PERFORMANCE EVALUATION OF STYRENE BUTADIENE STYRENE AND NANO-CLAY MODIFIED ASPHALT CONCRETE MIXTURES

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A thesis submitted in partial fulfilment of

the requirements for the degree

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in
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DEDICATION

I dedicate this Research to my Parents, Teachers and Colleagues without whom help, support and prayer this task would not be accomplished.

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In the end, I express my solemn gratitude with a genuine sense of reverence to my parents for their encouragement, heartfelt prayers, and kind wishes for the successful completion of my studies along with this research work. Mainly, I want to thank my parents from all my heart for always being with me and for motivating me at every point of my life. Without their support, nothing would have been possible.

(*Engr. Asmat Khan)*

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LIST OF ACRONYMS

AASHTO	-	American Association of State Highway & Transportation Officials
ASTM	-	American Society for Testing and Materials
BS	-	British Standard
DOT	-	Department of Transportation
ESAL	-	Equivalent Single Axle Load
AC	-	Asphalt Concrete
HMA	-	Hot Mix Asphalt
ARL	-	Attock Refinery Limited
SBS	-	Styrene Butadiene Styrene
NC	-	Nano Clay
DWT	-	Double Wheel Tracker
SIP	-	Stripping Inflection Point
HWTT	-	Hamburg Wheel Tracking Test
DS	-	Dynamic Stability
IDT	-	Indirect Tensile Test
ITS	-	Indirect Tensile Strength
M_R	-	Resilient Modulus
NCHRP	-	National Cooperative Highway Research Program
NHA	-	National Highway Authority
NMAS	-	Nominal Maximum Aggregate Size
OBC	-	Optimum Binder Content
SGC	-	Superpave Gyratory Compactor
SHRP	-	Strategic Highway Research Program
TSR	-	Tensile Strength Ratio
UTM	-	Universal Testing Machine
V_a	-	Air Voids
VFA	-	Voids Filled with Asphalt
VMA	-	Voids in Mineral Aggregate
LVDT	-	Linear variable differential Transducer

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ABSTRACT

Asphaltic materials are extensively used in construction of highways around the globe, owing to low initial cost compared to other materials. However, flexible pavements' propensity to rutting and fatigue cracking reduces its service life and necessitate quick repairs. The utilization of polymers and nanomaterials in Hot Mix Asphalt (HMA) is one of current research focus and have shown improvement in its performance by enhancing service life and decreased pavement distresses, such as rut resistance, stiffness modulus and resistance to moisture damage. Polymer and nano modified asphaltic materials are suitable solution for improvement of mechanical properties of HMA. This study focusses on the experimental evaluation of SBS and Nano-clay (NC) applicability in HMA, by incorporating 4.5% and 2%, 4%, 6%, 8% to the weight of bitumen respectively. Control and modified asphalt mixes were subjected to performance tests, such as Resilient Modulus (M_R), Indirect Tensile Strength and Hamburg Wheel Tracking Test. Results indicates that addition of SBS and Nano-clay has improved the HMA stiffness, rut resistance and moisture resistance. It has been observed that **4.5% SBS with 6% NC** content by weight of the binder content in mixture **outperformed** the other percentages of the modifier materials evaluated in the study, where **rut resistance** and **Resilient Modulus** have been **improved by 39% and 40%**, respectively, while **moisture susceptibility** has been **reduced by 22.6%.**

CHAPTER 1

INTRODUCTION

1.1. Study Background

Pakistan is number five in ranking in the list of countries by population. Due to social, economic factors, the country is urbanizing at a fast rate. The fastest urban growth in South Asia is happening in Pakistan, at a rate of 3% annually. The proportion of people living in urban areas has climbed from 17.7% to 36.4 percent between 1951 and 2017. Pakistan has the fastest pace of urbanization in South Asia, with 36.4% of the population residing in urban areas. By 2025, cities will be residence to over half of the nation's population. (UNDP 2018, 2011).

As there is a rapid increase urbanization, there is a significant increase in economic and social activities for which a major requirement is transportation improvement. In Pakistan, roads are the primary mode and most frequently used, transportation accounting for 92% of passenger and 96% of freight traffic. The need to protect this infrastructure needs appropriate, cost-effective, and long-lasting preservation methods that will not only lower rehabilitation and maintenance costs but also offer safe and affordable transportation services.

It is a grim fact that certain stretches of roads are prone to various forms of wear and tear. These have always existed, dating back to the first-time flexible paving roads were built. It is apparent that deterioration causes the pain of the roads for drivers to increase. Even after each maintenance cycle, the irregularity will often reappear in the same pattern of deterioration. Its poor design or other contributing elements are to blame for this. The fact that the maintenance requirements for deteriorating behavior vary greatly from one location to the another, based on the characteristics of the asphalt pavement constituents, is one of the most remarkable characteristics of this behavior. The deteriorations in pavement design and specification are numerous and frequently unabated (Kim, 2010).

It won't have escaped the attention of a road user that most roads in the world have persistent distortion, or rutting, that is one of the most common types of asphalt pavement deterioration. Another significant pavement degradation caused by high ambient temperatures and uncontrolled axle loads is rusting.

As described by Donkor (2005), Rutting "demonstrates itself as a longitudinal bowl-like surface depression in the wheel paths on flexible pavements" with the application of traffic loads. Because of this, the deformed path frequently produces lateral movement when it comes into touch with tire pressure, which could cause the asphalt layers to be thinner and cause minor side upheavals. Rutting can happen structurally, which is when the pavement layers fail, or non-structurally, which is when it only affects the bituminous layers. Basically, the rutting process is frequently an amalgamation of both mechanisms' results. (Rahman, 2004).

Cognitively, the main causes of rutting in hot mix asphalt (HMA) could be due to either consolidation at the premature stages of a pavement's life, Rutting might result from a lack of compaction when traffic loads from the upper layer spread and transfer to the layers underneath it, or it may be caused by bituminous mixtures with insufficient shear resistance. Therefore, it is a safe assumption that there will be tensile strain in another direction if there is shear in the first. As a result, the intensity of the deformation is dependent on the load pattern. ((Collop et al., 1995); Donkor (2005); Khan 2008; (Miljkovi & Radenberg, 2011) and (Tarefder et al., 2003)).

Different techniques are offered to escalate the functionality of the asphaltic concrete, which extends the pavement's useful life. Since the 1980s, polymer modified binders have received a lot of attention in an effort to reduce the distress caused by rutting, cracking, raveling, shoving, potholes, etc.(Sohel et al., 2020). The study discovered that polymer could improve the tensile and cohesive strength of asphalt mixtures by generating higher tensile strength than bitumen. The most well-known method for the overall improvement of mechanical properties of asphalt mixture is polymer modification. The utilization of polymer reinforcement began as early as the 1990s and styrene butadiene styrene is a well-known polymer has been used in many research studies showing good performance in pavement. Polymers used to enhance asphalt mixture properties.

Resilient Modulus (M_R) is a fundamental material property to estimate material stiffness. The test to evaluate the stiffness properties of asphalt concrete was performed under (ASTM D7369, 2009). It shows the material subjected to dynamic stress and corresponding strain under different conditions. For rapidly applied loads, as those experienced by modern highways, resilient modulus is defined as stress divided by strain.

For the moisture susceptibility, Tensile Strength Ratio (TSR) gives a measure of moisture susceptibility of asphalt concrete mixtures. Moisture damage in bituminous mixtures is defined as the loss of serviceability caused by moisture. The higher the TSR value indicates high water-resistant of the testing specimen.

Hamburg Wheel Tracking Test ($HWTT$) is a popular technique for evaluating the rutting resistance of HMA mixes. The main feature of this is that a wheel is running through the specimen and the LVDT attach to the wheel record the rutting at every wheel and the computer program plot the given data. The test run for a maximum of 20000-wheel passes. The final rut depth shows the rutting depth of the tested specimen.

In this research, we will investigate the suitability of the application of SBS and NC combinations as modifier in AC mixes. Performance evaluation will be carried out by subjection of controlled and SBS and NC modified asphalt mixtures to performance tests such as Resilient Modulus (M_R), Tensile Strength Ratio (TSR), Hamburg Wheel Tracking Test ($HWTT$). Different percentages of NC with a 4.5% of SBS will also be utilized in asphalt mixtures and their comparative analysis will also be a major field of interest.

1.2 Problem Statement

Major distresses of the flexible pavements like rutting and moisture damage are extensively involved in reducing the pavement life. So, to counter this, materials used for the construction should be such that it will improve the resistive measure of the binder materials. Styrene Butadiene Styrene (SBS) has shown good performance against rutting and direct tensile stresses. But it is has shown that it has low performance at storage. So, to counter this issue a partner material in addition to this material should be used. In order to enhance the performance of the pavement, Nano-clay should be utilized in addition to Styrene Butadiene Styrene.

Pakistan National Road infrastructure comprises approximately 260,000 km. Data indicates that a considerable financial allocation is done in terms of pavement maintenance. Pakistan is a poor and developing nation and cannot afford to invest a large portion of our financial capital on new road building and repair due to the high financial implications. Considering such concerns, it is strongly suggested to utilize polymers to enhance asphalt mixture properties through modifiers such as SBS and NC. Polymers such as SBS and NC in combination addition would provide a good solution to these problems and requirements keeping in view Pakistan extreme climatic and vehicular loading on pavements. The study emphasizes investigating the rutting potential, stiffness modulus and tensile strength of SBS and NC modified asphalt mixtures. This research would contribute positively to challenging requirements faced by national highways. Research findings will be shared with the concerned government department to be implemented and to preserve capital investments on national highways.

To that end, this research will examine the moisture susceptibility, rutting and stiffness response of SBS and NC modified asphalt concrete mixtures in combination. For carrying out the performance testing, Marshall cylindrical specimens were prepared to assess the effect of varying percentages (2%,4%,6%,8%) of NC with 4.5% of SBS by weight of Marshall specimen, and to analyze experimental data collected from moisture susceptibility test (TSR), Indirect Tensile Test (IDT) (ASTM D6931, 2012), Resilient Modulus (M_R) and Hamburg Wheel Tracking Test ($HWTT$)(AASHTO T 324-11). The experimental matrix for Marshall mix design and performance testing is shown in Table 1.1 and Table 1.2. Table 1.3 describes the performance tests conducted in this research.

Table 1.1 Experimental Matrix of Bitumen and Aggregates Testing

Characterization	**Gradation**	**NHA – B**
	Binder	ARL 60 / 70
	Aggregate	Super Babuzai Crush Plant, Katlang, KPK
Materials	**Tests**	**Standard**
Binder	Penetration	ASTM 5
	Softening Point	ASTM D36-06
	Flash & Fire Point Test	ASTM D92
	Specific Gravity	ASTM D70
Aggregate	Fractured Particles Test	ASTM D5821
	Aggregate impact value	BS 812
	Aggregate crushing value	BS 812
	Los Angeles Abrasion test	ASTM C131
	Flakiness & Elongation Index	ASTM D4791
	Deleterious Material Test	ASTM C142
	Specific gravity & Aggregate	ASTM C127
	water absorption Test	ASTM C128

Table 1.2 Marshall Mix Samples for Determining OBC

Description	**Bitumen Content (%)**	**No. of Samples**
Conventional Samples	3.5	3
	4	3
	4.5	3
	5	3
	5.5	3
	Total	**15**

For the study, the sample count was 81 in which 12 was gyratory compacted samples for Hamburg Wheel Tracking Test (HWTT) test and 52 were Marshall compacted specimens including Marshall mix design and performance testing.

Table 1.3 Performance Testing Matrix of Asphalt Concrete Mixtures

Tests	Standards	SBS %	NC %	Samples	Total
Moisture Susceptibility	ASTM D 6931 - 17	0	0	3x2	**36**
		4.50%	0%	3x2	
		4.50%	2%	3x2	
		4.50%	4%	3x2	
		4.50%	6%	3x2	
		4.50%	8%	3x2	
Hamburg Wheel Tracking Test	AASHTO T-324	0	0	2	**12**
		4.50%	0%	2	
		4.50%	2%	2	
		4.50%	4%	2	
		4.50%	6%	2	
		4.50%	8%	2	
Resilient Modulus	ASTM D7369 - 20	0	0	3	**18**
		4.50%	0%	3	
		4.50%	2%	3	
		4.50%	4%	3	
		4.50%	6%	3	
		4.50%	8%	3	
Total					***66***

1.3 Research Objectives

This research has been subjected to achieve the following objectives:

1) To investigate effect of Styrene Butadiene Styrene and Nano-clay as modifiers on bitumen/asphalt binder.
2) To evaluate the performance of Styrene Butadiene Styrene/ Nano-Clay modified asphalt concrete mixtures using Wheel Tracking, Resilient Modulus, and Indirect Tensile Strength Tests.
3) To determine efficacy of Indirect Tensile Strength (IDT) and Hamburg Wheel Tracking Test (HWTT) tests for moisture susceptibility of SBS/ NC modified HMA.

1.4 Scope and Limitation

To accomplish the stated objectives, a methodology for research was formulated. A few of the key tasks are mentioned as follows.

- A literature review based on Marshall Mix Design, SBS and NC was carried out to get an insight into SBS and NC in combination effect as a modifier on asphalt concrete specimens.
- This study involved four different percentages of NC with 4.5% of SBS in combination in HMA Marshall specimens with bitumen procured from ARL 60/70 and aggregate from babuzai quarry at Katlang.
- The fundamental components of an asphalt mixture, namely aggregate and binder, were analyzed in the laboratory to learn about their characteristics and to assess whether materials qualify the minimum standards as per specifications.
- Optimum Binder Content (OBC) was determined from Marshall Mix Design which was further utilized in the modified HMA specimens.
- Using the OBC from Marshall mix design, the 4-inch Marshall control and modified specimens were used to evaluate moisture susceptibility by Tensile Strength Ratio (ASTM D4867), Stiffness response through resilient modulus test (ASTM D 7369-20) and rutting resistance by Hamburg wheel tracking test (AASHTO T 324-11). The results of the performance tests listed in the test matrix were interpreted in the following chapters.

1.5 Organization of Thesis

This thesis is organized into five (5) chapters. Every chapter is briefly described below:

- **Chapter 1** The first chapter provides a short overview of the modified HMA mix performance test used in the study, as well as the issue description, goals, and scope of the study.
- **Chapter 2** gives includes previous research findings of SBS and NC modified asphalt mixtures along with the different tests with their significance and their procedure the study of previous research studies on SBS and NC combination modified asphalt mixtures and their response related to rutting and stiffness test parameters.
- **Chapter 3** explains the methodology adopted to achieve the research objectives. It includes the selection of material characterization, determination of OBC, utilization of SBS and NC in asphalt mixtures and analysis of results of control and modified asphalt mixtures.
- **Chapter 4** specify the experimental outcomes and analysis conducted on the control and modified Marshall specimens from M_R, $HWTT$ and TSR performance tests.

