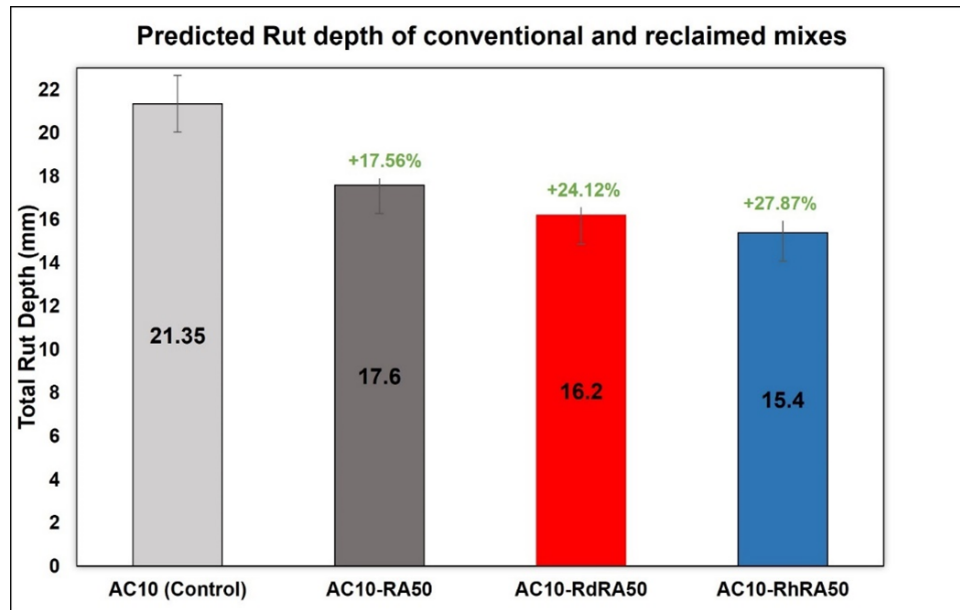


Figure 4.59: Rut depth analysis of conventional and RAP mixtures

4.6.10 Cracking analysis of conventional and RAP mixes

The cracking analysis shown in figure 4.60 compares the evolution of damage (%) in conventional and RAP mixtures over the pavement's service life (in years). The trends in figure 4.60 provide insights into the long-term performance of conventional and reclaimed mixes. Damage increases steeply over time, reaching 75% at the 10-year mark [245].

The AC10 (control) mixture exhibits the highest cracking rate due to its inability to dissipate stresses effectively. Lack of additives or modifications results in poor resistance to cracking under repeated loading [246]. This mix has the poorest long-term durability among all tested mixes.

In AC10-RA50 damage progresses faster than the control mix, reaching 85% by the 10-year mark. Incorporating RAP improves stiffness, increasing cracking rates in the earlier years. However, the stiffness can lead to brittleness over time, accelerating cracking in later stages. This mix shows no improvement compared to the AC10 (control) mixture [211].

AC10-RdRA50 (rubberized dry addition ECR with RAP) damage progression is slower than both the control and RAP-only mixes, with 65% damage at 10 years. Rubberized dry addition ECR improves flexibility and enhances resistance to crack initiation and propagation. This mix exhibits better cracking resistance due to its ability to dissipate energy and delay crack growth. Significant improvement over both control and RAP-only mixes, with prolonged service life [247]. In figure 4.60, AC10-RhRA50 (rubberized hybrid addition with RAP) The slowest damage progression among all mixes, reaching 60 % damage at 10 years. The hybrid combination of rubber and polymer provides a balance of stiffness and elasticity, effectively resisting both crack initiation and propagation. Superior stress dissipation and flexibility allow for the lowest cracking rates over time. Performance: Best long-term durability, with the lowest damage percentage at the end of the pavement’s service life [248].

As a conclusion, the conventional mixture AC10 (control) suffers less damage over time as compared to AC10-RA50 mix due its high stiffness. However, both mixes demonstrating its unsuitability for long-term use under heavy loads. Rubberized mixes with RAP (AC10-RdRA50 and AC10-RhRA50) show significant improvements in cracking resistance, with the hybrid mix performing the best. AC10-RhRA50 exhibits the slowest cracking progression due to its superior balance of stiffness and flexibility. This mix provides the most durable solution, minimizing damage over the pavement’s lifespan. Rubberized mixes with RAP addition significantly outperform RAP-only mixes by enhancing elasticity and dissipating stress more effectively, delaying the onset and progression of cracking.

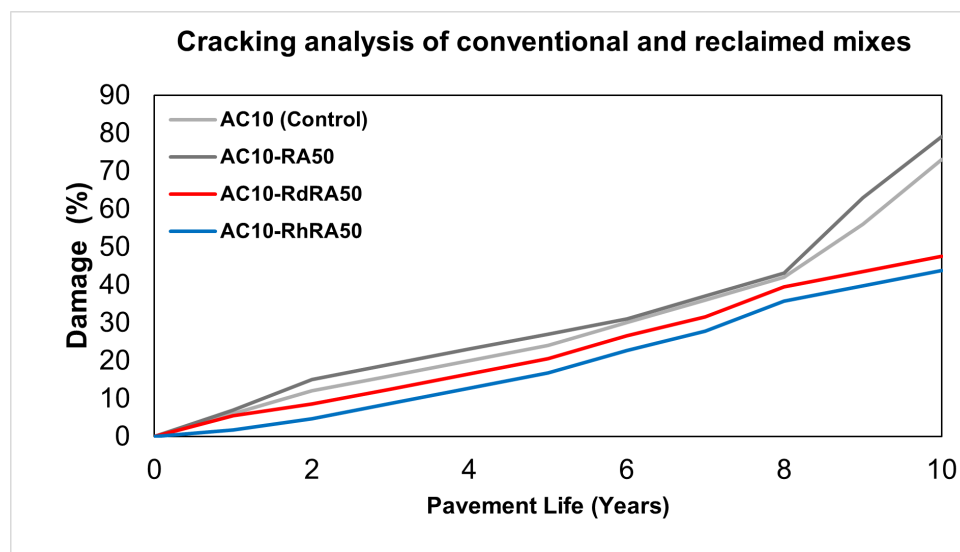


Figure 4.60: Cracking analysis of conventional and RAP mixtures

4.6.11 Conclusions and recommendations of section-V

As a response to RO-III in section 1.2 "To investigate whether a "advanced-level" SVECD damage-related characterization of paving materials can discriminate between performances conventional and APMs

- Stiffness using AMPT for dynamic modulus testing and 2s2p1D model.

- Rutting resistance using [AMPT](#) for [SSR](#) testing and [shift](#) model.
- Cracking resistance using [AMPT](#) for cyclic fatigue test and [SVECD](#) model.
- Thermal analysis using [CTC](#) model

This section provides a detailed analysis using advanced level damage related characterization based on [SVECD](#) approach for conventional and [RAP](#) mixtures considered as [APMs](#). The summary of the conventional and [RAP](#) mixtures is described in table 4.7 respectively. As a summary it can be observed that the Stiffness E^* for AC10-RA50 demonstrates the highest stiffness, especially at high temperature, due to the [RAP](#) content. For cracking resistance Both AC10-RdRA50 and AC10-RhRA50 perform significantly better than AC10 (control) and AC10-RA50, in [FlexPAVETM](#) v 1.1 simulations. The AC10-RhRA50 has the highest [Sapp](#) recommended for the heavy traffic. The rutting performance of the [RAP](#) mixes consistently outperforms AC10 (control) in resisting rutting and minimizing rut depths at both moderate (20°C) and high (55°C) temperatures. The [RSI](#) for AC10-RhRA50 exhibits the lowest [RSI](#), qualifying it for heavy traffic categories.

It can be summarized that the advanced damage related characterization provides a comprehensive difference between conventional and [RAP](#) mixtures, including AC10 RA50, AC10RdRA50 and AC10-RhRA50 are engineered to improve the performance of conventional mixtures e.g., AC10 (control) by addressing critical issues like cracking, rutting, and thermal sensitivity. As a concluding remarks for the reclaimed mixes, it can be noted that only [RAP](#) mixtures perform better in rutting resistance while the addition of rubber and polymers with [RAP](#) content also enhances its fatigue resistance.

Table 4.7: Advanced-level approach using (SVECD) damage-related characterization for conventional and reclaimed asphalt mixtures

Mixture ID	Stiffness (E*) (MPa) at 4.0 °C (10 Hz)	Diff(%)	Stiffness (E*) (MPa) at 40 °C (0.1 Hz)	Diff(%)	Sapp (%)	Range	Diff(%)	Cracking (%)	Diff (%)	RSI (%)	Diff (%)	Range	Rut depth (mm)	Diff (%)
AC10 (control)	26131	Control	611	Control	16	Sapp>36: E	Control	73	Control	6.53	Control	RSI<12:S	21.35	Control
AC10-RA50	28256	+8	667	10	14.4	Sapp> 30:V	+10	79	-8.2	3.8	41	RSI<4:H	17.6	17
AC10-RdRA50	21070	-20	581.2	13	21.1	Sapp>8:S	+32	47.5	+35	2.82	56	RSI<1:E	16.2	24
AC10-RhRA50	19013	-27	641	5	24.2		+62	43.7	+40	2.37	63		15.4	27

Chapter 5

Conclusions and future Recommendations

5.1 Conclusions

This research work aims to improve the gaps in QA/QC of pavement monitoring approaches adopted by the Italian road authorities. Addressing the main research question that how the pavement monitoring can be improved?. For this purpose, this research work proposed a basic level damage-related characterization approach based on IDT testing, and more advanced-level testing includes AMPT and damage-related characterization SVECD theory. The following conclusions have been drawn using basic and advanced level damage related characterization approaches and QA/QC testing by the Italian road authorities for pavement monitoring.

To assess whether the current Italian road authority characterization (including both QA/QC service life), can discriminate between the performance of conventional bituminous and APMs. This approach is based on the evaluation of the volumetric properties and IDT testing as follows:

- ITS at 25 °C for any bituminous materials.
- ITSR at 25 °C for crumb rubber modified bituminous materials.