- **Chapter 5** notify about the results and conclusions of this research work. In this chapter, we have also focused on future research frontiers and how we can adopt the findings of this research study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Hot mix asphalt (HMA), or flexible pavement roads, are typically thought of as having a service life of at least two decades. This consideration, though, is not always possible. According to experimental and field results from reading literature on the characteristics of pavement components and mixing proportions, this is presumably due to a number of factors, including structural (pavement layers) and non-structural elements (material behavior) (Roberto Firmeza Soares, 2005).

Because of changes in traffic patterns, multiple severe traffics loads (axle loads), new tire designs (with high pressures), and temperature variations, flexible pavement distresses have significantly increased in recent years. From this vantage point, experts analyzing the key factors influencing this problem have given special attention to decreasing the roadway deterioration. Furthermore, polymer cum nanomaterial modified asphalt, which has gained more head from the road construction world in recent years, is suggested as a way to mitigate the effects of contributing elements. (Taylor & Airey, 2007). Throughout an asphalt pavement's service life, permanent deformation has been shown to be a persistent issue. Understanding the mechanisms behind the rutting process is therefore essential for improving design in order to achieve adequate durability. Although the foundations of producing permanent deformations are pretty well understood, the principal function of estimating its growth rate quantitatively is challenging and open to interpretation (Archilla & Asce, 2006).

To better comprehend potential pavement engineering recommendations, it is vital to analyze earlier studies in this field. Therefore, the main goal of this study of the literature is to look at the crucial elements that result in the creation of a good model for modified asphalt mixtures that avoid irreversible deformation. In order to achieve the goals of this research, a number of studies on rutting and modified mixtures have been reviewed, introducing the key causes of rutting to get an updated understanding of the fundamental properties of the primary constituents of HMA with and without utilizing, polymer cum nano material modifiers.

Moisture damage, caused by the loss of bond between the asphalt binder/mastic and the aggregate under traffic loading and moisture condition, can cause a serious decrease in strength and durability in asphalt mixtures. Moisture damage can be attributed to direct adhesive failure or cohesive failure. Adhesive failure is characterized by the separation of the asphalt coating from the aggregate. It is caused by the action of water at the asphalt-aggregate interface(Lv et al., 2022).

Different theories as chemical reaction, molecular orientation, mechanical adhesion and surface energy are used to explain this adhesion bond. This research study also highlights how changed asphalt mixtures may enhance pavement characteristics against long-term deformation. This literature analysis also emphasizes how changed asphalt mixtures might enhance pavement characteristics against persistent deformation.

2.2 Hot Mix Asphalt (HMA)

The bituminous paving mix, Hot Mix Asphalt (HMA), is a blend of properly graded aggregates that are consistently mixed and covered with bitumen to make a road (Kanhdal & Koehler, 1989) Aggregates and bitumen are heated before mixing to achieve fluidity in the bitumen, which is necessary for proper mixing.

Design considerations for HMA pavement structures should be like those for other engineering constructions in terms of cost and durability. Premature failure of a pavement due to poor design results in higher repair costs. The best method to avoid future repair and maintenance issues is to determine the suitable construction materials and utilize proper design criteria for flexible pavements (Kanhdal & Koehler, 1989).

The most expensive material in HMA pavements is a bitumen. To make durable and economical pavements, the bitumen should be made more durable and resistant to pavement distresses, including fatigue, rutting, stripping, etc. The bitumen can be made more durable by adding certain modifiers which enhance its properties and make it more resistant to moisture-induced damages, rutting, and other pavement distresses.

2.3 Flexible Pavement Distresses

It appears that different experts have different classification systems for the major types of distress. The most prevalent road surface distresses have been categorized by (Lavin, 2003):

- Potholes
- Fatigue cracking: longitudinal, transversal, reflecting, block, and alligator
- Surface defects: raveling, flushing and polishing Surface deformation: rutting

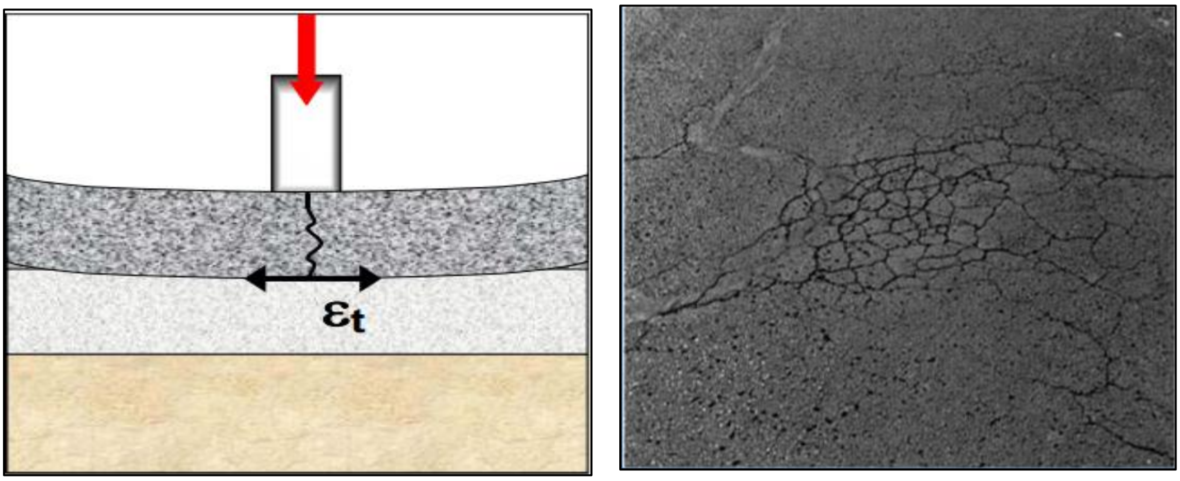
2.3.1 Potholes

One of the important issues that affects road users is pothole anguish, which is caused by cracks that form as a result of traffic loads and outsources. Starting with tiny ones, these can enlarge to larger pits that are lesser than 1m diametrically. Additionally, the size of potholes grows along with their depth, and their vertical edges might emerge from the top boundary with water present throughout the season. There are many elements that can cause potholes, based on the asphaltic road layers and the characteristics of ingredients like bitumen and aggregate (Lavin, 2003). According to their shapes, potholes can be classified as three types:

- Low severity; less than 25mm in depth and less than 450 mm in diameter.
- Medium severity; 25-50mm in depth and more than 450 mm in diameter.
- High severity; more than 50mm in depth and more than 450 mm in diameter

2.3.2 Fatigue Cracking

Asphalt pavement fractures are regarded as a primary source of damage and take on a variety of shapes depending on the influencing variables This problem has been a very popular research topic so that all circumstances during mixing operations are examined and, consequently, harmful effects from most types of fatigue cracking can be avoided.



(a) (b)

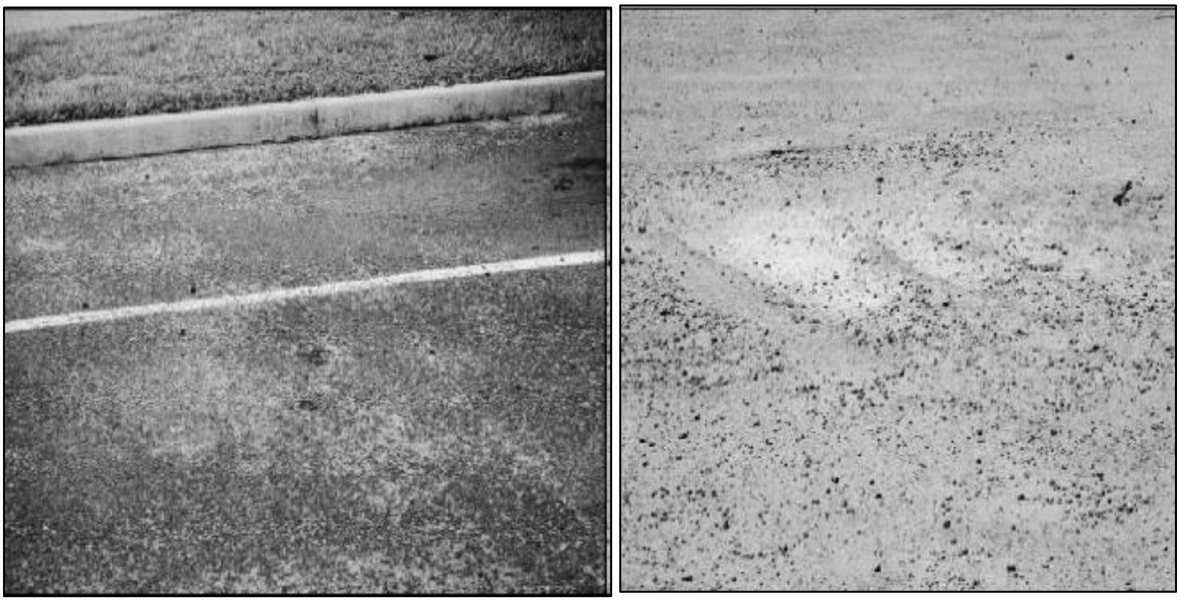
Figure 2.1: (a) Fatigue cracking initiation (b) Alligator Surface Cracks

Recently a study was carried out by (Moghaddam et al., 2011) predicts the alligator type (Figure 2.1 b), which is connected to repeated axle loads, is the most typical example of how fatigue fractures may manifest owing to diminishing service life. Tensile and shear forces consequently have an impact on the pavement layers' structure, which could result in cracks (Figure 2.1 a).

When identifying distresses and comprehending fatigue crack, experts generally don't really agree, according to a survey of various studies from the last decade. For instance, (Lavin, 2003) despite the fact that they each have separate contributing variables, transverse, longitudinal, reflecting, block, and alligator fatigue cracks are included. Fatigue crack is explained in detail by (Thom, 2008), who makes a tremendous advancement, in contrast to other engineers whose theories contain a great deal of ambiguity or lack of definition, exhibits a major advance in explaining the onset of fatigue fracture in words that are simple to understand. Considering fatigue in asphalt concreate (AC) surfaces as a homogenous material, like metallic materials, would not be a smart concept, he continues, as asphalt is hydraulically a multi-phase component.

2.3.3 Surface Defects

The elements that impact the asphalt pavement's top layer cause several flaws that are very dangerous for the amount of traffic they carry. These distresses as par (Lavin, 2003) might be classified to raveling, flushing, and polishing; the majority of these develop as a result of the asphalt pavement's surface layer being gradually removed as bitumen and aggregate are lost. This is possibly because temperature variations make asphalt brittle, which causes stones to separate from the binder material (Figure 2.2).



(a) (b)

Figure 2.2: (a) Raveling by stripping (b) High severity Raveling

2.3.4 Rutting

Repeated loads at high temperatures are typical distresses that cause rutting in asphalt pavements (Zhu et al., 2016). In the paving layers, Rutting is an adaptation of permanent deformation. In-wheel paths occurred because of the alliance of densification and shear deformation that appear in longitudinal depressions (Xu & Huang, 2012).

2.3.4.1 Factors Affecting Rutting of Asphalt Pavements

In Pakistan majority of the highways and Motorways do not show resistance to rutting in the early life of the pavement. There are several hot mix asphalt factors that contribute to deformation in flexible pavements, such as binder properties, gradation class(how coarse or fine), types of particles, and lastly, the amount of compaction effort applied. Furthermore, factors related to loading patterns include vehicle types, tire types and pressures, vehicle speeds, and axle loads. Environmental factors, including climate and pavement temperature, also affect the type and intensity of rutting.

2.3.4.2 Design Factors Contributing to HMA Rutting

Pavement layers' quality materials must be well-planned since, no matter how thickthe layers are or how tightly quality-controlled the construction method, rutting susceptibility can never be reduced if the material qualities are not appropriately developed. Adequate structural design of pavement layers, individual layer material characteristics, and construction quality assurance are all essential for flexible pavementsto perform successfully and satisfactorily, as stated previously.

2.3.4.3 Demerits of Rutting

Various reasons enforce the distress rutting to be considered as a phenomenon not desired in flexible pavements. It has numerous disadvantages, which affect road users as well as highway agencies. Some of these are discussed below.

- Rutting is a major contributing factor in causing hydroplaning because water accumulates in rut depressions. This accumulation of water can be dangerous in therainy season, as it reduces the skid resistance when brakes are applied.
- Rutting is responsible for causing functional failure of pavements by reducing the driver's comfort. Driver comfort is reduced because rut depressions are not uniform throughout the length of the road. This non-uniformity in rut depressions is a majorcause of driver discomfort.
- Rutting in flexible pavements is also responsible for the increase in vehicle operating costs. When a tire operates in a rutted section, there is more wear and tearof the tire. Secondly, the contact area of the tire with the pavement increases. Thus,tire friction increases, and as a result, fuel consumption increases. There is an increase in the fuel expenditure because the vehicle must make extra effort to overcome additional frictional resistance due to the rutted surface.
- Rutting also encourages the water to accumulate in the subgrade layer instead of draining out. Due to this, the base or subgrade layer becomes weak, and its load carrying capacity is reduced. The weakening of these base layers increases stress concentration on top surface layers; because of this phenomenon, early deterioration of pavement occurs.
- Rutting also causes safety concerns when vehicles travel at high-speed maneuver from one lane to the other. This observation is supported by the fact that the accidentrate

increases as the rut of pavement increases (Miljkovi & Radenberg, 2011).

Table 2.1 Factor Affecting Rutting Performance

Factors		**Change in factor**	**Effect of Change in Factor on Rut Resistance**
Aggregates	Surface Texture	Smooth to Rough	Increase
	Gradation	Gap to Continuous	Increase
	Shape	Rounded to angular	Increase
	Size	Increase in Maximum Size	Increase
Binder	Stiffness	Increase	Increase
Mixture	Binder content	Increase	Decrease
	Air Voids Content	Increase	Decrease
	Voids in Minerals Aggregates	Increase	Decrease
	Method of Compaction	--	--
Tests of Field Performance	Temperature	Increase	Decrease
	State of Stress/Strain	Increase in tire contact pressure	Decrease
	Load Repetitions	Increase	Decrease
	Water	Dry to Wet	Decrease when mixture is water sensitive

2.3.5 Moisture Damage of Asphalt Pavement

According to (Ahmed, 2014) the degradation of the toughness and durability of asphalt mixtures is known as moisture damage and is a result of the moisture effect. If the fine aggregate and asphalt binder don't have the necessary binding strength to maintain their bond integrity, moisture degradation may develop in asphalt mixes.

The relationship of moisture with the adhesion of the aggregate and binder in the asphalt mix increases the asphalt mixture's susceptibility to moisture during cyclic loading, which causes moisture damage. The criteria used for evaluation include ITS and TSR of minimum 80%, resilient modulus, striping slope and stripping inflection point (SIP).

According to the Washington State Department of Transportation, ITS is a very popular performance test used in the pavement sector to identify moisture damage. This test provides a reliable indicator of the mixture's crack propensity. Testing a mixture with and without moisture conditioning can help determine how sensitive the mixture is to moisture.

Study says moisture degradation happens when moisture present in air voids compromises the HMA's toughness and durability. There are two distinct types of moisture damage: cohesive failure and adhesive failure. Adhesive failure occurs between the aggregate and binder, whereas cohesive failure develops when the strength of the binder is decreased as a result of moisture degradation.

2.4 Polymer Modified Binders

Since the majority of new combinations within the conventional type of asphalt (HMA) fail to adequately prevent non-recoverable deformation, there is a strong tendency to adopt different types of polymers. The term "modified asphalt mixtures" refers to this procedure. Practical testing has shown that using polymer modified binders produces superior rutting resistance than using traditional HMA. ((Yildirim, 2007);(Ozen & Aksoy, 2007)).

2.4.1 Types of Polymers

Various kinds of polymers have been in very heavy demand lately. Researchers have suggested a wide range of polymers to fulfil the mechanical qualities and sustainability goals. Although modifiers have been used to improve the overall performance of asphalt pavement, it is still important to understand the physical and chemical characteristics of the asphalt and binder. It is still unclear exactly how polymers interact with such a complicated liquid as bitumen.(Yildirim, 2007). As a stepwise effort, according to (Lavin, 2003), the two categories of polymers are used to determine important mechanical properties, such as durability and viscosity of thermoplastic and thermoset asphalt binders. The first type is excellent for asphalt since it is primarily soft at high temperatures and hard at low ones. This process can be repeated. Thermoset, on the other hand, is soft under high temperatures and can become rigid at low temperature, but this process cannot be recycled. According to (Yildirim, 2007), the three types of thermoplastic polymers are elastomers, plastomers, and natural rubber.

2.4.1.1 Elastomers

Elastomers, a common form of polymer, are used to create a variety of synthetic thermoplastic rubber polymers, including polybutadiene, styrene butadiene rubber (SBR), styrene ethylene butadiene styrene (SEBS), and styrene butadiene rubber (SBR) and SBS. SBS is one among these that pavement engineers have been paying attention to. This may be due to a variety of variables, including its performance in asphalt components and economics. Due to the bitumen's elastic reaction, SBS can also improve the rheological characteristics of asphalt. After using and removing elastic recovery, this trait appears. As the force is removed, the viscous component also decreases. In actuality, these two aspects of the modified binders are crucial for preventing permanent deformation brought on by repetitive loads on the asphalt surface (Robinson, 2005). Despite the prospective advantages of modified binders, according to a related study by (Lavin, 2003), the polymers need to be applied precisely. This is done to achieve an appropriate equilibrium between the key bitumen constituents (asphaltenes, resins, and maltenes). For instance, the presence of maltenes is necessary for asphaltenes; otherwise, the workability of the binder may be impacted.

2.4.1.2 Plastomers

Plastomers, often referred to as crystalline polymers, poly-propylene, ethylene-vinyle acetate (EVA), ethylene methyl-acrylate (EMA), and poly-vinyl chloride, are emphasized as the second most frequently used polymers after elastomers (PVC). (Yildirim, 2007) claims that declining the temperature susceptibility of bitumen is important to control the possibility of rutting during hot summer seasons. It has been demonstrated that doing so results in a noticeably increased viscosity or stiffness of the asphalt binder, and as a result, elasticity behavior does not increase. Plastomers are less resistant to lower temperature cracking than elastomers are. EVA, a fundamentally random plastomer polymer, is the most common variety. This kind is perfectly acceptable because it offers a suitable level of resilience against cyclic loads. The basic operating principle of the polymers is that it effectively dissolves and splits inside the binder, as a result of a reduction in bitumen viscosity during elevated temperatures up to 100°C. As a result, the polymer tends to associate (recrystallize) at lower temperatures (below 90°C), stiffening the bitumen and raising its viscosity. As a result, it's crucial to make sure the asphalt thoroughly compacts before the EVA 28 phase changes. If it doesn't, the bitumen may harden quickly, creating inadequate densification, which could lead to premature failure.

2.4.1.3 Natural Rubber

The use of natural rubber in asphaltic road pavement to enhance binder qualities has gained popularity over the past few decades. Generally speaking, latexes (aqueous polymers) cannot alter asphalt concrete performance to the similar extent as elastomers or plastomers that are included into heated bitumen. Additionally, it is demonstrated empirically that rubber-modified mixes exhibit improved rheological properties, which are closely related to persistent deformation. When combined straight into the asphalt mixer without needing storage tanks, employing latex offers a workability that is appropriate in plants. It should be mentioned that rubber latex has several characteristics with synthetic thermoplastic polymers (Yildirim, 2007).

2.5 Styrene Butadiene Styrene (SBS)

SBS, also known as styrene-butadiene-styrene, is frequently utilized by pavement engineers and in the construction of highways. There are many advantages to adopting styrene butadiene styrene (SBS) as a thermo-plastic elastomeric polymer, based on the performances and qualities that apply to asphalt mixtures. At laboratory conditions, SBS may give an acceptable hardness in asphalt mixtures based on its optimal dosage, which is typically ranges from 3% from 6%, which can also upgrade visco-elasticity and prolongation. This is in addition to improving asphalt resistance to rutting. As a result, it may be possible to enhance adhesion, fatigue crack resistance, nonrecoverable resistance, and bleeding resistance. Due to the constant high loading and slow traffic movements in certain locations, it is suggested that it only be used at intersections (Sholar, 2005)

(Omrani et al., 2017) Support the aforementioned viewpoint by demonstrating how SBS can improve stiffness at high temperatures while also softening HMA at low laboratory temperatures. In light of this, these benefits might adequately improve HMA performance in both high temperature permanent deformation and cold temperature thermal cracking. To prove this point practically, it was shown that SBS has no effect on tensile strength when tested indirectly. An laboratory study was carried out by (Khodaii & Mehrara, 2009) the inclusion of coarse and dense graded mixtures can considerably alter mechanical characteristics, especially persistent deformation of both SBS-modified and unmodified asphalt mixes. The findings suggest that conducting dynamic Creep testing can identify irrecoverable deformation and that using varied SBS contents, coarse graded mixes produce lesser persistent deformations than dense graded mixes. It was discovered that 5% SBS shows an adequate effect on persistent distortion in the range of 4% and 6% SBS contents. Lower stresses, however, were discovered in Creep curves to have no impact on the test in adjusted mixes.

(Ozen & Aksoy, 2007) report that using SBS polymers could improve the performance of HMA. By performing wheel tracking and dynamic Creep tests at different loads and range of temperature, this was empirically confirmed. Additionally, changed asphalt mixtures' superior strength to controlled samples was shown by indirect tensile tests. To put it another way, changed mixtures' tensile strength appears that it is more resistant to tensile strains and fatigue cracks, which can result in a variety of fissures.

It appears that adding additional additives with SBS polymer enhances asphalt's mechanical qualities. (Chen & Huang, 2006) shown that adding Sulphur to SBS can improve asphalt's elastic recovery. The softening point of SBS can also be increased while penetration can be inhibited by adding different percentages of Sulphur. Similarly, (Tan, 2009) discovered that, although increasing the aforementioned capabilities, adding SBS to starch (ST) modified mixes might minimize moisture damage and temperature effects.

2.6 Nano Materials

Nano materials are that materials which are consists of the particles having sizes in nanometers. These materials are having large specific area. A new generation of perfect additives to enhance the characteristics of materials has just been created using nanoparticles (Jamshidi et al., 2015); (Yang & Tighe, 2013). Researchers in the pavement sector love adding nanomaterials as additives because they improve the characteristics of concrete and asphalt pavement (Sanchez & Sobolev, 2010).

Researchers in the pavement sector have paid particular attention to nano-clay (NC), one of the nanomaterials used to enhance the characteristics of asphalt mixtures (Ashish et al., 2017). NC has been able to be regarded as a complimentary ingredient in polymer modified binders because of its distinct qualities, including a high specific surface, surface electrical characteristics, and action exchange capacity (PMB) (Sadeghpour et al., 2010).

Because of its ease of manufacture, appropriate impact in small quantities, and low production costs, NC is one of the most widely used nanomaterials in the pavement sector and enhances the attributes of modified binders. (Ghaffarpour & Khodaii, 2009).

Precisely because of this novel material, nanotechnology is being employed in pavement engineering to improve asphalt materials using nanoparticles in the binder of Hot Mix Asphalt. Carbon Nanotubes, Nanoclay, Nano-silica, nanofibers, Nano-hydrated lime, Nano plastic powders, or polymerized powders have been or could be utilized in asphalt modification (You, 2013). Following types of Nanomaterials are used in bitumen so far.

2.6.1 Nano-Clay

The formation of common clays is subject to natural variation because they are naturally occurring minerals. The purity of the clay can have an impact on the properties of the nanocomposite. These sheets-like (layered) clays, which are composed of silica (SiO_4) bound to octahedrons (Al_2O_6) of diverse shapes, are the most common type of clay.The most common mineral clay is montmorillonite. Layers (platelets) of montmorillonitehave an average aspect ratio of 100–1500 (Ghaffarpour & Khodaii, 2009).

2.6.2 Carbon Nanotube

Carbon Nanotubes are the tubes of graphite carbon at the molecular scale and are among one the most widely used Nano-materials due to their strength, weight, considerable surface area, and compact scale. As compared to other modifiers, CNT incorporation improves substrate properties as well. As a result of their high strength, light weight, large surface area, and compact size, Carbon Nanotubes (CNTs) are among the most used Nanomaterials. As compared to other modifiers, CNT incorporation improves substrate properties. CNT dispersion of bitumen is a dynamic phenomenon because of the lower diameter to length ratio.

2.6.3 Nano Silica

Silica is a naturally occurring substance that is widely used in a variety of construction industries. The viscosity values of nano-modified asphalt were lowered slightly by adding nano-silica to the basal asphalt binder. The advantages of these nanomaterials include low production costs and great performance. Its properties and stress relaxation ability were identical to those of the conventional asphalt at low temperatures, as was the nano-silica modified asphalt binder. Improved anti-aging, fatigue, and rutting resistance of nano-silica-modified binder and HMA were also shown to be greatly improved (Yao et al., 2013).

2.6.4 Black Carbon

Carbon black is obtained by the incomplete combustion of heavy petroleum products, and its production occurs under controlled conditions via two processes:

- Thermal Decomposition of Liquid or Gaseous Hydrocarbons.
- Incomplete Combustion.

Carbon Black is finding its foot steadily in road engineering as an additive in asphalt binder for enhanced pavement performance. Carbon black filler as a bitumen modifier has positive implications on high-temperature performance, fatigue resistance, rutting depth, and other important engineering properties of the asphalt pavement.

2.6.5 Nano Graphene

Nano Graphene is graphene layers that have been stacked together. A monolayer of carbon atoms makes up a single graphene sheet. The carbon atoms in this structure are arranged in a hexagonal pattern. Graphene is stacked, rolled, and wrapped to make graphite and carbon nanotubes, respectively. Carbon allotropes can be synthesized usinggraphene as a building material.

GNPs to asphalt mixture pavements can improve a variety of compaction and mechanical properties, hence improving the pavement's durability and performance. The GNPs reinforced asphalt pavements have the potential for long-term applications in the road industry, including both new pavement and road construction and pavement repair,due to their cheap material cost relatively.

2.6.6 Nano Fibers

Carbon Nano-fiber (CNF) is known because of its high tensile modulus, high surface area, strong interfacial bonding, and high aspect ratio. Fibers with a high aspect ratio are thought to be capable of forming a strong nanocomposites network. In addition, nanofibers' bridge link effect can effectively prevent microcracks from forming because of the interaction between heavy vehicle loads and environmental influence. With the addition of CNF, the asphalt binder's mechanical and rheological qualities can be improved. (Li et al., 2017).

2.6.7 Nano Lime

Nano lime's ability to improve HMA such as rutting, fatigue, and moisture-damage resistance potential (Ashish & Singh, 2021). The optimum amount of lime used in asphalt binder is 5% without sacrificing final performance (Diab et al., 2012) . Reducing lime particles to nanoscale size enhances their surface area, which improves the binder-lime interaction (Das & Singh, 2018).

2.6.8 Iron Nanoparticles

Fe and iron oxides make up most iron nanoparticles. Depending on the amount of iron oxides in the mix, these materials can vary in color from red to brown to black. Zero-valent iron (ZVI), also known as zero-valent nano iron, is a commercially available type of Fe nanoparticles (nZVI). ZVI is a dry ferrous powder with alkaline properties that has a non-valent chain (pH from 11 to 12). Reactivity and the large specific surface area of ZVImay have a significant impact on the asphalt binder's characteristics (Crucho et al., 2019).

2.6.9 Nano Titanium dioxide (TiO_2)

Titanium dioxide is a naturally occurring form of titanium oxides such as Rutile, anatase, and brookite. Anatase (80%) and rutile (20%) make up the majority of nano-titanium dioxide. When compared to ordinary TiO_2, nano-TiO_2 has a much larger surface area, a much smaller diameter, and an extremely low opacity. Enhances the performance of modified asphalt by using nano-TiO_2. This is a brief summary of how TiO_2 nanoparticlesare produced in the reaction system. (Li et al., 2017).

2.7 Nano-Clay

Nano clay (NC) is most of the time used as a second modifier. When the active surface area of the NC (up to 700-800 m^2/g) is big, the NC and bitumen are more likely tointeract. One form of montmorillonite nano-clay had no effect on bitumen's stiffness or viscosity, whereas another type of montmorillonite Nano clay had an effect. However, oneof the Nano clays increased bitumen aging resistance in the short and long run. (Yang & Tighe, 2013).

As long as the nano-clay is mixed at the nanoscale, the bitumen's physical properties improve (such as tensile strength, tensile modulus, flexural strength, and modulus thermal stability). NC-modified bitumen has better elasticity and a lower dissipation of mechanical energy than unmodified bitumen (Ghaffarpour & Khodaii, 2009).