The current Italian road authority specifications and conventional testing methods QA/QC for the improvement of pavement monitoring summarized in Table 5.1 showed minimal ability to differentiate between conventional bituminous mixtures, (rubberized and reclaimed asphalt mixtures) used as APMs.

Moreover, from Table 5.1 it can be summarized that no clear difference has been found in ITS values at (N210) gyrations and 5.0 % air voids.

As a conclusion, a sophisticated approach is required for the discrimination of conventional bituminous mixtures and APMs. Moreover, from the results shown in Table 5.1, the fixed 5.0% air voids can be utilized for the damage-related characterization.

Table 5.1: QA/QC testing for conventional bituminous materials and APMs with ANAS Italian road authority specifications

Mixture ID	No of gyrations	Void Content (%)	Void Content Range (%)	Diff (%)	ITS at N210 (MPa)	Range (MPa)	Diff (%)	ITS 5 % (AV)	N210 VS 5%	CTI (MPa)	Range (MPa)	Diff (%)
AC10 (Control)	N10	14.9	11-15	Control			Control	1.38	2.8	109		Control
	N120	6	3-6		1.42							
	N210	5.5	≥2									
AC8-Rd	N10	11.9	11-15	20			6.34	1.46	3.3	111		1.83
	N120	4	3-6	16								
	N210	2.6	≥2	18	1.51							
AC10 -Rh	N10	13.7	11-15	8		ANAS 0.72-1.60	11.27	1.53	3.16	118	ANAS ≥65	8.26
	N120	4.5	3-6	25								
	N210	3.6	≥2	34	1.58							
AC10-RA50	N10	13.1	11-15	12		ANAS CSA ≥0.6	9.86	1.51	3.2	110	ANAS CSA 50-150	0.92
	N120	4.2	3-6	30								
	N210	2.4	≥2	56	1.56							
AC10-RdRA50	N10	14	11-15	6			16.9	1.59	4.21	120		10
	N120	5	3-6	9								
	N210	2.9	≥2	11	1.66							
AC10-RhA50	N10	11.9	11-15	20			16.9	1.57	5.4	120		10
	N120	4.4	3-6	27								
	N210	2.5	≥2	4	1.66							

To investigate whether a 'basic' damage-related characterization of pavement materials can discriminate between performances of [APMs](#). This approach is still based on volumetric characterization and [IDT](#) testing for any bituminous materials including [APMs](#),

- Rutting resistance using [HT-IDT](#) at 55 °C testing for the Sicily region.
- Cracking resistance using [Ideal-CT](#) testing at 25 °C.
- Moisture sensitivity using [ITSR](#) testing at 25 °C.

Comparison between conventional bituminous materials and (rubberized, reclaimed asphalt mixtures) utilized as [APMs](#) using the basic damage-related characterization approach, for cracking resistance using [Ideal-CT](#) testing, moisture sensitivity using [ITSR](#), and rutting resistance using [HT-IDT](#) testing under high traffic and environmental stresses are summarized in table 5.2.

- Conventional bituminous materials exhibit higher cracking rates over time due to their stiffness and inability to dissipate stress effectively. Hybrid mixes like (AC10-Rh) offer the best balance of stiffness and flexibility, slowing cracking progression and enhancing long-term durability.
- Conventional mixtures AC10 (control) fail to meet the ANAS standard for moisture resistance. Rubberized mixtures, particularly hybrid mixes, achieve superior performance in wet conditions.
- Rubberized and hybrid mixes significantly outperform conventional mixtures in rutting resistance. The hybrid composition (rubber + polymer) offers the improved performance against permanent deformation.

Comparison between conventional and [RAP](#) mixtures using the basic damage-related characterization approach, for cracking resistance using [Ideal-CT](#) testing, moisture sensitivity using [ITSR](#), and rutting resistance using [HT-IDT](#) testing under high traffic and environmental stresses are summarized in table 5.2.

- Conventional asphalt mixtures AC10 (control) exhibit more resistance to fatigue cracking due to their low stiffness as compared to the [RAP](#) only (AC10-RA50) mix, which leads to stress concentration and cracking under repeated loads.
- Occurrence of cracking in [RAP](#) only mixtures (AC10-RA50) are higher compared to rubberized, or hybrid mixtures combine with [RAP](#), especially if utilized in colder climates or under high traffic loads. Reclaimed asphalt mixtures, especially those with 50% [RAP](#) content AC10-RA50, exhibit increased stiffness, leading to reduced flexibility and higher susceptibility to cracking. They benefit from improved fatigue life when combined with rubber or polymer additives, as in AC10-RdRA50 and AC10-RhRA50.
- Conventional mixtures have poor performance in moisture sensitivity tests but often fail to meet advanced moisture resistance standards such as [ITSR](#) benchmarks. Their performance deteriorates more significantly when subjected to water saturation and freeze-thaw cycles compared to alternative mixtures.
- [RAP](#) mixtures with additives exhibit more moisture resistance than conventional mixtures due to the aging and brittleness of reclaimed materials. Moisture sensitivity improves significantly when [RAP](#) is combined with rubberized additives, achieving better performance than conventional mixtures.
- Reclaimed mixtures without additives perform poorly in high-temperature conditions, with significant rutting

observed during [HT-IDT](#) and stress sweep rutting tests. When rubberized or hybrid technologies are applied, reclaimed mixtures surpass conventional mixtures in rutting resistance, making them suitable for heavy traffic regions. As a conclusion, the basic damage-related characterization, including cracking, rutting, and moisture tests, effectively discriminates between conventional bituminous materials and [APMs](#).

The hybrid mixtures (AC10-Rh), demonstrate superior performance in terms of cracking, moisture sensitivity and rutting compared to conventional mixtures. It can be concluded that the hybrid mixtures can be utilized for the future construction of road pavements. And also the basic-level damage related characterization approach can be utilized to discriminate between the conventional and [APMs](#).

Table 5.2: Basic level approach using damage-related characterization for conventional and APMs

Mixture ID	CT-Index at 5% Voids	Range (MPa)	Diff (%)	ITSR (%)	Range (MPa)	Diff (%)	HT-IDT (MPa)	Range (MPa)	Diff (%)	CTI (MPa)	Diff (%)
AC10 (control)	126		Control	79.1		Control	0.25		Control	114	Control
AC8-Rd	139		10	82.9		4.8	0.30		20	140	22
AC10-Rh	181	CTindex ≥ 80	43	86.9	ITSR ≥ 80	10.9	0.34	>0.44: Excellent 0.32 $\leq$ 0.44: Good $\leq$ 0.20: Poor	36	147	28
AC10-RA50	112		-11	78		-1.39	0.28		12	132	15
AC10-RdRA50	121		-4	81.8		3.4	0.33		32	150	31
AC10-RhRA50	117		-7	82.5		4.3	0.37		48	170	49

To investigate whether an "advanced" SVECD damage-related characterization of paving materials can discriminate between performances of APMs. This approach is based on volumetric characterization and testing with an AMPT coupled with damage modeling based on SVECD for any bituminous materials incorporating APMs as follows:

- Stiffness using AMPT for dynamic modulus testing and 2s2p1D model
- Rutting resistance using AMPT for SSR testing and shift model.
- Crack resistance using the AMPT and SVECD model.
- Thermal analysis using CTC model

• Advanced Laboratory testing using SVECD theory, and other performance models (shift and CTC) provides a more comprehensive differentiation shown in table 5.3 and predictive capability for long-term performance of conventional and rubberized asphalt mixtures.

Comparison between conventional and rubberized mixtures using the advanced level damage-related characterization approach has been carried out in terms of stiffness, cracking, rutting and thermal resistance using SVECD theory. It can be concluded that rubberized mixtures, especially the hybrid mixtures outperforms the conventional mixtures in terms of cracking and rutting resistance. However, the rubberized mixtures shows poor performance in terms of the stiffness.

The advanced-level damage related characterization approach by SVECD theory can be utilized to discriminate between the conventional and APMs.

• The database of conventional and rubberized asphalt mixtures provided in Table 5.2 can be utilized for the design of wearing courses according to Italian specifications and can be used as a reference for QA/QC, pavement monitoring and future development of PMS.

RAP mixtures showed poor performance in terms of cracking. In terms of moisture sensitivity the RAP mixtures showed minimum improvement as compared to conventional mixtures. The best resistance to the rutting as compared to the to conventional mixtures has been observed by the utilization of RAP mixtures as shown in Table 5.2. It can be concluded that the RAP mixtures can be utilized for the highly permanently deformed road infrastructures. The basic-level damage related characterization approach can be utilized to discriminate between the conventional and RAP mixes used as APMs.

- Advanced Laboratory testing using SVECD theory, and other performance models (shift and CTC) provides a more comprehensive differentiation shown in table 5.3 and predictive capability for long-term performance of conventional and RAP mixtures.
- Comparison between conventional and RAP mixtures using the advanced level damage-related characterization approach has been carried out in terms of stiffness, cracking, rutting and thermal resistance using SVECD theory.
- It can be concluded that RAP mixtures, especially with (polymers and ECR) outperforms the conventional mixtures in terms of cracking and rutting resistance. However, the rubberized mixtures shows poor performance in terms of the stiffness.

- The advanced-level damage related characterization approach by [SVECD](#) theory can be utilized to discriminate between the conventional and [APMs](#).
- The database of conventional and [RAP](#) mixtures provided in Table [5.2](#) and [5.3](#) as a response to RO-IV in section [1.2](#), of this research work can be utilized for wearing courses, designed according to Italian specifications and can be used as a reference for [QA/QC](#), pavement monitoring and future development of [PMS](#).

Table 5.3: Advanced-level approach using [SVECD](#) damage-related characterization for conventional and [APMs](#)

Mixture ID	Stiffness (E*) (MPa) at 4.0 °C (10 Hz)	Diff(%)	Stiffness (E*) (MPa) at 40 °C (0.1 Hz)	Diff(%)	Sapp (%)	Range (%)	Diff(%)	Cracking (%)	Diff(%)	RSI (%)	Diff (%)	Range(%)	Rut depth (mm)	Diff(%)
AC10 (control)	26131	Control	611	Control	16		Control	73	Control	6.53	Control	RSI<12:S	21.35	Control
AC8-Rd	22205	-15	641	5	19.5	Sapp>36: E Sapp> 30:V Sapp>8:S	22	57	22	3.99	40	RSI<4:H	18.2	14.75
AC10-Rh	23304	-10	727	19	25		56	44	40	3.54	45	RSI<1:E	17	20.37
AC10-RA50	28256	+8	667	10	14.4		+10	79	-8.2	3.8	41		17.6	17
AC10-RdRA50	21070	-20	581.2	13	21.1		+32	47.5	+35	2.82	56		16.2	24
AC10-RhRA50	19013	-27	641	5	24.2		+62	43.7	+40	2.37	63		15.4	27

5.2 Recommendations

As a final recommendation to the practitioners,

- It is recommended that Italian road authorities should integrate at least the basic level damage-related characterization methods, based on IDT testing, into their specifications to accurately predict performance and optimize material selection.
- The advanced level damage related characterization based on SVECD theory is recommended for those regions that can afford the high-cost equipment like AMPT.
- The use of RAP mixtures in combination with rubber and polymer additives to achieve sustainable and cost-effective pavement solutions. Incorporating RAP can reduce material costs to achieve comparable or superior performance. Expanding the use of reclaimed asphalt materials in combination with rubber and polymer additives to achieve sustainable and cost-effective pavement solutions.
- Further studies should explore the long-term performance of hybrid mixes under varying climatic and traffic conditions to validate their widespread application.
- Develop a performance-based pavement management system (PMS) to integrate laboratory findings into practical applications for road construction and maintenance.

5.3 Future Lines

One of the greatest challenges for pavement engineers and decision makers is to effectively maintain and rehabilitate pavement networks with limited resources and its growing demand. The effective maintenance and rehabilitation's decision taken at a right time and right place, helps to improve the road performance, extend the service life, and provide the economic benefits to pavement networks. In order to feed the pavement management system (PMS) generally, two approaches of the data collection from the pavement structures are mostly being practiced i-e, reactive pavement monitoring and proactive pavement monitoring approach. The reactive monitoring approach is about to get the response from the pavement structures for example the use of field coring or nondestructive testing (NDT) techniques. While the proactive monitoring approach is about the automated response recorded from the pavement structures for example the use of embedded sensors. To improve the pavement monitoring in future work. This research work has also proposed a concept of 'Live labs' based on four scenarios to improve the pavement monitoring using reactive and proactive monitoring techniques for more sustainable management of road pavements.

5.3.1 Road pavement management without monitoring and planned maintenance-current practices

The first scenario is the management of road pavements without monitoring and without any planned maintenance, which is believed to be practiced by most of the road pavement authorities. Since 1960, the traditional approaches of pavement maintenance and rehabilitation decisions based on the experts opinions was one of the common case and being practiced nowadays even in developed countries. These approaches involves the partial or full replacement of the pavement structures upon the appearance of the functional or structural damage without any monitoring and planned maintenance based on allocated budget. The flow process of the current practices without monitoring and planned maintenance can be seen in figure 5.1. As a future line this scenario starting from the pavement design and follows the process of road maintenance without any planning. How it can be improved?

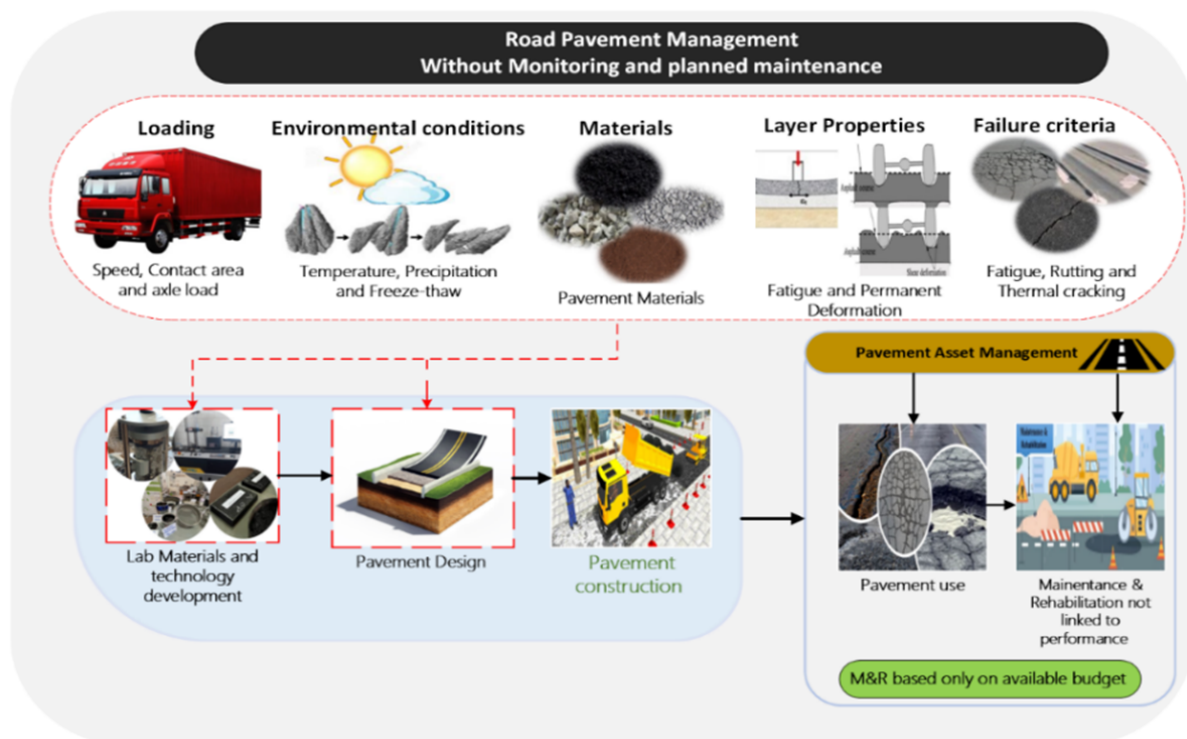


Figure 5.1: Current practices of road pavement management without monitoring and no planned maintenance

5.3.2 Road pavement management with reactive monitoring and planned maintenance

The reactive monitoring based on destructive or non destructive evaluation techniques has been developed for the purpose of data collection from the pavement structures shown in figure 5.2. For example the falling weight deflectometer **FWD** is a non destructive testing device that is capable for the evaluation of structural capacity of the pavement structures. For the generation of the transient impulse dynamic load, the heavy mass is dropped from a certain height. The structural data for the deflection of the pavement can be obtained by using the string of geophones mounted in **FWD**. One of the common destructive method for reactive monitoring of the pavement

structures is coring of the samples. The samples prepared and tested in the laboratory have poor correlations with the field performance. For field based investigation of physical, volumetric and performance based analysis the samples are cored from the field. Ekarizan et al, developed a prediction model for the pavement performance by means of the laboratory testing based on collected core samples from the field. Although the reactive monitoring has many advantages but the coring restricts the smooth flow of the traffic, causes accidents and provide damages to the pavement structures. The researchers in past years have moved from destructive method towards the non destructive method, but most of the equipments and the methodologies are still reactive instead of the proactive. High labour cost, intensive techniques for monitoring and the continuous capturing of the data from the pavement structures is still a research question

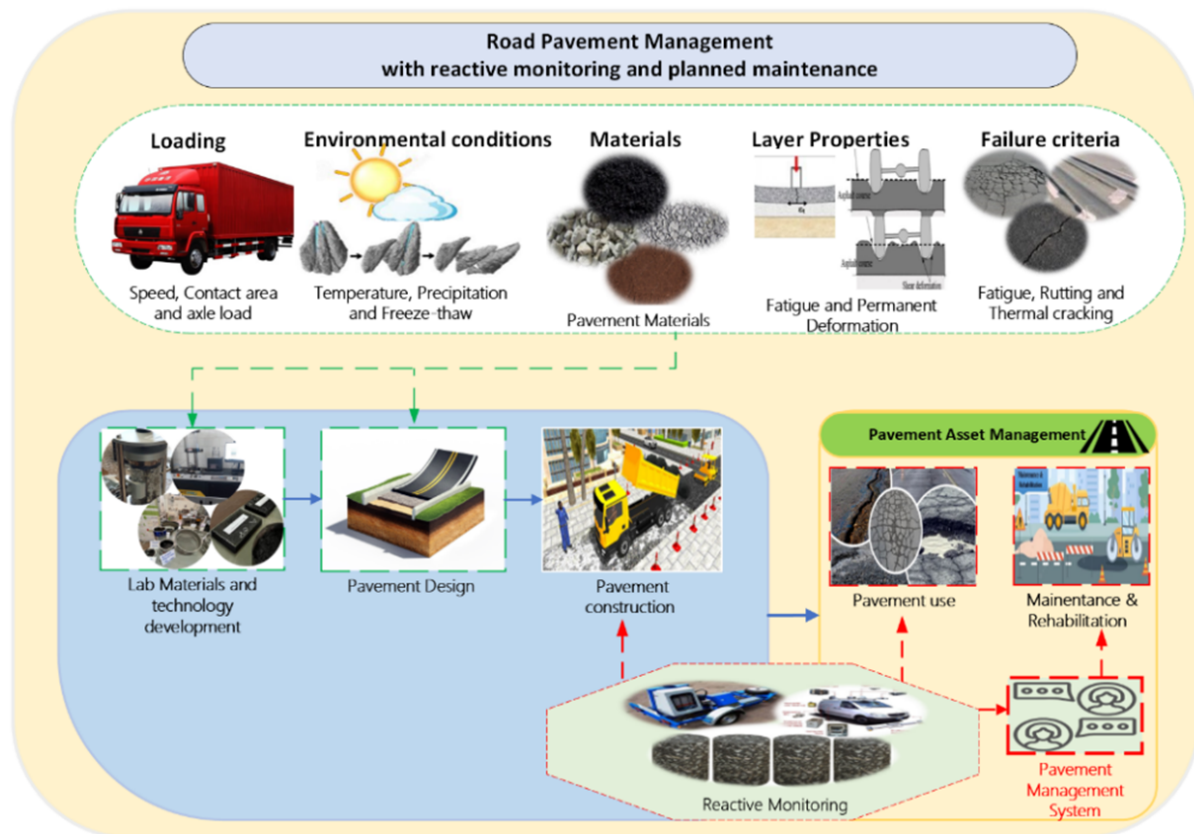


Figure 5.2: Road pavement management with reactive monitoring and planned maintenance