The asphalt binder was reinforced and modified by using bentonite clay (BT). The rutting resistance of the modified asphalts was greater. Asphalt fracture resistance and low-temperature rheological qualities were greatly enhanced by mixing with BT (Zare-Shahabadi et al., 2010). SBS copolymer-modified asphalt's performance attributes have been further improved using the secondary modification of nano clays (Yu et al., 2007). Inorder to reduce asphalt pavement permanent deformation or rutting, sodium montmorillonite nano clays show promising results in laboratory tests. As a cost-effectivesolution to improve polymer systems' mechanical, thermal, and barrier properties, nano clay has been proposed. It is made up of stacked layers of silicates. Dispersion of Nano clay in the hydrogel in homogenous manner results in an enhancement of its properties. But there is a difficulty when dispersion is done with organic polymers when they are in their extreme hydrophilic phase. There are comprehensive revisions on the application, penetration, and processing of Nano-Clays. Some common techniques of dispersion are

- Exfoliation
- Flocculation
- Intercalation

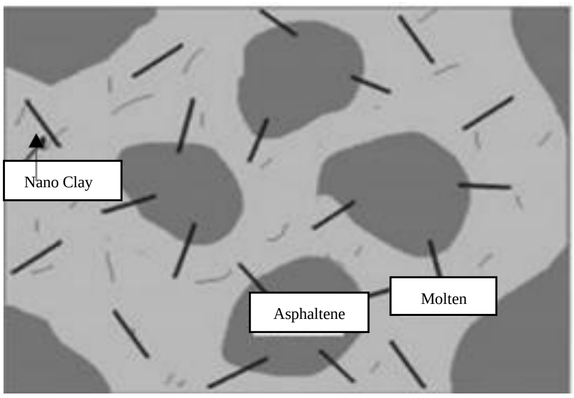


Figure 2.3 Nano-clay modified Bitumen microstructure

Nano-clay can be effectively used as a modifier in asphalt pavements in the regions exposed to a higher temperature, but Nano-clay modification is least desired for pavementsin areas where the temperature drops quite low, is because Nano-silica doesn’t add to the asphalt binder thermal cracking resistance at low temperatures.

2.8 Utilization of SBS and NC in Bitumen

Table 2.2 Overview of the Previous Research on SBS/ NC Modified Asphalt/ Asphalt Concrete

Research Paper Description	Modifiers Type & Percentage	Test Conducted	Findings
(Jahromi & Khodaii, 2009) **Construction and Building Materials**	Nano Clay (NC) 2, 4, 6, & 7% **Optimum 7%**	Dynamic Shear Rheometer (DSR)	• 7% Nano clay (NC) modified bitumen has a 225% increase in stiffness as aging is reduced at a higher frequency. • NC modification enhances bitumen stiffness and decreases phase angle, which results in aging effects reduction.
(Farias et al., 2016) **Construction and Building Materials**	NC and SBS 4% NC, 4% SBS, 4% of SBS with 5% NC	Dynamic Shear Rheometer (DSR) Multiple Stress Creep Recovery tests (MSCR)	• Complex modulus (G*) and phase angle (δ) of the bitumen were increased by the addition of SBS and SBS/NC. • NC enhances the SBS modified bitumen's ability to withstand storage stability at high temperatures. • SBS with NC-modified bitumen is more effective due to its improved elastic recovery and plastic deformation resistance.
(Ashish & Singh, 2018) **Construction and Building Materials**	Nano Clay and Carbon Nano Tubes NC 0, 2, 4, & 6% CNT 0, 0.4, 0.75, 1.5, & 2.25% CNT optimum 1.5%	Dynamic Shear Rheometer (DSR)	• Nano clay (NC) has a greater impact on asphalt binder stiffness than CNT. • In terms of viscosity, CNT has a far greater impact than NC. • To increase the rutting performance, CNT doses of up to 1.5% may be recommended. If the asphalt contains more than 1.5% CNTs, the agglomeration problem will reduce the asphalt's stiffness. • Maximum 2.25 % of CNT in asphalt binder can be utilized due to limiting 3Pa-Sec at 135°C viscosity value.
(Mousavinezhad et al., 2019) **Construction and Building Materials**	NC & SBS NC 0.5, 1.5, 2.5, 4,& 5% SBS 1, 3, 5, 8, & 10% Optimum NC 4% with SBS 8%	Dynamic shear rheometer (DSR) wheel track test (WTT)	• In addition, the complex modulus of NC and SBS modified asphalt binders increased while the phase angle decreased. Compared to an unmodified asphalt complex modulus (G*). • NC and SBS Modified asphalt have an enhanced elastic/ viscous capability. • NC 4% with SBS 8% exhibit lower permanent deformation (Rutting) due to a significant increase in binder stiffness and viscosity.
(Islam et al., 2021) **Construction and Building Materials**	SBS 3%, 4.5% and 7% by weight of binder, Optimum 4.5%	Linear Viscoelastic regime (LVE) Gel permeation Chromatography (GPC) MSTR	This study shows that the optimum content of SBS in bitumen is 4.5% by weight of bitumen. The work suggest that the storage affect the bitumen (SBS-Mb) unaffected up to a temperature of 150°C in closed containers for 3 to 7 days. But the storage at 180°C for 3 to 7 days affect the performance and degrade the Marshall stability by 33%, the rut depth increased by 68% and a drop in fatigue life

Research Paper Description	Modifiers Type & Percentage	Test Conducted	Findings
(Nian et al., 2018) **Construction and Building Materials**	(SBS) 5% by weight of bitumen	Dynamic Shear Rheometer (DSR) Multiple Stress Creep Recovery (MSCR)	This study shows that after a number of freeze-thaw cycles the viscous loss was excessive and the loss rate increased gradually. But due to SBS modifier in bitumen the elasticity was increased significantly, and the bitumen survives the water-temperature cycles. This shows that the SBS modified bitumen has low sensitivity to temperature than that of bitumen thus it improves the anti-rutting ability of the pavement. But with the increase in freeze-thaw cycles the fatigue resistance decreases.
(Alireza et al., 2020) **Construction and Building Materials**	SBS 3% NC 2%, 4% & 6% NL 4% & 6% by weight of bitumen Optimum (3% SBS) + (4% NC) + (6% NL)	Dynamic Shear Rheometer(DSR) Multiple Stress Creep Recovery (MSCR)	This study shows that the addition of nano-clay, nano-lime and SBS in combinations will improve the resistance against the permanent deformation of the pavement and will increase the rutting resistance; it will also improve the storage stability and temperature susceptibility. The MSCR test results shows that the SBS modified bitumen indicate positive effect on improving the rutting parameter greater at 58°C then other temperature.
(Mousavinezhad et al., 2019) **Construction and Building Materials**	Butadiene Styrene (SBS) Steel Slag Aggregates	Dynamic Shear Rheometer (DSR) Marshal Test Repeated Load Axial Test (RLA) Wheel Tracking Test (WTT) Static Creep Test Dynamic Creep Test	This study shows that bitumen treated with nano-polymers has increased viscosity and improved adherence to aggregate particles. The Marshal Test was used to determine the ideal bitumen concentration in mixtures of standard and modified asphalt. The resistance to rutting was then assessed using two tests: the wheel track test and the repeated load axial (RLA) test. The results showed that the addition of nano-polymer improved the bitumen rheological qualities while improving toughness and viscosity, respectively, by an average of 25% and 100% and lowering the penetration grade. Improvements were also seen in the rutting resistance and depth of the asphalt.
(Siddig et al., 2018) **Construction and Building Materials**	Ethylene vinyl acetate (EVA) copolymer and Nano-clay (NC) 1, 3, 5, and 7 wt%.	Dynamic Shear Rheometer (DSR)	Performance of asphalt binders at high temperatures is examined in this study in relation to the addition of nano-clay (NC) and ethylene vinyl acetate (EVA) copolymer. NC-, EVA-, and polymer-modified Nano-clay were created as three different types of modified binders utilizing melt blending at concentrations of 1, 3, 5, and 7 wt%. Their physical and rheological characteristics were assessed using dynamic shear rheometers, viscosity measurements, and common tests (penetration and softening point). The outcomes demonstrate that EVA and NC considerably enhance the binding qualities. Particularly, the improved rutting parameter following binder modification suggests that it performs well at high temperatures.

Research Paper Description	Modifiers Type & Percentage	Test Conducted	Findings
(Mansourian et al., 2019) **Construction and Building Materials**	Nano-clay with EVA and HDPE	Dynamic Shear Rheometer (DSR)	To discover the improvements in asphalt's qualities, numerous performance and traditional tests were combined with nano-clay, EVA, and HDPE. The test findings demonstrated that the polymer Nanocomposite can enhance the asphalt binder's low temperature resistance and rutting resistance, regardless of the viscoelastic behavior of the asphalt, which may be linear or nonlinear.

2.9 Resilient Modulus

Resilient Modulus measures the stiffness of the material. A pavement's "resilient modulus" is a measure of stress by the amount of strain it experiences in the form of traffic loads during its lifetime.

$$M_R = \frac{\sigma_d}{\varepsilon_r}$$

Where:

M_R = Resilient Modulus

σ_d = Repeated Deviator Stress (cyclic)

ε_r = Recoverable Strain

Pavement materials are stiff and, as a result, deform permanently with each load application. At this point in the load application process, a tiny amount of permanent deformation accumulates, which is represented by the plastic strain.

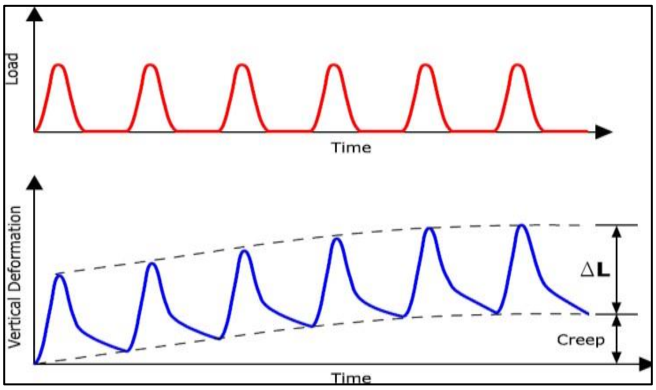


Figure 2.4 Recoverable strain under cyclic loading (with rest period)

The plastic strain increases with increment in load cycles. In the resilient modulus test, a compressive load is applied to a cylindrical HMA sample in the vertical using a haversine loading waveform, and the horizontal recoverable deformation is determined after 100 to 200 cycles of loading. When a normal-thickness asphalt layer is bent, the asphalt layer's radial stiffness resists the applied stress rather than the vertical stiffness. Consequently, Diametral test findings are most useful for calculating radial tensile strain in afatigue study. It is possible to test thin cores with this method, making it ideal for determining the thickness of thick asphalt layers. The Resilient Modulus under Indirect Tensile Tension setup is performed under the (ASTM D 4123, 1995).

2.9.1 Universal Testing Machine (UTM)

UTM is commonly used to test tensile strength, indirect tensile stiffness modulus, moisture susceptibility, fatigue test, etc., of asphalt concrete. The Universal Testing Systemconsists of the following parts:

- Control and Data Acquisition System (CDAS)
- Personal Computer with the compatible operating system as specified by the manufacturer
- A suite of UTM software and support files
- Pneumatic axial and pneumatic confining stress loading system
- Hydraulic Power Supply
- Environmental Chamber

The servo-feedback loading control electronics, as well as transducer data collection and timing, are all provided by CDAS. Overall system control is managed by the PC, whichis guided by the UTS application program. The PC processes, displays, reports, and archives the data collected by the CDAS during specimen testing. A PC-based pendant allows for axis-jogging operations as well as hydraulic power pack control.

2.9.2 Indirect Tensile Strength

The ITS with specifications as (D6931, 2011) is employed to assess the tensile strength of compacted Hot Mix Asphalt (HMA) samples. The tensile strength of HMA is important since it provides a strong indication of the mix's cracking propensity. High- tensile-strain HMAs are better able to withstand loads and resist cracking than their low- tensile-strain counterparts. A continuous rate of vertical deformation is applied until failureoccurs in the IDT test.

The compressive force is applied at a rate of 50 mm/min throughout the test at a temperature of 25℃. In the vertical diametral plane, the loading mechanism provides homogenous tensile stress transverse to the applied load. IDT test results in the HMA sample being split. When applied loads are supplied perpendicular to the vertical axis, thetensile stresses along the vertical diametric plane tend to be uniform.

Before conducting the resilient modulus test, the IDT strength value for each kind of combination compacted in the laboratory with identical mix characteristics is calculated. In performing resilient modulus testing, load values ranging from 10% to 20% of the ITS value obtained at 25°C should be utilized for each kind of combination. The ASTM D 6931test specification is utilized to evaluate the IDT strength of HMA mixes.

2.9.3 Resilient Modulus Test Procedure

Both laboratories compacted specimens and field cores can be utilized for the resilient modulus test. Following are the factors that affect the resilience modulus of hot mix asphalt mixes:

- Type of testing equipment utilized (Indirect Tension by UTM, Triaxial, etc.)
- Compaction Method used (Marshall vs. Superpave Gyratory Compactor)
- Specimen geometry (Thickness and Diameter)
- Loading Waveform (Triangular or Haversine)
- Loading Duration
- Test Temperature

Load pulse configuration recommended by (ASTM D 4123, 1995) is in the form of $(1 - cos\theta)/2$ from the contact load $P_{Contact}$ to the maximum load P_{Max}, with periodic load variation.

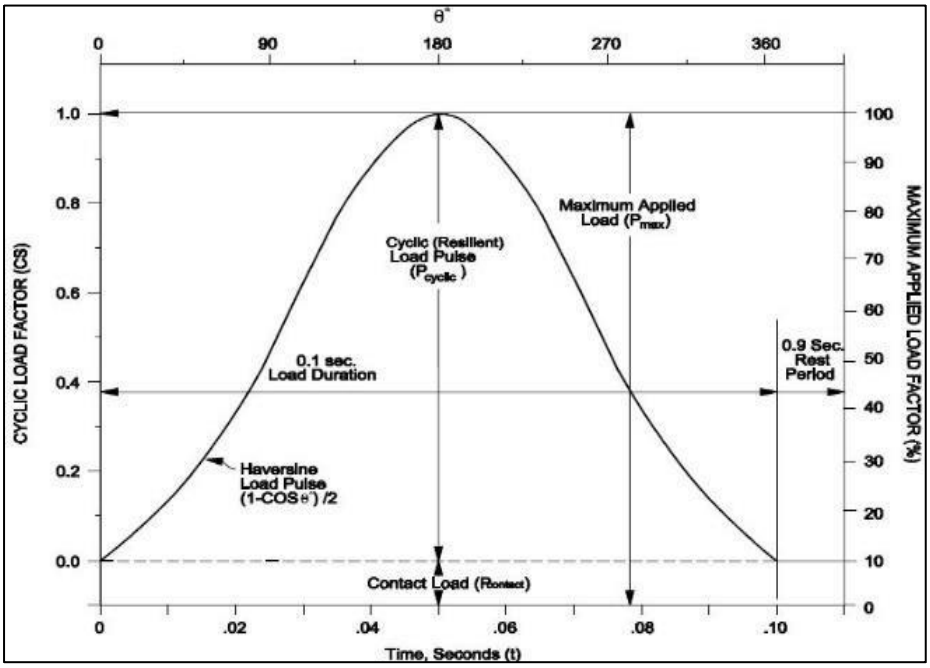


Figure 2.5 Load Pulse Representing the Haversine Loading

The test procedure consists of three steps:

- Tensile strength determination
- Specimen preconditioning with 100 repeated load cycles
- Resilient modulus determination

2.9.3.1 Tensile strength Determination

Before commencing the test for resilient modulus of the compacted mix, a baseline is to be established for initially loading the sample during the resilient modulus testing. For this purpose, in performing resilient modulus testing, load values ranging from 10% to 20%of the ITS value obtained at 25°C should be utilized for each kind of combination. The (D6931, 2011) test specification is used to determine the ITS of HMA mixes.