5.3.3 Monitoring approaches and planned maintenance based on proactive APT

Before the advent of motorcar in early 1900s, the approximate increase in traffic loading, speed (100 Km/hr.), axle configuration (9 tons dual truck tire and tire pressure (upto 700 KPa) had also impacted the maintenance and rehabilitation cost of the pavement structures. This leads the researchers and pavement engineers for the proactive appraisal of the structural behavior of the pavements with an increase in quality of construction materials, pavement design, performance, and pavement response under the traffic loading and environmental conditions in a compressed period. To address these issues an accelerated pavement testing system (APT) was developed with the capability of testing prototypes or actual layered pavement structures for the monitoring of accumulated damage

caused by controlled loading and environmental conditions. The APT system is divided into two categories i-e full scale system (standard truck tire or a combination of truck tires) are applied on pavement structures for the loading purpose, while small scale system (reduced tire size and its load) is applied to collect the damage data from the pavement structures. The examples of full-scale system are test tracks (National Center of Asphalt Technology (NCAT) Test Track, Minnesota Road Research Project (Mn-ROAD), circular or linear tracking device. While mobile load simulator (MMLS3) can be considered as an example of small-scale testing. The monitored data generated from the APT structures, has the advantage of close resemblance of real pavement structures. During the last decade the full scale APT focused on pavements material characterization, modeling, performance, behavior, design and the development of the databases for pavement performance to have cost saving approach for the improved monitoring and better analysis of the pavement structures. Hence, the provision of APT facilities ameliorates the pavement design and its performance in many countries, using their own local materials, loading patterns, construction practices and record of pavement response under different environmental conditions. One of the main challenges faced by the APT is difference of the loading conditions and climatic conditions observed in the field, the maintenance and preservation activities of the pavement structure by the APT are not suited because of the low speed of the test. This could be one of the future considerations. The flow process of reactive monitoring with planned maintenance and proactive APT approach has been shown in figure 5.3.

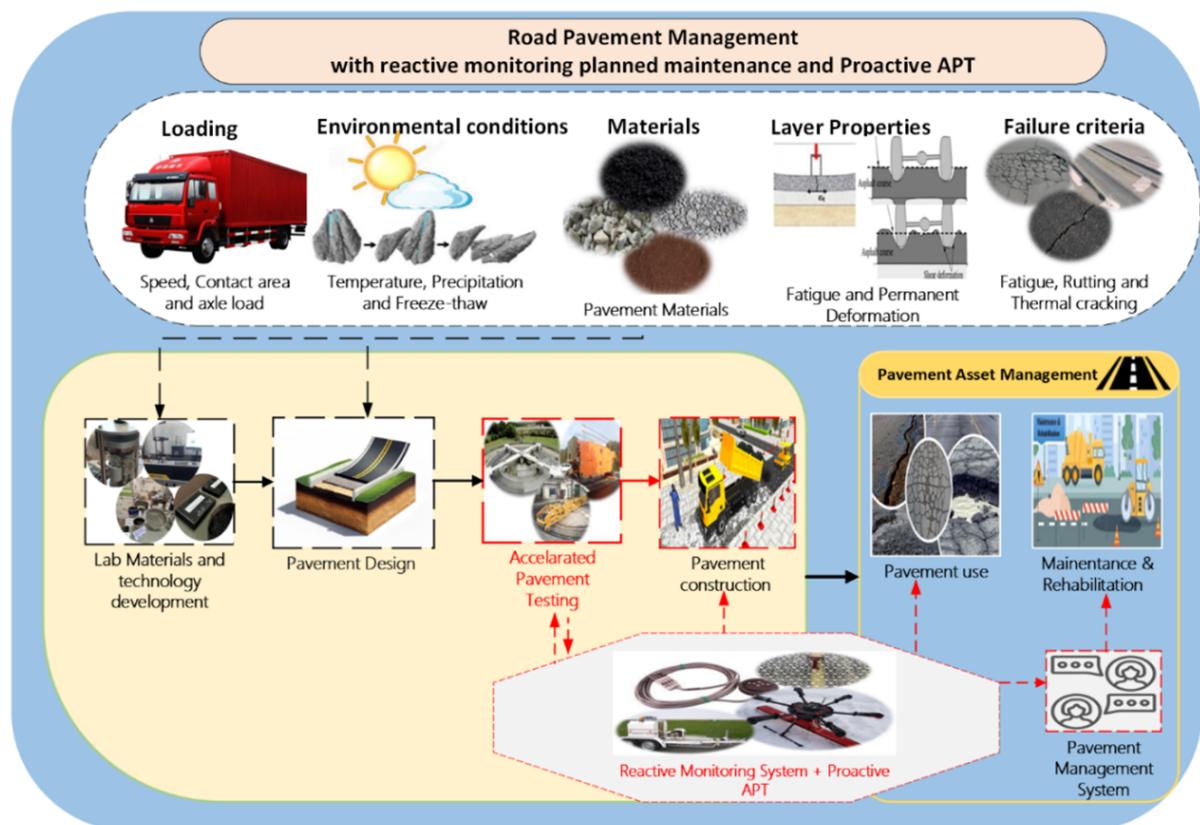


Figure 5.3: Road pavement management with reactive-proactive monitoring and planned maintenance with proactive APT

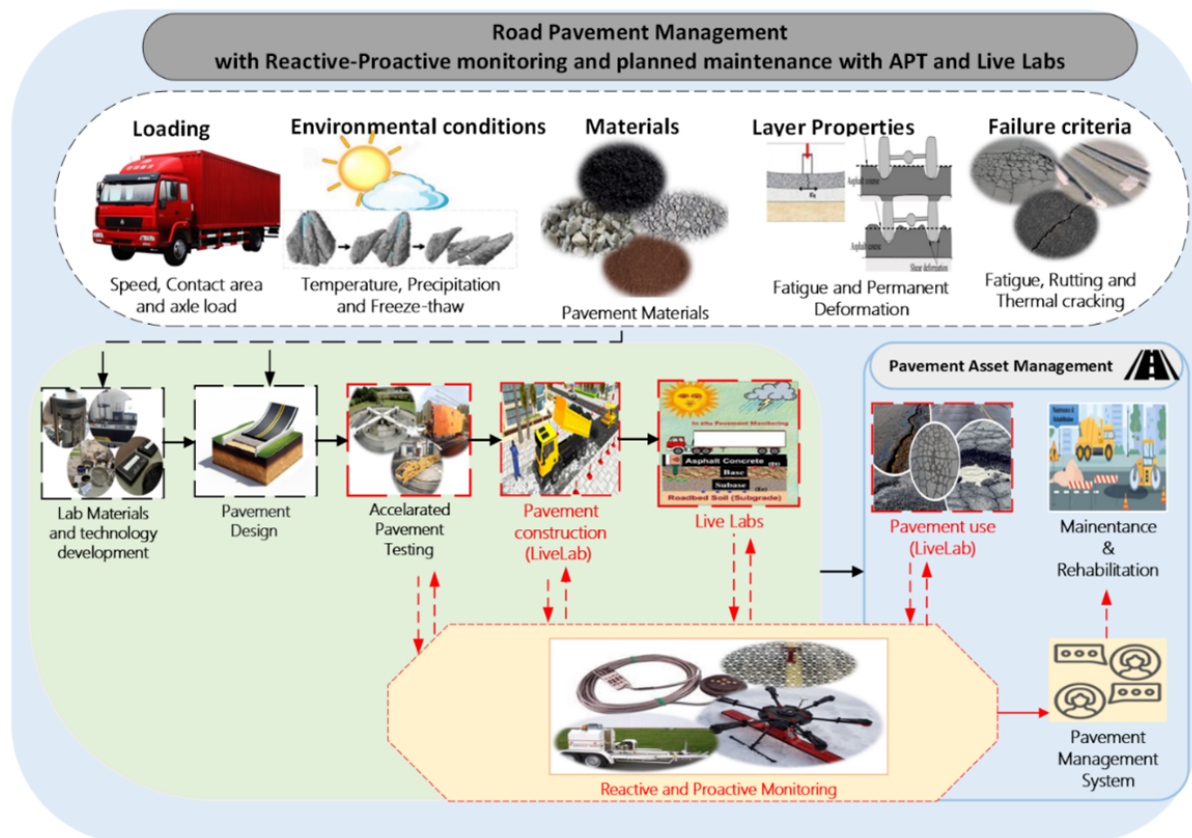


Figure 5.4: Road pavement management and maintenance using APT and Live LABS

5.3.4 Reactive-Proactive monitoring approach with APT and Live LABS

Accelerated pavement testing (APT) forms a strong linkage between the laboratory evaluation of pavement materials and the field behavior of materials that used as pavement layers. The performance prediction models generated by the laboratory testing can be calibrated using APT facilities. The planned maintenance using APT and LIVE LABS can be analyzed in figure 5.4. Both the reactive and proactive approaches can be of the scenario for future monitoring of the pavement structures. The evolutionary approach followed by the APT in the form proactive monitoring for the behavior and performance of the pavements, expands the knowledge and eliminates the research gaps identified by the previous research. In this regard, the identified future trending research based on four points includes, in situ field performance comparison, pavement design improvement, unprecedented climate change, development of reliability models and calibration of pavement instrumentation on full scale for feeding of PMS. Regarding the in-situ sensors, although the in-situ pavement monitoring by means of sensing technologies are capable of continuously monitoring the data, but due to its initial stages of research development in terms of damage detection, high cost, survival inside the pavement structure, replacement inside the pavement structure, data aquisition and its collection are the issues that need to be addressed before the field implementation of sensing technologies.