2.9.3.2 Specimen preconditioning

Specimen preconditioning must take place in a temperature-controlled environment. To keep the specimen in contact with the loading strip, the specimen contact load, also known as the seating load, is applied to the HMA sample vertically. For preconditioning cycles, a total of 100 to 200 load applications are required. However, the stable deformationdetermines the minimal amount of load applications for a particular scenario. It's worth mentioning that the total vertical distortion is less than 0.001 inches, which is within the required range (0.025 mm). It is not essential to test the vertical deformation if a specific value of the Poisson ratio is assumed since Poisson's ratio has a negligible impact on the resilient modulus value.

2.9.3.3 Resilient Modulus Determination

Five load pulses with almost consistent deformation will be performed after the preconditioning processes to test and record the M_R. The M_R of bituminous paving mixturesis obtained by the following equation:

$$M_R = \frac{Deviator\ Stress}{Recoverable\ Strain} = \frac{\sigma_d}{\varepsilon_r} = \frac{\sigma_1 - \sigma_3}{\varepsilon_r}$$

$Where$:

$M_R = Resilient\ modulus\ (MPa)$

$\sigma_d = Deviator\ Stress$

$\sigma_1 = Maximum\ Principle\ Stress$

$\sigma_3 = Confining\ Pressure$

$\varepsilon_r = Recoverable\ Strain$

2.10 Indirect Tensile Strength Test

Tensile strength of hot mix asphalt is particularly important since it is a reliable indicator of the likelihood of mix cracking. A particular HMA is more likely to resist cracking and permit larger strains before failure than an HMA with a low tensile strain at failure if it has a high tensile strain at failure. The indirect tensile test utilizes the same testing apparatus as the diametral repeated load test and applies a constant rate of vertical deformation until failure occurs. The test is carried out by applying a compressive load at a deformation rate of 50 mm/min at a temperature of 25 C, parallel to the vertical axial plane of a cylindrical specimen with a diameter of 4 inches or 6 inches. The loading configuration creates a constant tensile stress perpendicular to the applied load and along the vertical diametral plane. The final outcome of the IDT test is splitting of the HMA specimen. Figure 2.3 displays the stress distribution for the indirect tension test on the vertical diametral plane.

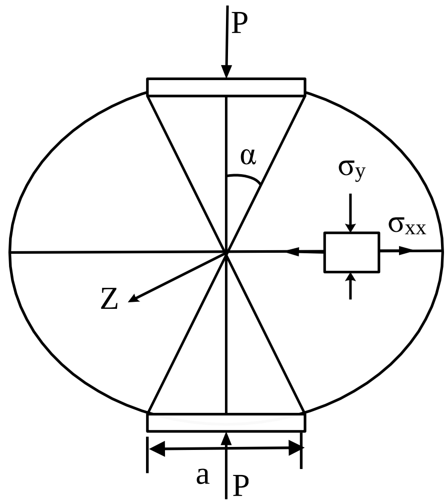


Figure 2.6 Indirect Tensile Test Schematics

2.11 Moisture Susceptibility Test

In HMA pavements, moisture sensitivity is a major source of discomfort. Moisture infiltration into the mix should not cause HMA to deteriorate significantly. HMA mixtures are prone to moisture damage because of a lack of adhesion between the aggregate and binder. Moisture-sensitive HMA mixes are those in which the binder to aggregate connection weakens when exposed to water. Pavement distress may arise if the deterioration is severe enough.

2.12 Tests for Moisture Susceptibility

A moisture susceptibility test was accomplished to find the risk of moisture damageto HMA mixes. Testing for moisture susceptibility can help predict long term stripping. The HMA moisture susceptibility can be measured using a variety of methods. The following are brief explanations of the tests performed to determine the HMA's ability to resist moisture.

2.12.1 Boiling Test (D3625M-20, 2020)

This test is used to visually observe the loss of adhesion caused by boiling water in uncompacted asphalt-coated aggregate mixes. The test is useful for determining the relative water susceptibility of bitumen-coated aggregate.

2.12.2 Static-Immersion Test (AASHTO T 182)

It is determined by immersing an HMA sample for 16 to 18 hours in water and then examining it through the water to see how much of the aggregate surface remains covered with asphalt binder. Similarly simple and subjective, this test does not require a determination of strength.

2.12.3 Lottman Test (AASHTO-T283, 2003)

Tests are performed on three different sets of compressed samples. This group is not subjected to any conditioning. Vacuum saturation with water is applied to Group 2, which reflects four years of field performance. A freeze-thaw cycle is used in conjunction with a vacuum saturation environment for Group 3, which reflects field performance lasting between four and twelve years. On each specimen, the IDT strength of the conditioned samples is compared to that of the conventional samples by a split tensile test. Typically, a TSR of 0.7 to 0.8 is used as a minimum.

2.12.4 Hamburg Wheel Tracking Test (AASHTO T 324-17, 2017)

HMA samples that have been compacted either by Marshall or SGC are examined underwater. The results provide a rough estimate of moisture susceptibility.

2.12.5 Immersion Compression (AASHTO T 165, 1996)

Instead of performing a split tensile test on the samples, a modified Lottman test employs an unconfined compressive strength test. Because of the lack of precision, samples with apparent indications of stripping might have a strength ratio close to 1.0.

2.12.6 Modified Lottman (AASHTO-T283, 2003)

Lottman test is a hybrid test that compares the split tensile strength of unconditioned materials to that of samples that have been partly soaked with water. The conditioned specimens are subjected to an optional freeze-thaw cycle in a vacuum environment during the test. Although it is anticipated that the tensile strength of water-conditioned samples would be reduced, extremely low results indicate the phenomena of moisture damage.

2.13 Hamburg Wheel Tracker Test

Rutting is one of the most prevalent pavement permanent deformations, caused by cyclic traffic loads and characterized by the accumulation of minor pavement material deformations in the form of longitudinal depressions along the wheel paths. The specimens were evaluated using a Double wheel tracker to determine their resistance to persistent deformation in order to investigate rutting propensity. The DWT is an electrically powered device that can move a steel wheel with a diameter of 203.2mm and a width of 47mm across a test specimen. The weight of the steel wheel is 158+1.0 lbs, and the average contact stress produced by the wheel contact is 0.73 MPa with a contact area of 970 mm2. Just like the influence of the rear tire of a double axel is produced by the contact pressure of the steel wheel. As the rut depth increases, the contact area expands, and the contact stresses become more varied. In a forward and backward motion, the steel wheel passes over the object. DWT steel wheel must pass the sample roughly 60 times per minute. The highest speed of the wheel over the specimen is nearly 1 ft/sec, which is achieved at the center of the sample. With the help of DWT, rutting tests can be carried out on dry, wet, and air modes. In this research, the dry mode was used to determine the susceptibility of asphalt mixtures to rutting. These three modes can be utilized by adjusting the DWT at anticipated test conditions. Figure 3.35 shows the Double wheel-tracking device used for conducting rutting tests. Before conducting the test, two 2.5-inch-thick specimens were obtained by sawing the samples from the top and bottom surfaces. These specimens were cut using the wheel tracker tray's silicone mold.

The steel tray containing the sample was stowed under the wheel and secured. The wheel tracker system was activated. The sample information was then entered into the software. The wheel's speed was set to 25 ppm (passes per minute). The number of passes was set to 10,000 (5000 cycles) as required for determining the rutting potential of asphalt mixtures, including grade 58 bitumen (ARL 60/70). The wheel tracker was used by selecting a dry mode for the determination of rut damage at 40°C temperature. Finally, the test was run, and the wheel started moving forward and backward on the mounted specimen. The number of passes was shown on the laptop connected with the machine. One complete to and fro movement of the wheel was taken as two passes. The LVDT (Linear Variable Differential Transformer) measures the impression of a rut in millimeters of the unit at the same time as the motion of the wheel. The machine automatically stopped when the required number of passes were achieved. Results were saved for further use.

Calculations for Stripping Inflection Point (SIP) are made in accordance with Iowa DOT recommendations (Schram & Williams, 2012). In the beginning, a 6-order polynomial regression is used to fit the curve. Where the polynomial's first derivative reaches a local minimum close to the test's beginning, a creep slope is inserted. When the first derivative reaches a local maximum close to the test's conclusion, a stripping slope is then put into the equation. The SIP is provided as the number of passes that correspond to the intersection of the two slopes.

2.13.1 A novel quantitative analysis of HWT test results for Moisture Susceptibility

It is generally accepted that post-compaction, visco-plastic deformation, and moisture degradation all played a part in the rutting that occurred during the HWT test (stripping) (Yildirim et al., 2007). In this study, an unique analysis method is suggested to separate these three behaviors and assess the influence of moisture damage exclusively (Lv et al., 2022).

2.14 Summary

Nano Materials, such as Nano Clay (NC), have been examined in earlier studies as a moderator in Hot mix asphalt (HMA), while SBS has proved to be effective in improving the performance of Hot mix asphalt (HMA). Based on previous studies, the properties of the modified HMA are dependent on various factors, such as the type and percentage of nanomaterial and polymers used in asphalt mixes. In this study, two modifiers are used which are styrene butadiene styrene (SBS) and Nano Clay (NC), will be utilizedin the asphalt mixture. After their addition as a modifier, the modified mixes will be subjected to different performance tests. Furthermore, performance tests used in this study,such as WTT, ITS, TSR, and M_R are also discussed.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

This chapter covers the developed methodology for achieving specified goals of this research, as stated in chapter one, which includes acquiring necessary material, sample preparation for Marshall mix design, and then using that OBC for preparation of Marshall and super pave gyratory samples for performance testing and evaluating the significance of polymers and nanomaterials such as Styrene Butadiene Styrene (SBS) and nano clay (NC) as a modifier in asphalt concrete specimens. The material characterization of binders and aggregates, as well as the specifications of different tests performed, is clarified. This study's optimum binder content (OBC) forHMA mixes was determined using the Marshall mix design approach. NHA-B class gradation is being followed in this study. This investigation utilizes SBS and NC as a modifier in asphalt concrete. SBS percentage in modified samples are 4.5% as optimum SBS content with the addition of NC in 0%, 2%, 4%, 6%, 8% in modified HMA. The percentage of SBS and NC was used to determine the weight of bitumen. The details of Performance Tests for rutting, resilient modulus; indirect tensile strength test; moisture susceptibility test; and TSR for moisture susceptibility are also discussed.

3.2 Research Methodology

The study was conducted on asphalt concrete specimens with aggregate procured from Babuzai, Katlang quarry along with gradation of NHA-B was adopted with bitumen from ARL refinery of penetration grade 60/70 is utilized in this research study. OBC from Marshall's mix design was used to make conventional and SBS, NC modified Marshall and super pave gyratory specimens. The Conventional and modified asphalt mixtures replicates were analyzed rutting, Resilient Modulus, IndirectTensile strength, and TSR for moisture susceptibility by performance tests. Testing was conducted in controlled environments as prescribed by the respective specifications. Results obtained from performance tests were analyzed, and successive conclusions and recommendations were made, as stated in the following chapters. Figure 3.1 depicts the research methodology used in this study.

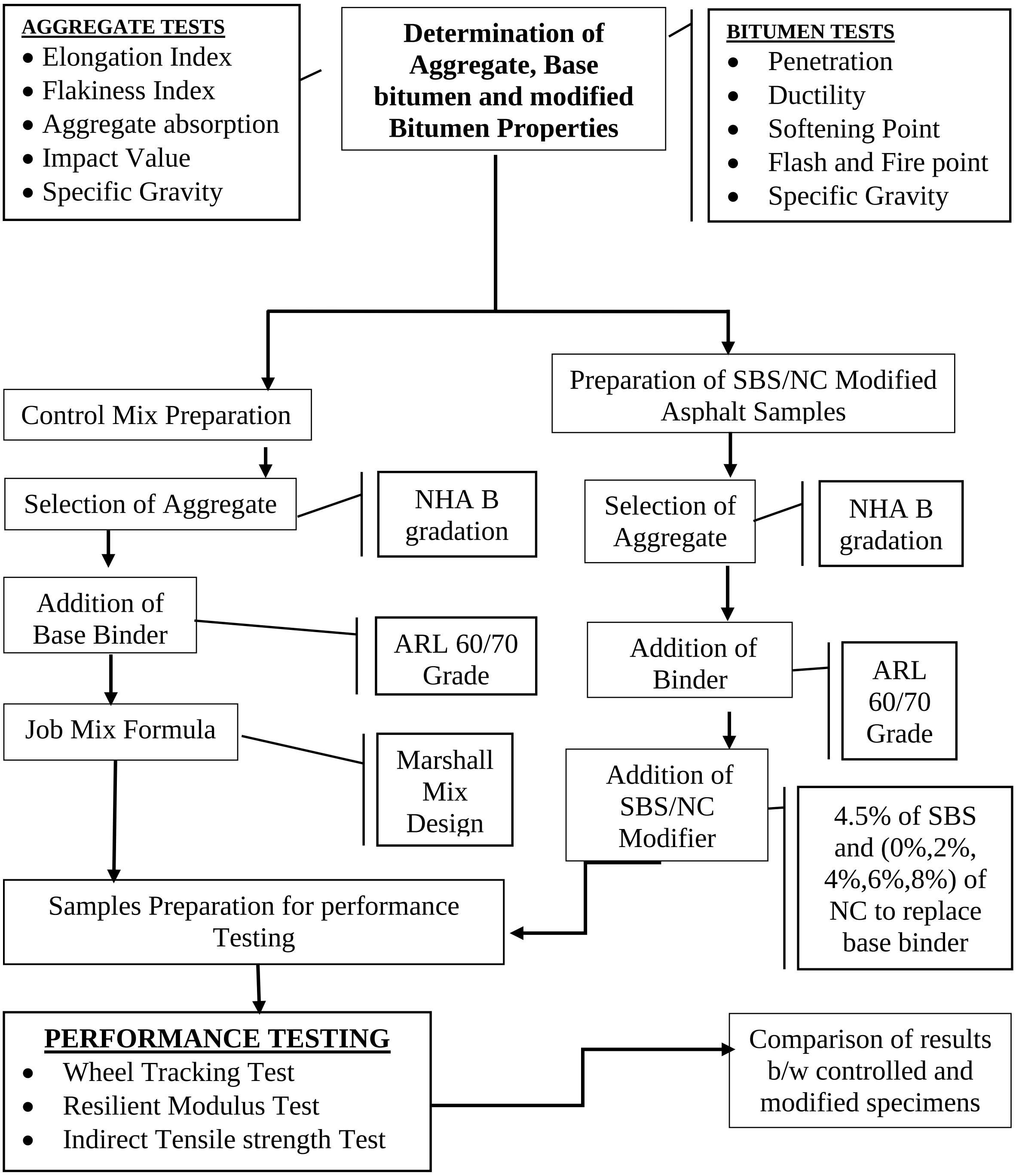


Figure 3.1 Flow Chart for Research Methodology

3.3 Characterization of Selected Material

3.3.1 Material Selection

Aggregate is procured from Babuzai, katlang, KPK quarry, and a binder of 60/70 Penetration grade from Attock Refinery Limited (ARL) for this study. A Binder of 60/70 grade has been chosen since it is commonly utilized in Pakistan's road infrastructure network and its suitability with the local climatic requirements (colder to the mild environment). Styrene Butadiene Styrene (SBS) were imported from Shijiazhuang Tuya Tech. Co., Ltd, Shijiazhuang, China. Where Nano Clay (NC) was sourced from Miz Builders, Bahria Orchard Lahore, Pakistan. Lahore.

The asphaltic mixture is the composition of aggregates, bitumen, and air. Usually, the aggregate is 95% by weight as it provides the main portion of confrontation to permanent deformation, and the remaining 5% is the weight of the bitumen. Air being weightless has no percentage in the mix. Concerning volume, the asphaltic mix is composed of 86% aggregate and 10% Bitumen, and the air occupy the remaining 4% volume. To meet the required standards of asphaltic mixtures, detailed laboratory testing of selected materials, the aggregate, and bitumen is required.

Figure 3.2 Babuzai Quarry Site

This investigation utilizes SBS and NC as a modifier in asphalt concrete. SBS percentage in modified samples is 4.5% as optimum SBS content with the addition of NC in 0%, 2%, 4%, 6%, 8% in modified HMA. The percentage of SBS and NC was utilized to the weight of bitumen to evaluate the nanomaterial's effect on HMA mixtures. The physical properties of SBS and NC are given in Table 3.1 and Table 3.2.

Different mixing methods are used to blend the nanomaterials in asphalt mixtures, such as the dry and wet methods. In this research study, the dry mix method is utilized to incorporate Nanomaterials in asphalt concrete. High shear mixer having 3000 RPM for 90 minute was used during the mixing process of polymer and nanomaterials in Asphalt, where asphalt temperature was kept above 180°C.

Table 3.1 SBS Properties

Description	Remarks
SBS type	SBS YH-791H
Structure	Linear
Place of Origin	Shijiazhuang City, China
Purity	>95%,
Appearance	White granules
Brand Name	TY
25C, 5% Styrene solution viscosity	2240 mPa.s
S/B %	30/70
Tensile Strength	20 MPa
Hardness	~76A

Table 3.2 Nano-Clay Properties

Description	Remarks
Color	Greyish Yellow
Montmorillonite content	>75%
Moisture content	Max. 10%
API water loss (cm^3)	Max. 15%
PH	9.5
Sieve analysis	99% Pass the sieve No. 200
Free Swell Index	600+ %
Liquid Limit	292%
Plastic Limit	48.55%
Shrinkage Limit	25.7%
Bentonite formula is	$Al_2H_2Na_2O_{13}Si_4$

Pictorials of SBS and NC that are utilized in this research are given in Figures 3.3 and Figure 3.4.

Figure 3.3 Nano Clay (Bentonite) Size 200µm

Figure 3.4 SBS Granules

3.3.2 Aggregate Testing

To ensure that asphaltic concrete is strong and durable, aggregate testing is a critical component of the Job Mix Formula. HMA is 92 to 96% aggregate by volume. Aggregate properties were evaluated in laboratory experiments on each stockpile. The following tests are carried out in the laboratory:

Table 3.3 Tests Conducted on Aggregates

S.No.	Tests	Standard
1	Aggregate Impact Value Test	BS 812
2	Fractured Aggregate	ASTM D5821
3	Crushing Value	BS 812
4	Los Angeles Abrasion test	ASTM C 131
5	Flakiness and Elongation Index	ASTM D4791
6	Deleterious Material Detection	ASTM C142
7	Water Absorption & Specific Gravity Test	ASTM C127 & ASTM C128

3.3.2.1 Aggregate Impact Value Test

The impact value shows the ability of aggregates to withstand traffic impact loads. There is a risk of fracture due to the traffic impact load and pounding action. Testing for aggregate impact value occurs according to the guidelines set forth in (BS812-112, 1990) and IS 383.

The impact testing equipment, a cylindrical mold with a 75mm diameter and a depthof 50mm, a tamping rod with a circular section of 10mm and a length of 230mm, and sieveswith sizes 1/2, 3/8”, and #8 were all needed for measuring impact value (2.36mm). For theImpact Testing Machine, 350g of aggregate was collected and tamped 25 times in three (3)layers.

The sample was placed in the machine's bigger mold, and 15 blows were deliveredfrom a height of 38cm with a weight of a hammer of 13.5-14kg. The aggregate was extracted and passed through filter #8. The proportion of aggregate passing through a 2.36mm sieve was used to get the impact value. Results are manifested in Table 3.4.

Figure 3.5 Aggregate Impact Value Apparatus

3.3.2.2 Aggregate Crushing Value Test

The aggregates must be strong enough to withstand traffic loads to produce a better quality and strength pavement. This test is performed under standard (BS812-112, 1990). Plunger with a 150 mm piston diameter in a steel cylinder with open ends, base plate, and a hole across it so that a rod could be inserted to raise it, cylindrical measure, balance, tamping rod, and a compressive testing machine were the tools utilized in this test. Aggregates were sieved through a series of sieves, with those passing through 12” and retaining 3/8” being chosen. The aggregate sample was washed, oven-dried, and weighed ($W1$), and three (3) covers were placed to the cylindrical measure, with each layer receiving25 tamping. The specimen was then placed in a steel cylinder with a base plate, and the plunger was then inserted. After that, it was analyzed in a compressing machine. At a rate of 4 tons per minute, the weight was added until the total weight reached 40 tons. Before being sieved at 2.36mm, crushed material was removed from the steel cylinder and separated from the material. The stuff that has passed must be gathered and weighed ($W2$). $W2/W1\ x\ 100$ was applied to determine the crushing value of aggregate. Results are mentioned in Table 3.4.

Figure 3.6 Aggregate Crushing Value Apparatus

3.3.2.3 Flakiness and Elongation Index of Aggregates

The dimensional ratios of aggregate particles of different sieve sizes are determined using the flat and elongated particle test. This characterization is used to detect aggregate that has a propensity for obstructing compaction or has trouble achieving VMA requirements owing to aggregate deterioration. Flat or elongated particles are more difficult to compress because they tend to lock up (rather than orient) more quickly during compaction. Compression fractures can also occur along their weak and narrow dimension, resulting in smaller aggregate grades and possibly lower VMA than expected. This test is performed under (ASTM 4791, 2019) guidelines.

A flaky particle is one whose average sieve size is less than 0.6 times its actual size. Elongated particles have a length of more than 1.8 sieve sizes greater than their mean sieve size. It may be carried out by following two distinct methods. For all non-Superpave applications, the first technique is used, which is identical to the original procedure designed to identify flat and elongated particles. For Superpave requirements, a second technique is used, which mostly involves comparing the maximum and lowest particle dimensions. Compacting flat and long particles is more difficult since they lock up more rapidly throughout the process. Compaction also causes aggregate particles to reorient, andthese particles have a propensity to shatter during compaction, resulting in a finer aggregate gradation, which helps to minimize Voids in Mineral Aggregates (VMA). The proportion of elongated and flat particles must be less than or equal to 15%, according to ASTM standards. The results of a test on a few aggregates are within permissible limits. Results are mentioned in Table 3.4.

Figure 3.7 Flakiness and Elongation Test Apparatus

3.3.2.4 Fractured Particles

Fractured Particles is accomplished using (ASTM D 5821, 2014). A fracture particleis an aggregate particle with the lowest number of broken faces as specified. Fractures face refers to the angular, rough surface of an aggregate particle that has been fractured due to artificial or natural crushing. This test may be used to determine the percentage of a coarse aggregate material that includes fractured particles that satisfy the requirements by countingor mass. By increasing the friction between the particles, it is necessary to give maximal shear strength to bound or unbound aggregate mixes. It also provides aggregate stability insurface treatment and increases aggregate friction on the pavement's surface. Only coarse aggregate is used in this test. A percentage of greater than 90% is the minimum requirementfor coarse aggregate to pass this test. The outcome of the coarse aggregate from the Margalla quarry was 100%, which is satisfactory. The results obtained are mentioned in Table 3.4.

3.3.2.5 Deleterious Material Test

Clay content in aggregate samples is the primary objective of this test. It is carried out as per (ASTM C 142-97, 1998) on aggregate as obtained from the Babuzai, Katlang quarry in the current study. The inclusion of a large quantity of silt and clay, or any other organic particles that may absorb water, is essential to the asphalt concrete's longevity, water tightness, and strength. Although crude, it can be used to estimate how much clay and other organic particles are in the aggregate used to manufacture asphalt mixture. The bitumen and aggregate connection may be weakened or broken because of these particles;for results, consult Table 3.4.

3.3.2.6 Los Angeles Abrasion Test

Abrasions of Los Angeles HMA aggregates must withstand disintegration, deterioration, and crushing when subjected to traffic loads. This test assesses aggregate durability and toughness. Following the usual procedure, the test is carried out by (AASHTO T96-92, n.d.). LA test verifies the aggregate's toughness and abrasion properties, i.e., its resistance to wear owing to high traffic loads. Because the aggregate inthe mix is subjected to high repetitive load levels, which causes fragmentation, deterioration, and crushing, the quality of abrasion resistance is essential to verify. The LAAbrasion machine, a weight balance, a set of sieves, and steel balls known as charge wereutilized in this test. For this process, testing methodology or grade B was used. The Los Angeles abrasion instrument was filled with 2500 g of aggregate held on 12” and 3/8” sieves, for a total of 5000 g ($W1$) of aggregate, as well as 11 steel balls or charges. It was then given a 500-revolution spin at 30–33 rpm speed. A 1.7mm sieve was used to separatethe particles. The weight of the sample that passed through it ($W2$) was recorded by $= W2/W1 \times 100$ was used to calculate the abrasion value. According to NHA standards, coarse aggregates must have an abrasion value of 30% or less. Results are mentioned in Table 3.4.

Figure 3.8 Los Angeles Abrasion Machine

3.3.2.7 Water Absorption and Specific Gravity Test

An object's specific gravity is the ratio of the weight in the air of that object to the weight in the air of that object's distilled water. The tests for specific gravity and water absorption are outlined in AASHTO T 85-91 and (ASTM C127, 2001). Due to the porous nature of the aggregates, water is absorbed by each individual particle in their pores, and this alters their density. When making asphalt paving mixes, the density of fine and coarse particles is essential. It is frequently used by engineers in the design of paving and construction projects. When determining the amount of binder absorbed and the VMA, the bulk-specific gravity is employed. Specific gravity, which represents the weight volume properties of aggregate material, is sometimes referred to as relative density. It's a material's mass to volume ratio at a constant temperature. Fine aggregates are coarse aggregates that pass-through filter No. 4 but do not pass-through sieve # 4. Separately, the specific gravities of course and fine aggregate were determined.

3.3.2.8 Specific Gravity of Coarse Aggregates

(ASTM C127, 2001) is utilized to assess the specific gravity and water absorption of coarse aggregate. Passing sieve #4, the aggregates were then baked in an oven and soaked in water for 24 hours to remove the aggregates that remained on sieve #4. After that, the aggregates were rolled in a cloth, and their saturated weight was measured. After this, the submerged weight of aggregates was determined, and their specific gravity and water absorption were calculated. Unlike the oven-dried sample, the aggregate pores are filled with water in the saturated surface dry condition.

3.3.2.9 Specific Gravity of Fine Aggregates

Fine aggregates, like coarse aggregates, are permeable to water and can be used to provide a porous surface. A standard procedure was used for this test, such as AASHTO T 84-93 and (ASTM C128, 2008). Aggregates that passed sieve #4 were soaked in water for around 24 hours, and then aggregates were then sprayed in a tray to dry to the point where they were saturated on the surface. The cone was put on a flat surface, filled with fine aggregate, then compressed with twenty-five (25) strikes with a tamping rod. The aggregates were seen when the cone was removed. If the particles had the form of the mold,they were not SSD. The same method was used after drying the aggregate again till the aggregate was somewhat slumped with the cone removal. After filling a pycnometer to a certain level with the water, it was weighed. After saturated surface drying, sand was placedin the flask and weighed again. After oven drying sand at a temperature of 110 ^{o}C, the specific gravity and absorption were determined. Table 3.4 summarizes the test results conducted on the aggregates.

Table 3.4 Aggregates Tests Results

Type of Test		**Results %**	**Specification**	**Standards**
Fractured Particles		99%	90% (Min)	ASTM D 5821
Los Angeles Abrasion		28%	30% (Max)	ASTM C 131
Flakiness Index of Aggregates		9.3%	10% (Max)	ASTM D 4791
Elongation Index of Aggregate		3.7%	10% (Max.)	ASTM D 4791
Impact Value of Aggregate		17.23%	30% (Max.)	BS 812
Crushing Value of Aggregate		20.52%	30% (Max.)	BS 812
Water Absorption	Fine Aggregate	2.55%	3% (Max.)	ASTM C 128
	Coarse Aggregate	0.81%	3% (Max.)	ASTM C 127
Specific Gravity	Fine Aggregate	2.628%	-	ASTM C 128
	Coarse Aggregate	2.632%	-	ASTM C 127
Clay Percentage	Coarse Aggregate	0.562%	-	ASTM C-142
	Fine Aggregate	2.812%	-	ASTM C-142

3.3.3 Binder Testing

Consistency, safety, and cleanliness are the three most essential characteristics of a binder in infrastructure and engineering applications, according to the AI MS 4 guidebook. The density of the asphalt binder changes as the temperature rises. Therefore, evaluating asphalt binder consistency requires a standard temperature. To evaluate the consistency of bitumen binder, a penetration test is commonly employed (Asphalt Institute MS-4, 1988). while softening point test and ductility provide further information and assurance about its consistency. To characterize the asphalt binder, the following tests were carried out in the laboratory.

Table 3.5 Tests Conducted on Bitumen

S. No.	Test	Standard
1	Flash & Fire Point	ASTM D 92
2	Penetration Test	ASTM D 5
3	Softening Point	ASTM D 36
4	Ductility	ASTM D 113
5	Specific Gravity	ASTM D70

3.3.3.1 Flash and Fire Point

Flash and fire points are critical to ensuring that the safety criteria for the job site are met within the specified range. The test is executed as per the (ASTM D92, 2005). The temperature at which the fumes of a bitumen sample in Cleveland Open Cup abruptly flare in the occurrence of an open flame is known as the binder's flashpoint. The temperature at which the surface of the binder catches fire and produces flames for at least five seconds is known as the fire point. Bitumen was poured into a metal cup until it reached a specific volume. After that, it was heated at a steady pace while a test flare was passed over it at certain intervals. The temperature at which the flash and fire erupted was recorded once the criteria were met. Three separate tests were conducted to determine these temperatures for each binder. The flashpoint should always be higher than 232 ^{o}C, according to the standards.