Chapter 6

Appendixes

Note: This training part has been conducted as period abroad as a part of PhD research in North Carolina state University United States

A1: Mix preparation, cutting and coring of specimen

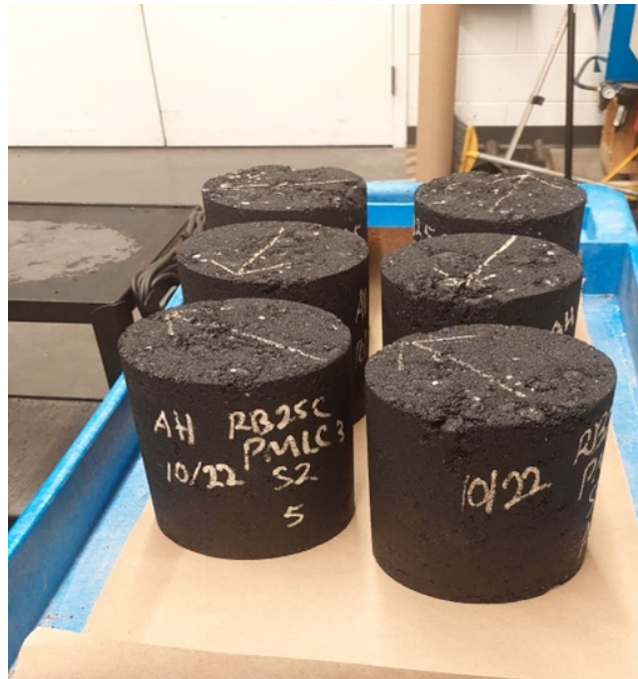
The collection of component materials was carried out from Barnhill Contracting Inc., Rockingham, North Carolina, United States. The materials were collected for types of mixes for Intermediate RI19.0C and base RB25.0C layer. The total amount of mixtures for both the intermediate and base layer for each test is summarized in Table. The quantity estimation of these mixture was done by total gyratory X number of buckets. The percentage of stockpiles for RB25.0C were #467M, #78M, Sand, RAP and RAS (22%, 29%, 21%, 25% and 3%) While for RI19.0C were 67M, #78M, Sand, RAP and RAS (25%, 22%, 25%, 25%, 3%) including percentage of RAP of 4.8% and RAS of 22.5% respectively. Please note that the observation was done only for RB 25.0C base mixtures.

Summary of Mixtures, Test Types, and Specimen Weights for BMD+ Evaluation

BMD+	Mixture	Material Source	No. of Mixtures	No. of Tests			Gyratory		Weight (lb.) Amount
				$ E^* $ (3 tests)	Cyclic (3 tests)	SSR (4 tests)	Gyratory	Total Gyratory	
	RI19.0C (New)	PMLC Samples	4	1	1	4	6	24	432
		LMLC Specimens	5	1	1	4	7	35	630
	RB25.0C (New)	PMLC Samples	4	3	3	4	10	40	720
		LMLC Specimens	5	3	3	4	11	55	990

Compaction of RB25.0C Samples

The Superpave gyratory compaction was done for RB25.0C samples were done using the specimen height of 180 mm and the mixing temperature. The compacted samples after cooling are shown in Figure. For the account of sensitivity and impact on AMPT testing the upper and lower side of gyratory compaction was noted accordingly shown in Figure.



Compacted specimen (180mm height)

Coring and Cutting of Samples

To proceed with Dynamic modulus and cyclic fatigue tests the gyratory samples were cored and cut for the size of 38 X 110 mm in size. The coring and cutting should be done in way that both the surfaces should be same to avoid the sensitivity in dynamic modulus and cyclic fatigue testing. The coring and cutting process can be shown in Figure



Coring and cutting of specimen

Air voids analysis

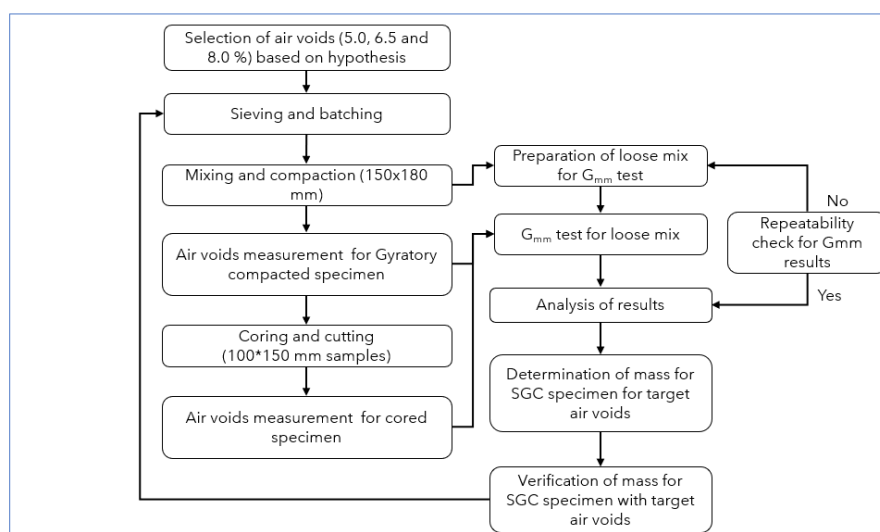
The bulk specific gravity test (G_{mb} and theoretical maximum specific gravity (G_{mm}) tests were conducted for compacted as well as cored and cut samples for air voids analysis according to the standard specifications. As suggested by the previous researches it is necessary to carry out the air voids study for the following reasons:

Air Void Considerations in Mix Design vs. Performance Testing

Item	Cause / Difference	Implication / Reference
Geometry difference	Mix design uses 150×(110–120) mm specimens; performance testing uses 150×180 mm specimens	Chehab et al. 2000
Compaction method	• Mix design: N_{design} to reach target height • Performance testing: compaction based on final specimen height	C. Castorena et al. 2017
Mass of material	Quantity of compacted mass unknown for both methods	K. Kuchiishi et al. 2023
Air void measurement	Measured air voids do not match target air voids in both methods	C. Castorena et al. 2017
Coring effects	• Coring not performed for mix design • Coring affects air voids in performance specimens — lower voids in core center vs. sides	Chehab et al. 2000, K. Kuchiishi et al. 2023
Study purpose	• Mix design: determine optimum binder content • Performance testing: assess dynamic modulus, cyclic fatigue, and SSR behavior	K. Kuchiishi et al. 2023

How to perform the air voids study

The following flow process illustrates the airvoids study



How to perform the air voids study

Gluing of samples

Please note that the gauge points to the test specimen should not be attached to the larger aggregates on specimen. It may affect the final outcomes of the results.

A1.2: Dynamic modulus testing

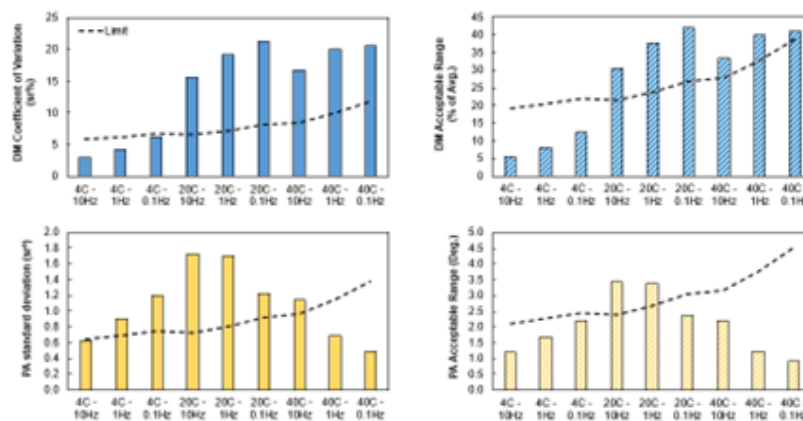
The dynamic modulus test was conducted using AASHTO TP 132-19 (2021). The conditioning time for the temperature of (4, 20 and 40°C at each temperature is 1hr 30 minutes. The samples should be kept 1 hr in Oven and 30 minutes in AMPT. The selected frequencies were (0.1, 1.0 and 10 Hz) for RB25.0C mixtures. Please make sure the gauge length should be selected 70 mm in AMPT machine for each specimen

Data quality indicators

Data Quality Indicators (DQIs) must be checked during the test. The data quality indicators can be seen in Table 3.

Table 3—Data Quality Statistics Requirements

Data Quality Statistic	Limit
Deformation drift	In direction of applied load up to 100% in the direction opposite of the applied load
Peak-to-peak strain	50 to 75 microstrain
Load standard error	10%
Deformation standard error	10%
Deformation uniformity	30%
Phase uniformity	3°



Actions to Improve DQIs

If the data quality indicators are meeting the requirements, then the following actions as mentioned in the standards AASHTO TP 132-19 (2021) should be taken as listed in Table 4.

Repeatability and Reproducibility Limits

The repeatability and reproducibility limits should be checked after the analysis in FlexMAT cracking. The following data should be pasted in Dynamic modulus precision analysis excel sheet developed the NCSU civil engineering department. 1. Frequency 2. Dynamic modulus 3. Phase angle and 4. average temperature. For repeatability, all the coefficients of variation should be below the limits. Figure shows an example of repeatability errors. In this case the specimen needs to be revised based on data quality indicators.

Literature to be considered for 2s2p1D model and DQIs

For the better data quality assessment, the following literature on 2S2P1D model and dynamic modulus testing should be considered.

1. Cao, W., Lacroix, A., & Kim, Y. R. (2022). A triaxial linear viscoelastic characterization framework for asphalt concrete based on the 2S2P1D model. *European Journal of Mechanics-A/Solids*, 92, 104469.