Figure 3.9 Flash and Fire Point of Bitumen Apparatus

3.3.3.2 Penetration Test

One of the first methods of determining the quality of an asphalt binder and its consistency is the penetration test. Since 1959, it has been used to evaluate the quality of binders. The test method was accomplished in accordance with the (ASTM D5, 2008) and (AASHTO T 49-93, 2019). It is one of the earliest tests for determining the consistency of asphalt binders. It determines the softness and hardness of a binder to categorize it into several standard classes. A soft and thin binder has a higher penetration value. Binder witha low penetration value is preferred in hot areas, while a binder with a high penetration value is preferred in cooler climates. To begin, the binder is heated to a sufficienttemperature for it to flow and not trap any air, but it should not be heated too much since this will affect the binder's characteristics. A temperature-controlled water bath is then usedto keep the binder at a consistent temperature of 25 °C. After the container has achieved the required temperature, it is removed and tested in a penetrometer by passing a 100g loadthrough a needle for 5 seconds. Two samples of each bitumen were tested for penetration, with penetration values are taken at five points on each specimen.

Figure 3.10 Bitumen Penetration Test Apparatus

3.3.3.3 Softening Point

This test is run under (ASTM D36, 2006) and (AASHTO T 53-92, 2008). Although it is a viscoelastic substance, the bitumen softens with increasing temperature, and its viscosity lowers. The temperature at which a 3.5g steel ball can no longer be maintained by a sample of asphalt binder when it is submerged in water. As a result, it is the average temperature at which two bitumen discs become sufficiently soft to enable steelballs to fall 25mm. First, the binder was heated to a temperature that allowed it to flow while maintaining its characteristics. Then it was pressed into horizontal discs using a mold.The balls were put on the discs after being placed in the device. The temperature was raised ul the binder enabled the balls to fall through the distance stated above the discs after being placed in the device.

Figure 3.11 Bitumen Softening Point Apparatus

3.3.3.4 Ductility Test of Bitumen

The ductility test is run under (ASTM D 113, 1999) and (AASHTO T51-93, 2017). The asphalt binder's ductility is measured by the length of the binder's thread when stretched under standard test conditions, expressed in centimeters. This means that asphaltswith low ductility are often believed to have weak adhesive characteristics and consequently poor performance, whereas asphalts with high ductility are more susceptible to temperature changes.

The stretching and adhesion properties of the binder are assessed in this test.Bitumen's ductility is regarded as a key and significant physical characteristic. It depicts the behavior of bitumen as temperature changes. The experiment was conducted at a temperature of 25 °C. Binder specimens are tested for ductility by pulling them apart at 5cm/min and 25 °C with a standard-sized binder specimen (a briquette with a 1 in 2 cross- sectional area), the distance it lengthens without breaking is called ductility. The specimenmust be at least 100cm long to pass the ductility test. Under high and frequent traffic pressures, asphalt mixes are made from less ductile bitumen fracture.

Figure 3.12 Ductility Test of Bitumen Apparatus

Table 3.6 Virgin Bitumen Tests Results

Test Description	Result	Specification	Standard
Penetration Test @ 25°C	66	60/70	ASTM 5
Flash Point Test (°C)	262°C	232°C (Min)	ASTM D 92
Fire Point Test (°C)	291°C	270°C (Min)	ASTM D 92
Specific Gravity Test	1.03	1.01-1.06	ASTM D 70
Softening Point Test (°C)	49.1°C	49°C -56°C	ASTM D36-06
Ductility Test (cm)	108	100 (Min)	ASTM D 113-99

3.4 Gradation Selection

NHA class B aggregates were used in dense-graded surface course mixes in line withNHA (1998) requirements. Marshall Mix Design stated the maximum aggregate size for NHA class B wearing coarse gradation to be 19 mm, which was slightly smaller than that.

Table 3.7 Gradation selection for performance testing

Sieve Designation		NHA-B Specification Range (% Passing)	Our Selection	% Retained
mm	**inch**			
19	**3/4**	100	100	0
12.5	**1/2**	75-90	82.5	17.5
9.5	**3/8**	60-80	70	12.5
4.75	**#4**	40-60	50	20
2.38	**#8**	20-40	30	20
1.18	**#16**	5-15	10	20
0.075	**#200**	3-8	5.5	4.5
Pan	**Pan**	. . .	. . .	5.5

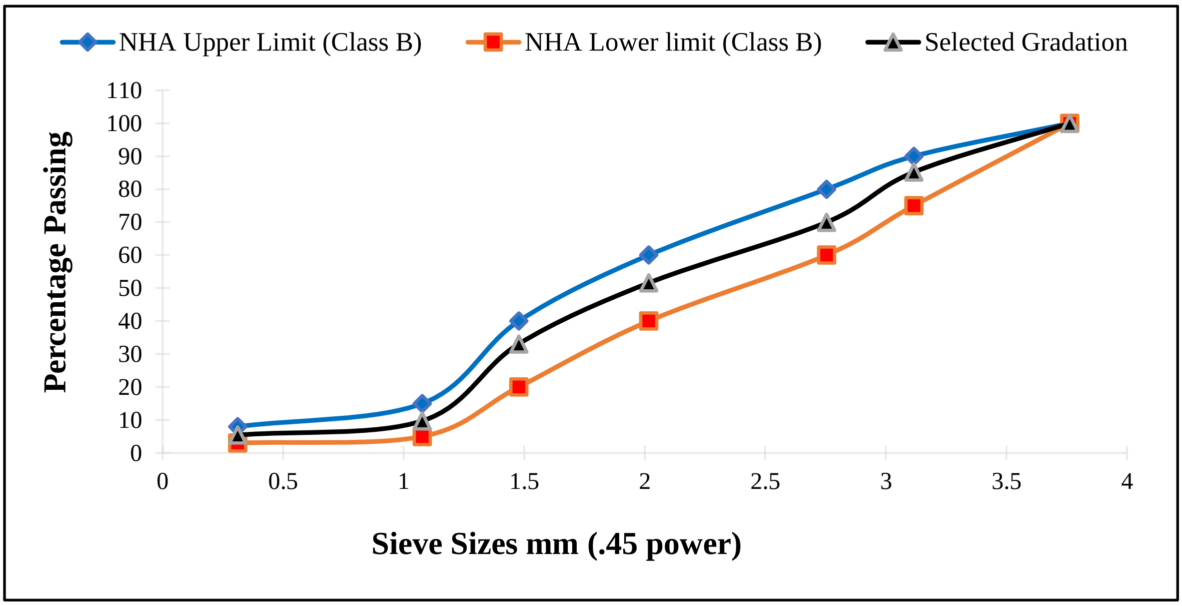


Figure 3.13 NHA Class-B Gradation Plot

3.5 Asphalt Mixture Preparation

Because the pavement built with the optimum combination of aggregate and binder will have excellent performance and a long-life span, the fundamental idea for designing asphalt mixes is the optimal combination of aggregate and binder. Because the aggregate structure is essential in preventing deformation, mix design should include a mix that can resist densification under traffic stress while causing minimal changes in air voids after construction.

Five different binder contents were used to produce specimens (3.5, 4.0, 4.5, 5.0, and 5.5 percent). The objective for the five trial blends was to find the mix that performs optimally at a minimal bitumen dosage of 4% void content. The bituminous mixes used to determine OBC were made according to (ASTM D 6926-10, 2010a), the industry standard for bituminous sample preparation using the Marshall Apparatus. The volumetric properties, stability, and flow were assessed, the Marshall Mix design criteria were verified,and the OBC was computed at the end. The following procedure was adopted for Marshall samples preparation.

3.5.1 Aggregate and Bitumen Preparation

To begin, the collected aggregates were sieved through a series of sieves shown on the gradation table and put in separate buckets. These were dried to consistent weights at 105°C to 110°C after sieve examination. We dried aggregates for several days at 105°C to 110 degrees Celsius after sieve analysis to attain a constant weight. If the Marshall Mix design approach is used, 1200-gram aggregates are required to compact a 4-inch diameter sample using the Marshall Mix design technique (ASTM D 6926-10, 2010a). The followingequations were utilized to calculate the amount of asphalt required for each specimen:

$$M_T = M_A + M_B$$

$$M_B = \frac{X}{100} \times M_T$$

$Where$:

$M_T = Total\ Mix\ mass\ in\ gram$

$M_A = Aggregate\ mass\ in\ gram$

$M_B = Bitumen\ mass\ in\ gram$ and $X = Bitumen\ in\ percentage$ (%)

Table 3.8 Weight of Aggregates and filler used in samples preparation

Sieve Size (mm)	Sample weight (gm)
19	0
12.5	210
9.5	150
4.75	240
2.38	240
1.18	240
0.075	54
Filler	66
Total	**1200**

Table 3.9 Weight of Bitumen required for each percentage

Bitumen (%)	Weight (gm)
3.5	42
4	48
4.5	54
5	60
5.5	66

The weight of aggregates and filler required for each specimen according to Marshall Mix design criteria (ASTM D 6926-10, 2010b) is provided in table 3.8 above**.** Table 3.9 shows the amount of bitumen required for each specimen by Marshal Mix design.

3.5.2 Mixing of Aggregate and Bitumen

The mechanical mixer is recommended by (ASTM D 6926-10, 2010b) for the appropriate mixing of bitumen and aggregates. After extracting the dried, heated aggregates and heated bitumen from the oven, they were immediately transferred to the mechanical mixing equipment. The temperature range for mixing was 160°C to 165°C, which corresponds to the temperature in Pakistan when bituminous mixes are produced (NHA Specifications). Furthermore, the binder viscosity range of 0.22 - 0.45 Pa.sec indicated by the Superpave mix design matches this mixing temperature (SP-2).

Figure 3.14 Mixer for Preparation Asphalt Mixture

3.5.3 Conditioning and Compaction of Asphalt Mixture

The (ASTM D 6926-10, 2010b) guideline suggests that the asphalt mixture be conditioned for two hours before compaction. As an outcome, after mixing, the bituminous mix was transferred to a metal pan and heated at 135°C for compaction. The mix was compacted at 135°C using an Automatic Marshall Compactor after two hours of conditioning. Mold assembly includes the cylinder, base plate, and extension collar. The cylinder is 3 inches tall with a 4-inch interior diameter. Both ends of the mold may be swapped out for the collar. A piece of filter paper was put in the mold assembly after it was properly cleaned and heated to a temperature between 95°C - 150°C.

The mixture was then scooped and spatulated into the mold, which was then filled after a piece of filter paper was put over it. Compaction pedestals were then used to hold the mold assembly. On the mold, the hammer was correctly positioned. For this study, the design requirements for a dense-graded wearing course were ESAL's 30 (millions) or a highly loaded pavement. To mimic heavy traffic, 75 blows were delivered mechanically to the sample's face for compaction purposes. After the blows were finished, the mold assembly was removed, the specimen was inverted, the mold was rebuilt, and the same number of blows were delivered on the specimen's opposite face.

Figure 3.15 Marshall Compactor

3.5.4 Extraction of Marshall Specimen

The assembly was removed after both sides were compacted, and the sample was allowed to cool to a reasonable temperature before being removed. An extraction jack was then used to remove the specimen from the mold. These removed specimens were laid out on a level surface to cool up to room temperature. These samples were made with 0.5 percent increments of bitumen content to identify the best performing combination with the least amount of binder and 4 percent air voids on which the OBC for asphalt mixture is established.

3.5.5 Number of Specimen Replicates for Each Job Mix Formula

Three specimens were created for every asphalt binder percentage and combination of aggregates. The gradation adopted for the specimen was NHA-B. Five different binder ingredients were used to produce specimens (3.5, 4.0, 4.5, 5.0, and 5.5 percent). Five experimental blends were utilized to establish the combination that works optimally at a minimum bitumen concentration of 4% air voids.

Figure 3.16 Compacted Marshall Specimens

3.6 Diagnosis of Stability, Flow, and Volumetrics

Following the measurement of theoretical maximum specific gravity (Gmm) and bulk specific gravity Gmb, the volumetric characteristics of the mixes, including Voids in Mineral Aggregates (VMA), Voids Filled with Asphalt (VFA), Air Voids (Va), and unit weight, were determined using their respective formulae. (ASTM D 2041, 2011)and (ASTM 2726, 2000) were used to determine the Gmm and Gmb of bituminous pavement mixes. The samples were maintained in a water bath for 1 hour at 60°C after Gmb determination and then evaluated for stability and flow using Marshall Test equipment.

3.6.1 Bulk Specific Gravity

Following cooling to room temperature, the bulk specific gravity of the samples was measured according to (ASTM D1188, 2000). The specimen was first weighted dry, then submerged in water for a while till the voids were filled with water, and then weighted again. Samples were taken from the water and dried with towels; their weights were recorded as a result of this drying process. The bulk specific gravities of each sample of thecombination were deduced after the test was completed.

Figure 3.17 Bulk Specific Gravity of Marshall Specimens

3.6.2 Marshall Stability and Flow

Marshall samples can endure maximum stress at 60°C, referred to as "stability." As a non-destructive bulk specific gravity test was used, materials were heated to 60°C for 30 to 40 minutes before testing. Samples were removed from the water bath and loaded into the Marshall testing machine at a rate of 50.8 mm/min until they achieved their maximum load capacity. The number is recorded now when the load begins to drop and isreferred to as Marshall Stability. A displacement gauge is connected to the sample frame before the test, and the deformation in the vertical direction is recorded in increments of 0.25 mm. The deformation at maximum load is measured and referred to as the flow value.The resistance to shear and rutting is influenced by the friction and cohesion between particles in the asphalt mixture. This test was accomplished in compliance with (ASTM D6927-15, 2010).

Figure 3.18 Marshall Test Machine

3.6.3 Maximum Theoretical Specific Gravity

Maximum theoretical specific gravity (Gmm) refers to the combined specific gravity of aggregate and bitumen in asphaltic blends when air spaces are removed. Air voids are estimated with the aid of Gmm, which is larger than or equal to Gmb, and is oneof the most crucial attributes of asphalt mixes. The Superpave mix design Gmm is utilizedto detect air voids in the field. (ASTM D 2041, 2011) and (AASHTO T 209, 2022) were used to conduct this test. The laboratory developed loose mix sample was initially weightedin dry condition. It was then put in a vacuum container filled with water. To remove the entrapped air, a vacuum of 25–27 mm of Mercury was given to the pycnometer. An agitatorwas used to agitate the pycnometer. After the agitation, the weight was measured. Using this information, the Gmm of the sample was calculated as the sample mass divided by the volume of water it occupied.