Troubleshooting Table for Data Quality Issues

Item	Cause	Possible Solutions
Deformation drift not in direction of applied load	Gauge points are moving apart	Reduce LVDT spring force. Add compensation springs. Reduce test temperature.
Peak-to-peak strain too high	Load level too high	Reduce load level.
Peak-to-peak strain too low	Load level too low	Increase load level.
Load standard error > 10%	Applied load not sinusoidal	Adjust tuning of hydraulics.
Deformation standard error > 10%	1. Deformation not sinusoidal 2. Loose gauge point 3. Excessive noise on deformation signals 4. Damaged LVDT	1. Adjust tuning of hydraulics. 2. Check gauge points. Reinstall if loose. 3. Check wiring of deformation sensors. 4. Replace LVDT.
Deformation uniformity > 30%	1. Eccentric loading 2. Loose gauge point 3. Sample ends not parallel 4. Poor gauge point placement 5. Non-uniform air void distribution	1. Ensure specimen is properly aligned. 2. Check gauge points. Reinstall if loose. 3. Check parallelism of sample ends. Mill ends if out of tolerance. 4. Check for specimen non-uniformity (segregation, air voids). Move gauge points. 5. Ensure test specimens are cored from the middle of the gyratory specimen.
Phase uniformity > 3°	1. Eccentric loading 2. Loose gauge point 3. Poor gauge point placement 4. Damaged LVDT	1. Ensure specimen is properly aligned. 2. Check gauge points. Reinstall if loose. 3. Check for specimen non-uniformity (segregation, air voids). Move gauge points. 4. Replace LVDT.

2. Kuchiishi, A. K., Castorena, C., Kim, Y. R., & Underwood, B. S. (2023). Ruggedness evaluation of dynamic modulus test using small-scale test specimens for asphalt mixture performance tester. Transportation Research Record, 03611981231165780.
3. Kim, Y. R., Guddati, M. N., Choi, Y. T., Kim, D., Norouzi, A., Wang, Y. D., ... & Underwood, B. S. (2022). Hot-Mix Asphalt Performance Related Specification Based on Viscoelastoplastic Continuum Damage (VEPCD) Models (No. FHWA-HRT-21-093). United States. Federal Highway Administration. Office of Research and Technology Services.

A1.3: Cyclic fatigue testing

For determining the damage characteristic curve of small specimen and Failure Criterion in AMPT the cyclic fatigue test is performed according to the specification of AASHTO T 411-23 (Revised in 2023).

Gluings Details

For end plates, top and bottom (3.4 gm black solution) and (2.0 gm white solution) Ratio is 1.7. For gauge points is 15 minutes time for plates, 30 minutes.

Conditioning Time

The conditioning time of the specimen to reach the target temperature can be obtained by the dummy specimen. In our case is 40 minutes.

Temperature selection

The temperature should be selected based on different PG binder grades as mentioned in the standards with or without MSCR designations as mentioned in Table 5 and Table 6, respectively.

Recommended Test Temperatures for Different Standard PG Binder Grades

PG Low Temperature, °C	46	52	58	64	67	70	76	82
-10	15	18	21	21	21	21	21	21
-16	12	15	18	18	21	21	21	21
-22	9	12	15	15	18	21	21	21
-28	9	9	12	12	15	18	21	21
-34	9	9	9	9	12	15	18	21
-40	9	9	9	9	9	12	15	18
-46	9	9	9	9	9	9	12	15

Recommended Test Temperatures for Different PG Binder Grades with MSCR “H”, “V”, or “E” Designations

PG Low Temperature, °C	46	52	58	64	67	70	76	82
-10	18	21	21	21	21	21	21	21
-16	15	18	21	21	21	21	21	21
-22	12	15	18	18	21	21	21	21
-28	9	12	15	15	18	21	21	21
-34	9	9	12	12	15	18	21	21
-40	9	9	9	9	12	15	18	21
-46	9	9	9	9	9	12	15	18

Load Check

Go down with hydraulic pressure slowly and one click at a time. Note: IF full hydraulic pressure is applied, it will break the glue and the crack will not be analyzed

Data quality statistics requirements for the fingerprint test

The following parameters should be analyzed as listed in Table 7 before the start of the cyclic fatigue test.

DQIs for Fingerprint

Indicator	Limit
Standard error of the applied stress	$\leq 10\%$
Average standard error of the measured strains	$\leq 10\%$
Uniformity coefficient of the measured strains	$\leq 35\%$
Uniformity coefficient for the phase angle measurements	≤ 3 degrees

Calculation of repeatability parameter using Excel code for test Cyclic Fatigue Test

After analysis in FlexMAT cracking, it is needed to check the DR Failure Criterion for your specimen. Use the excel code to calculate the repeatability parameter for your specimen. If it does not meet, please repeat the specimen. The standard specifications of v_{norm} should match the specified in Table 8 respectively. For the new mixtures these limits needs to be added based on the laboratory investigation.

Single Operator Precision for Damage Characteristics Curve and D^R Results

Mixture	Repeatability (maximum allowable range)	Reproducibility (d_{2SpD})
9.5S	0.088	0.090
9.5L-E	0.065	0.098
12.5S	0.129	0.090
12.5L	0.089	0.098
19S	0.214	0.090
25L	0.124	0.098

Possible Literature to be considered for Cyclic Fatigue and SVECD model

For the level of research already carried out on SVECD model and cyclic fatigue testing the following literature is undertaken for better understanding.

1. Wang, Y. D., Ghanbari, A., Underwood, B. S., & Kim, Y. R. (2021). Development of preliminary transfer functions for performance predictions in FlexPAVE™. *Construction and Building Materials*, 266, 121182.
2. Ding, J., Jeong, J., Lee, K. C., Underwood, B. S., Kim, Y. R., & Castorena, C. (2023). Repeatability and Reproducibility Analysis for Functional Test Results from Asphalt Mixture Performance Tester Cyclic Fatigue Test. *Transportation Research Record*, 03611981231159409.
3. Kim, Y. R., Guddati, M. N., Choi, Y. T., Kim, D., Norouzi, A., Wang, Y. D., ... & Underwood, B. S. (2022). Hot-Mix Asphalt Performance Related Specification Based on Viscoelastoplastic Continuum Damage (VEPCD) Models (No. FHWA-HRT-21-093). United States. Federal Highway Administration. Office of Research and Technology Services

A1.4: Stress sweep rutting test

The High Temperature (HT) and Low Temperature (SSR) Testing was carried out under the specifications of (AASHTO TP 134). It is important to note that calculate the degree of days (DD) for low temperature and high Temperature. Check the height of the specimen before SSR Test. Please note that more than 2.6 gm of high vacuum grease liquid may affect the confining pressure. Please also check the bubbles of the chamber that should be 2 to bubbles in 10 seconds. Check the calibration of machine before testing at low and high temperature. The preparation of SSR specimen is shown in Figure

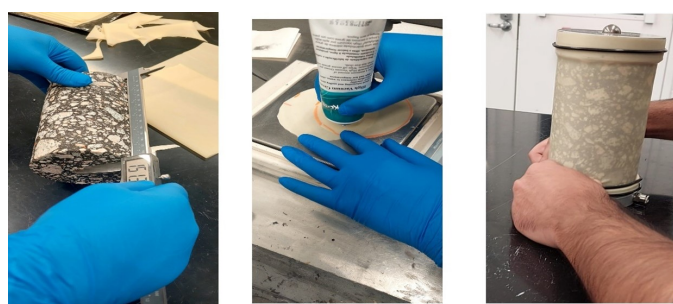


Figure 6.1: Stress Sweep rutting test

Possible literature for SSR testing

There is a limited amount of literature found on SSR testing. The following literature is under consideration for the better understanding of rutting strain index (RSI) and Stress Sweep Rutting test.

1. Kim, Y. R., Guddati, M. N., Choi, Y. T., Kim, D., Norouzi, A., Wang, Y. D., ... & Underwood, B. S. (2022).

Hot-Mix Asphalt Performance Related Specification Based on Viscoelastoplastic Continuum Damage (VEPCD) Models (No. FHWA-HRT-21-093). United States. Federal Highway Administration. Office of Research and Technology Services.

2. Jeong, J., Underwood, B. S., & Kim, Y. R. (2021). Rutting performance prediction using index-volumetrics relationships with stress sweep rutting test and Hamburg wheel-track test. *Construction and Building Materials*, 295, 123664.

3. Amir Ghanbari, Benjamin Shane Underwood & Youngsoo Richard Kim (2022) Development of a rutting index parameter based on the stress sweep rutting test and permanent deformation shift model, *International Journal of Pavement Engineering*, 23:2, 387-399, DOI: 10.1080/10298436.2020.1748190.

A2: FlexMAT Analysis

After the successful testing on AMPT the following templates were used for the 2s2P1D, SVECD and SSR model. AS mentioned above the DQIs and the Vnorm were checked for 2s2P1D and SVECD model. All the analyzed data should be matched with the standards.

A2.1: FlexMAT cracking

In order to proceed with SVECD damage modeling in FlexMAT cracking the following parameters needs to be checked

FlexMAT Cracking

Operator	Specimen No & ID	Fingerprint [E*] (MPa)	DMR, 0.85<DMR<1.15	Nf	log (Nf)	Cum. (1- C)	DR*	Sapp
Ala'	S21	7265	0.96	2440	3.39	1265	0.518	16.8
	S22	7081	0.94	21490	4.33	12463	0.580	21.9
	S23	6793	0.90	9260	3.97	4466	0.482	13.4
	S24	7062	0.94	18860	4.28	10161	0.539	18.8
Kazuo	KTraining_6A_test1	7274	0.96	9280	3.97	5275	0.568	22.8
	KTraining_test2_6B	7726	1.02	9460	3.98	5270	0.557	20.5
	KTraining_test4_6D	7764	1.03	24990	4.40	14092	0.564	21.1
Dong	TR-(7-1)-18C	7678	0.97	7310	3.86	4202	0.575	24.3
	TR-(7-2)-18C	7448	0.95	9100	3.96	5087	0.559	23.5
	TR-(7-3)-18C	7540	0.96	2980	3.47	1376	0.462	14.4
Usman	S21	8111	1.07	6650	3.82	3257	0.49	18.3
	S22	7828	1.03	10420	4.02	5840	0.56	19.2
	S23	7721	1.02	9770	3.99	5187	0.53	18.8

Paramters to be check in FlexMAT cracking

A2.2: FlexMAT Rutting

In order to proceed with shift model in FlexMAT rutting, the following parameters needs to be checked

Cyclic Fatigue Output/ Parameters check

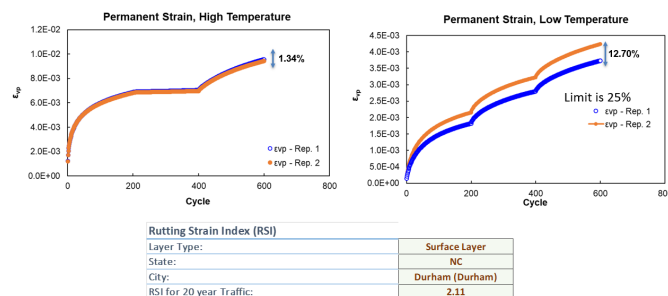
Average							
Operator	Fingerprint [E*] (MPa)	DMR, 0.85<DMR<1.15	Nf	log (Nf)	Cum. (1-C)	DR ^a	Sapp
Ala'	6979	0.93	16537	4.19	9030	0.534	18.0
Kazuo	7588	1.00	14577	4.11	8212	0.563	21.5
Dong	7555	0.96	6463	3.77	3555	0.532	20.7
Usman	7887	1.04	8947	3.94	4761	0.527	18.8

Standard deviation							
Operator	Fingerprint [E*] (MPa)	DMR, 0.85<DMR<1.15	Nf	log (Nf)	Cum. (1-C)	DR ^a	Sapp
Ala'	132	0.02	5256	0.16	3361	0.04	3.54
Kazuo	223	0.03	7364	0.20	4158	0.00	0.97
Dong	95	0.01	2569	0.21	1582	0.05	4.48
Usman	165	0.02	1646	0.09	1097	0.03	0.39

Paramters to be check in FlexMAT cracking

Parameter	Parameter Value	Limit	Acceptance
v_{norm}	4.86	22.00	PASS
D^R Range	0.071	0.088	PASS

Vnorm check in FlexMAT cracking



Shift model parameters check

A3: FlexPAVE Analysis

Introduction

The FlexPAVE program is an efficient pavement analysis tool and efficient framework developed by combining time-scale separation and layered viscoelastic analysis.

- Time-scale differences among temperature variations, traffic frequency variations, and fatigue/rutting.
- Stress-strain analysis can be performed using Fourier transform-based layered structural analysis
- Efficiently captures the effects of the viscoelasticity of the pavement material, the temperature (thermal stress and changes in the viscoelastic properties), and the moving nature of the traffic load

FlexPAVE analysis option

Two types of analysis can be done

1.Pavement Response Analysis: Produces only stress-strain and displacement results

2.Pavement Performance Analysis: Gives all the information, including fatigue cracking, thermal stress, rutting data, etc., depending on the selection of the performance items.

Inputs required for FlexPAVE

Material Properties

The material properties need to be selected for each material according to the tested mixtures in AMPT. Furthermore the dynamic modulus, cyclic fatigue and SSR data should be imported from FlexMAT cracking and rutting

General Information **Design Structure**

Structure General Information

Structure Name: Flexible 3-Layer Pavement

Pavement/Lane Width (m): 3.65

Add Layer Remove Layer Move Layer

Layer Properties

Layer: AC

Thickness (cm): 10 ☐ Infinite Layer

Material Type: Asphalt Concrete

Specific Gravity (optional): 2.5 Expansion Co. (1/C): 0.00005

GR Based Criterion ☒ DR Based Criterion ☐

Strength/Modulus

Poisson's Ratio	Einf (KPa)	Ref. Temp. (C)	Shift Factor a1	Shift Factor a2	Shift Factor a3
0.3000	9.7300e+04	5	6.9619e-04	-0.1620	0.7928

Fatigue

Alpha	C11	C12	Initial C	Gamma	Delta
4	0.0017	0.5449	0.8000	1000000	1.3580

Rutting

Beta	Epsilon0	NI	TR(C)	p1	p2	d1	d2
0.8026	0.0052	0.8024	61	0.6069	0.0719	0.0396	1.6831

Import Damage Data Import Rutting Data

Please note that FlexPAVE 1.0 uses the power function with the C11 and C12 coefficients to define damage characteristic curve instead of an exponential function.

Import Prony Series Data Help...

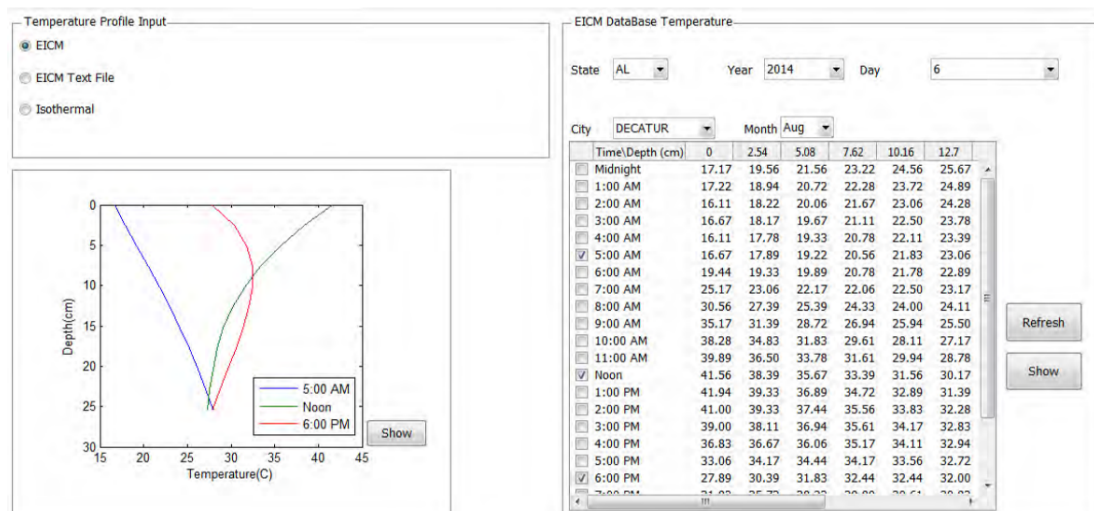
Material properties in FlexPAVE

Traffic data

This section allows users to select the type of traffic data used for analysis. At present, FlexPAVE only supports design Vehicle analysis. A Traffic Spectrum option will be introduced in future versions [12]. For the mixture tested at NC state USA, the details for traffic data have been provided in the traffic data section. Please note that for each mixture the traffic data was entered manually considering the heavy traffic for the worst condition in each case of the mixture.

Climatic Data

To simulate the temperature conditions for the Sicily region, the manual input option in the temperature profile was entered. The pavement temperature was considered according to the hot climate conditions of the Sicily region [8]



Climatic data in FlexPAVE

The figure displays two screenshots from the FlexPAVE software interface. The left screenshot, titled 'Design Vehicle Information', shows a diagram of a truck with three axles: Single axle, Tandem axle, and Tridem axle. Below the diagram are input fields for 'Choose a Vehicle' (New Vehicle), 'Special Truck', 'Axle Type' (Single Axle), 'Wheel Type' (Single Tire), 'Distance (m)' (0), and 'Axle Load (kN)' (80). The 'Design Velocity (m/s)' is set to 27. The right screenshot, titled 'Traffic Information', shows input fields for 'AADTT' (750), 'Growth Type' (Linear), and 'Growth Rate (%)' (2). Below these are 'Monthly Adjustment Factor' (MAF) and 'Hourly Truck Distribution (%)' (HTD) tables. The MAF table shows values for each month from January to December. The HTD table shows values for each hour from Midnight to 6:00 PM.

Month	MAF
January	1
February	1
March	1
April	1
May	1
June	1
July	1
August	1
September	1
October	1
November	1
December	1

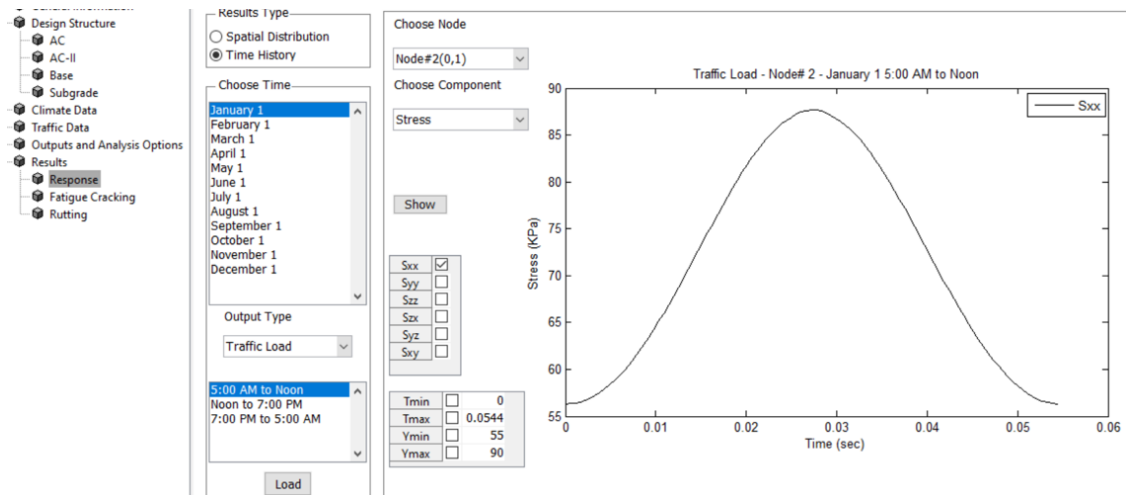
Hour	HTD
Midnight	4.1667
1:00 AM	4.1667
2:00 AM	4.1667
3:00 AM	4.1667
4:00 AM	4.1667
5:00 AM	4.1667
6:00 AM	4.1667
7:00 AM	4.1667
8:00 AM	4.1667
9:00 AM	4.1667
10:00 AM	4.1667
11:00 AM	4.1667
Noon	4.1667
1:00 PM	4.1667
2:00 PM	4.1667
3:00 PM	4.1667
4:00 PM	4.1667
5:00 PM	4.1667
6:00 PM	4.1667

Traffic data in FlexPAVE

FlexPAVE Outputs (Response)

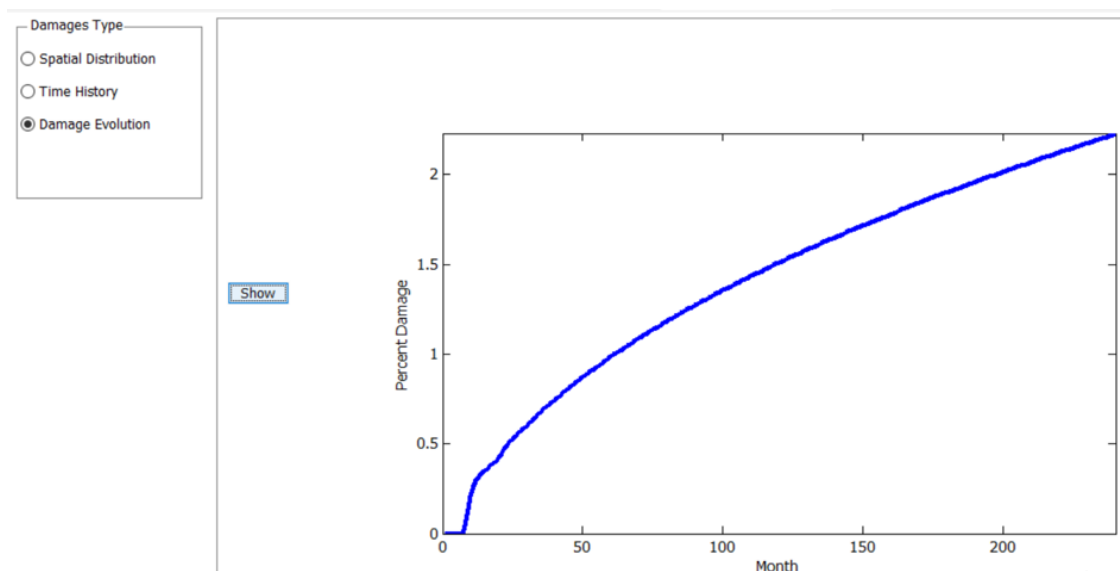
This analysis provides stress, strain, and displacement results.

Pavement performance analysis: This more comprehensive analysis includes all response data plus additional performance metrics, such as fatigue cracking, thermal stress, and rutting, based on the selected performance criteria. For the analysis for any type of mixture this option has been selected for further analysis.



FlexPAVE outputs response

FlexPAVE Outputs (Fatigue Cracking)



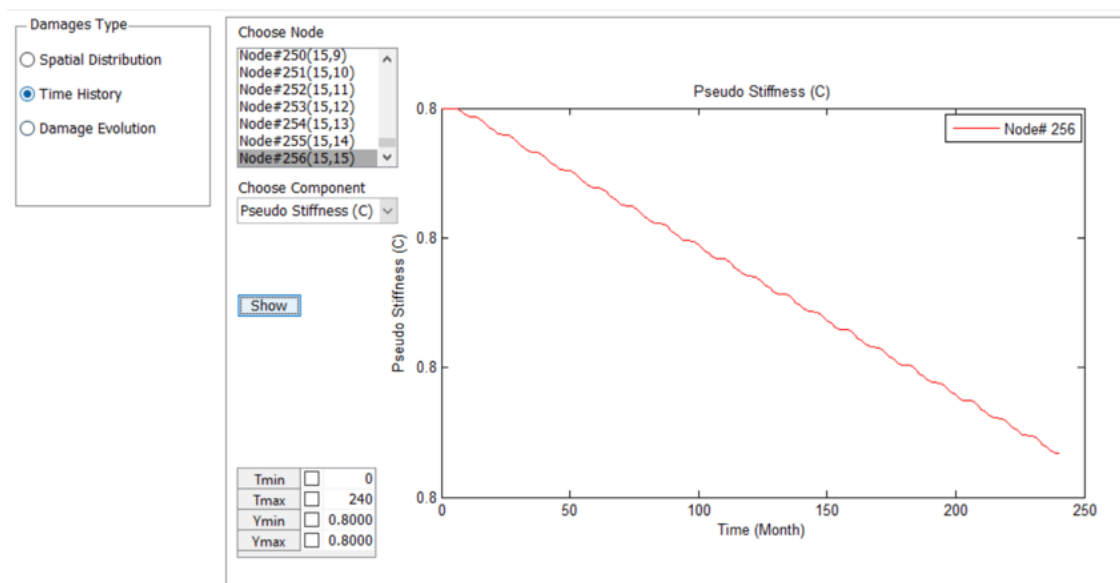
FlexPAVE outputs Fatigue cracking

FlexPAVE Outputs (Pseudo stiffness)

FlexPAVE allows to analyze fatigue damage for any type of mixture by plotting damage (C), which represents the modulus after damage [147]. This can be visualized as:

Contour plots: It shows damage distribution at a specific point in time.

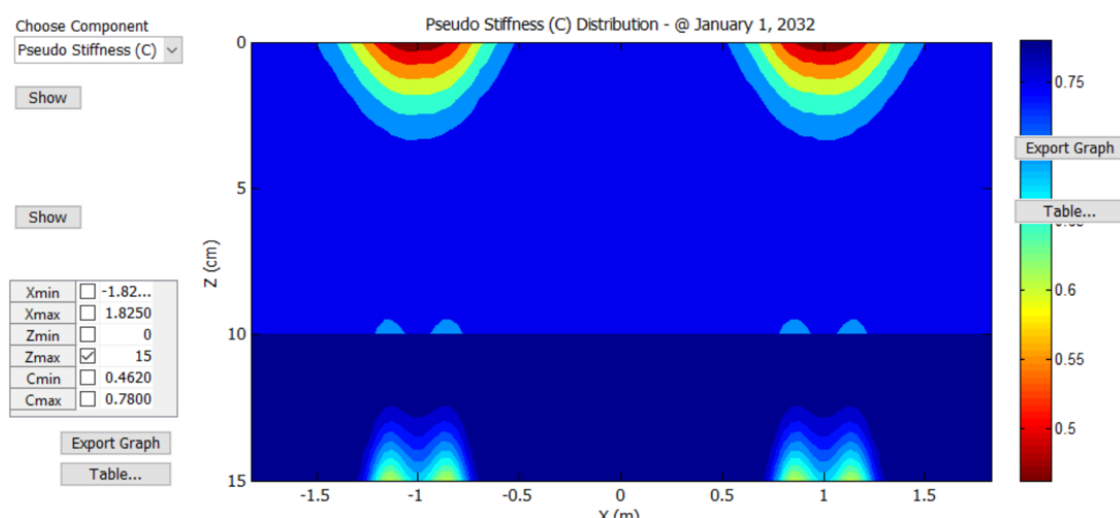
History plots: It tracks damage progression at a predefined node [148, 149]. The damage factor distribution, defined as N/N_f [150] where: N = Current number of load cycles N_f = Number of cycles to failure. When the damage factor reaches 1.0, the asphalt element is considered fully cracked.



FlexPAVE Outputs (Pseudo stiffness)

FlexPAVE Outputs (Damage analysis)

FlexPAVE enables, to track damage evolution over the entire pavement lifespan for any type of mixture. This is visualized through plots that display how damage progresses over time. It allows percent damage evolution as a function of time, providing insights into the long-term deterioration of the pavement structure.

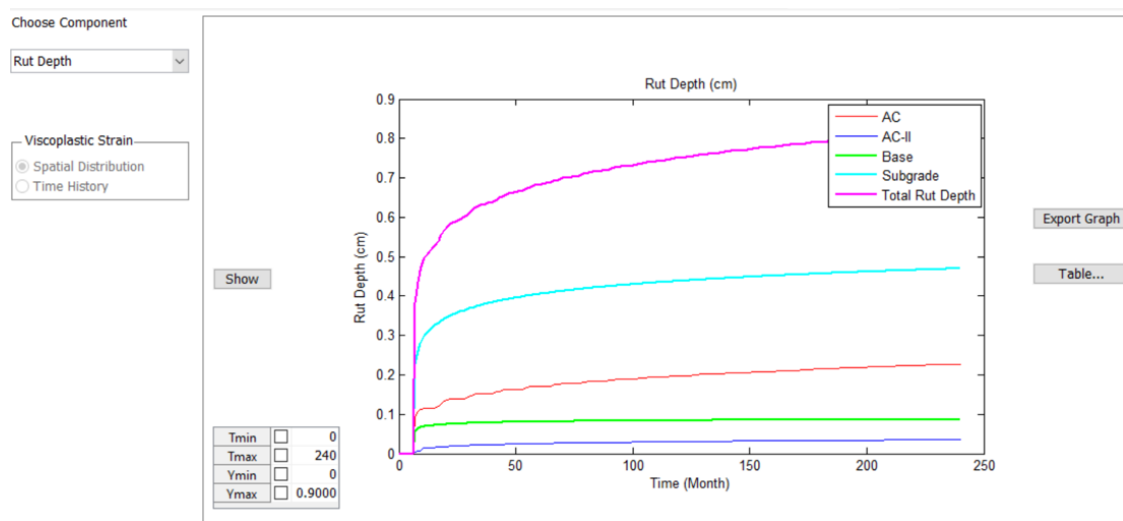


FlexPAVE Outputs (Damage analysis)

FlexPAVE Outputs (Rutting)

For the rut depth and permanent deformation analysis of any mixture, the [shift](#) model incorporated in FlexPAVE allows to predict the rut depth over the design of the pavement structure. It is important to note that a rutting database should be created for any type of region to be tested. Here it is to be noted that for the Raleigh region, United states the rutting database was already available in FlexMAT rutting. This database is utilized for shift

model and as well as in FlexPAVE simulations for the rut depth analysis



FlexPAVE Outputs (Rutting)

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