Figure 3.19 Maximum Theoretical Specific Gravity Machine

3.6.4 Air Voids in Asphalt Mixture

The air voids in the compacted mixtures are the small air spaces volume between the coated aggregates. Air voids is the percent of the compacted mixture′s bulk volume (Gmb) in relation to its maximum specific gravity (Gmm). The quantity of air spaces in a mixtureis critical and directly linked to stability and durability.

$$V_a = 100 \times \frac{G_{mm} - G_{m_b}}{G_{mm}}$$

$Where$:

$Va = \ Air\ voids$ (%)

$Gmm = \ Maximum\ theoretical\ specific\ gravity$

$Gmb = \ Compacted\ mix\ bulk\ specific\ gravity$

3.6.5 Voids Filled with Asphalt

The portion of the mineral aggregates' voids containing asphalt is known as VFA. When the number of air spaces decreases, VFA rises in inverse proportion. The asphalt-filled voids only include the bitumen-filled portion of the empty space between the aggregates, not the air or absorbed bitumen in the aggregates. The formula used to calculate these void percentages is as follows:

$$VFA = \frac{VMA - Va}{VMA} \times 100$$

$Where$:

$VFA = \ Voids\ filled\ with\ asphalt$ (%)

$VMA = \ Voids\ in\ mineral\ aggregate$

$Va = \ Air\ voids$ (%)

3.6.6 Voids in Mineral Aggregate

Voids in Mineral Aggregates refers to the amount of intergranular space between the aggregate particles in the compacted mix, including air voids and the effective asphalt content (VMA). The bulk specific gravity of samples is used to determine the percentage VMA expressed in relation to the bulk volume:

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

$Where$:

$VMA = Voids\ in\ mineral\ aggregate$

$Ps =\ \%age\ of\ aggregate\ by\ total\ weight\ of\ mix$

$Gsb =\ Aggregate's\ bulk\ specific\ gravity$

$Gmb =\ Compacted\ mixture's\ bulk\ specific\ gravity$

3.7 Marshall Specimen Volumetrics Results

The volumetric properties, stability, and flow of this mix are shown in the table below:

The curves connecting asphalt content and volumetric properties, stability, and flow wereconstructed according to the MS-2 manual to estimate the OBC of Asphalt mixtures.

Table 3.10 Volumetric properties of Marshall samples

%AC	Gmb	Gmm	Va(%)	Vb (%)	VMA(%)	VFA(%)	Stability (KN)	Flow (mm)
4	2.356	2.491	5.41	9.17	14.00	61.40	10.31	2.45
4.5	2.390	2.472	3.34	10.34	13.23	74.70	9.88	2.71
5	2.396	2.454	2.34	11.42	13.46	82.40	8.108	2.91
5.5	2.398	2.428	2.01	12.54	14.35	82.67	12.53	3.16

Table 3.11 Volumetric properties at Optimum Binder Content (OBC)

Marshall Parameters	Measured Value	Criteria	Remarks
OBC (%)	**4.3**	At 4% Air Void	----
Unit Weight (g/cm3)	2.36	NA	----
VMA (%)	13.6	13 (Min)	Pass
VFA (%)	70	65-75	Pass
Stability (KN)	15.4	8.006 (Min)	Pass
Flow (mm)	2.6	2.0-3.5	Pass

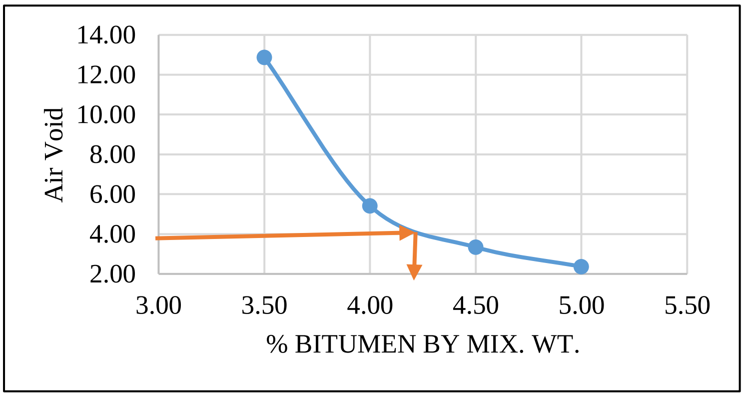


Figure 3.20 Air Voids vs. Bitumen Content

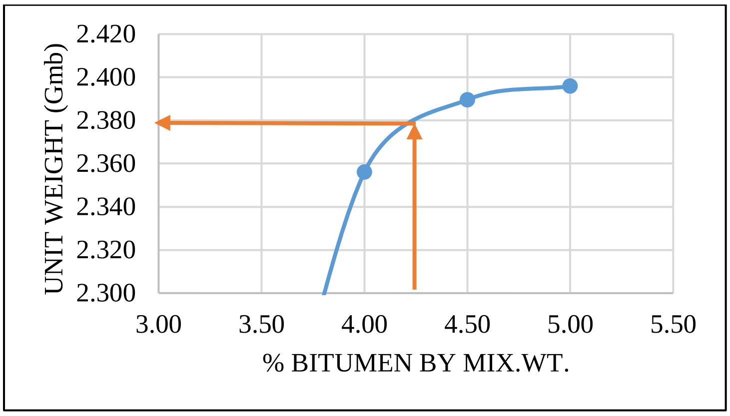


Figure 3.21 Unit Weight vs. Bitumen Content

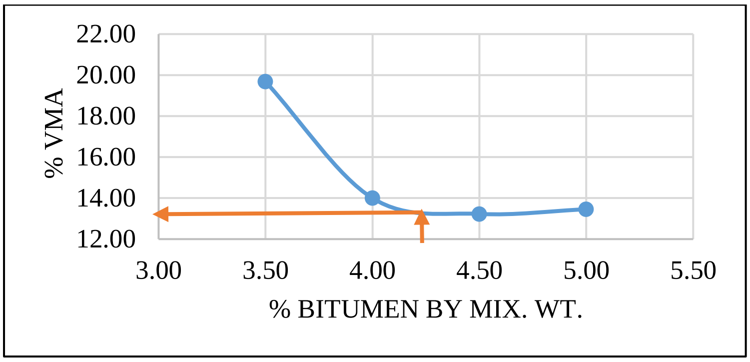


Figure 3.22 %VMA vs. Bitumen Content

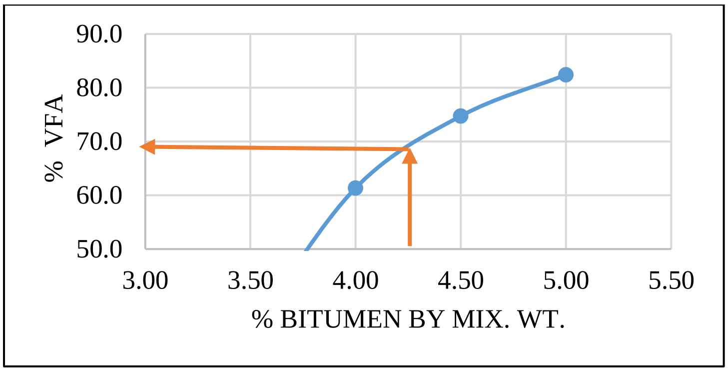


Figure 3.23 %VFA vs. Bitumen Content

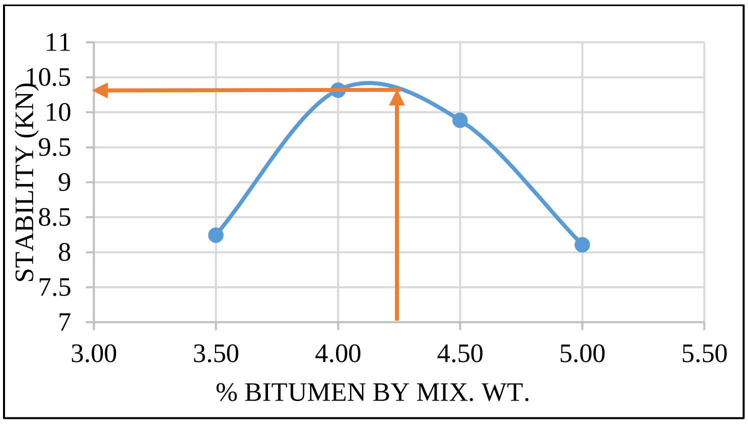


Figure 3.24 Stability vs. Bitumen Content

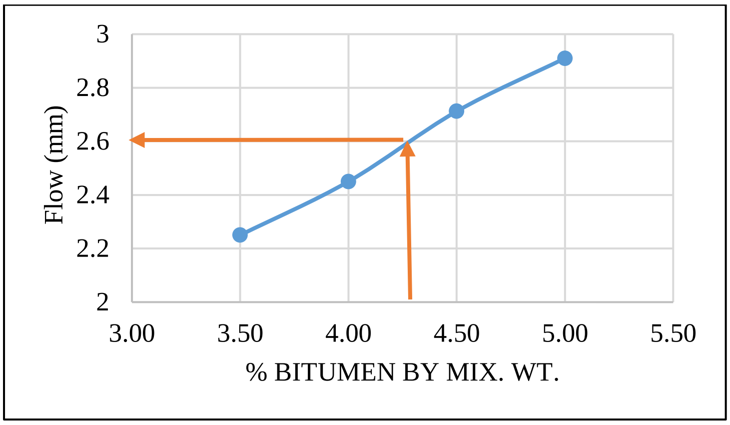


Figure 3.25 Flow vs. Bitumen Content

3.8 Super Pave Gyratory Samples Preparation for Performance testing

For Nano Materials such as CNT and NC modified HMA samples, heated aggregates, and modified bitumen are mixed for 3 minutes by the mixer. Modified bitumen consists of SBS 4.5%. Further, Modified bitumen was prepared using Optimum SBS 4.5% in addition to NC at 2%, 4%, 6%, and 8%.

3.8.1 Laboratory-Preparation of HMA gyratory sample

For the preparation of the laboratory gyratory, the HMA mixture using 7300 grams of the mix was taken for conventional and modified samples of asphaltic mixtures. Mixing aggregate and asphalt with a mechanical mixing machine has been suggested by (ASTM D 6925, 2003) and (AASHTO T312, 2015) at 160°C and 170°C to match Pakistan's paving mix production temperatures (NHA Specifications).

Total weight of Gyratory sample (gm) =7300gm

Aggregate (95.7%) = 6986gm

Binder (4.3%) = 314gm

Table 3.12 Gyratory Samples Preparation

Sieve Designation		NHA-B Specification Range (% Passing)	Our Selection (% Passing)	% Retained	Retained Wt (gm)
mm	**inch**				
19	**3/4**	100	100	0	0
12.5	**1/2**	75-90	82.5	17.5	1222.5
9.5	**3/8**	60-80	70	12.5	873.2
4.75	**#4**	40-60	50	20	1397.2
2.38	**#8**	20-40	30	20	1397.2
1.18	**#16**	5-15	10	20	1397.2
0.075	**#200**	3-8	5.5	4.5	314.3
Pan	**Pan**	. . .	. . .	5.5	384.4

3.8.2 Mixture Conditioning

Conditioning asphalt mixture for roughly two hours before compaction is recommended by (AASHTO R 30-02, 2019). It was then placed in a metal container in an oven that had been heated to the temperature of the mixture's compaction plus three degrees Celsius. After 60 + 5 minutes, the mixture was mixed by hand to guarantee consistency.

3.8.3 Compaction of Specimens

Compacting at 135°C with the Superpave Gyratory Compactor was completed after the prepared mix had been conditioned. Afterward, the mix was put into the 6-inch- diameter mold, which had been prepared to 100 °C. The mold was immediately transferredto the Superpave Gyrator Compactor (Figure 3.26).

Figure 3.26 Superpave Gyratory Compactor

Figure 3.27 shows Superpave Gyratory Compactors compacted specimens. Toaccount for the high traffic criteria of design EASLs > 30 million, the laboratory-designedsamples were compressed to 125 gyrations (N Design). Gyratory compacted samples had a 150 mm diameter and were roughly 170 mm in height. The gyratory compacted was further cored from the center using a portable coring machine. For the required height-to-diameter ratio of 1.5, the specimen was reduced to 150 mm in height using a saw cutterto meet the specifications of the dynamic modulus test specimen specified in (AASHTO-TP 62-07, 2009). The saw cut specimen was examined for its good parallel diameter sidesand necessary dimensions, among other standards.

3.8.4 Cutting of Asphalt Mixture

To meet the AASHTO T-24 (2011), specimens were saw cut to 6” in diameter, and the height of 150 mm. Figures 3.27 show specimens being saw-cut.

Figure 3.27 Saw cutted specimen in silicone mold of HWTT

3.9 Tensile Strength Ratio (TSR) for Moisture Susceptibility

The moisture susceptibility test was conducted as specified by (ASTM D 6931-12, 2007). Three specimens per mix were tested unconditioned. To condition these unconditioned samples, they were placed in a water bath set at 60°C for one hour before testing. Conditioned specimens were evaluated on another set of three specimens per mix. Conditioning of samples was conducted in compliance with ALDOT-361. A 24-hour 60°C water bath was used, followed by an hour in a 25°C bath, to soak the specimens. Both unconditioned and conditioned samples were loaded diametrically at a rate of $50mm/min$. Tensile strength was estimated for each specimen using the specimenmeasurements and failure load. The average conditioned tensile strength was then determined by the average unconditioned tensile strength to get the tensile strength ratios.

Typically, the minimum permissible value for the TSR is between 0.7 and 0.8. The equation employed to estimate the tensile strength of every subgroup is following:

$$\text{Tensile Strength, } s_t = \frac{2000P}{\pi Dt} \text{ (in KPa)}$$

$Where$:

$P = \ Max\ load\ (in\ N)$

$t\ and\ D = \ Height\ and\ Diameter\ of\ specimen\ (mm)$

Moisture damage is indicated by the TSR value. When the conditioned subset is divided by the unconditioned subset, a tensile strength ratio is calculated. All combinations can be figured out using the formula.

$$TSR = \frac{\mathrm{S}_2\ : Average\ Tensile\ Strength\ of\ Conditioned\ Samples}{\mathrm{S}_1 : Average\ Tensile\ Strength\ of\ Unconditioned\ Samples}$$

3.10 Indirect Tensile Strength Test

This test is normally conducted in UTM in conformity with ASTM D6391 standards. Two salient features of the asphalt mixture can be determined by conducting this test. Moisture susceptibility of asphalt is typically assessed by comparing the sample's indirect tensile strength before and after conditioning in water to a standard test sample. The cracking potential of HMA can also be assessed by determining the tensile strain at failure.It is more likely that an HMA with a high tensile strain till failure is more resistant to cracking. During this test, a cylindrical sample is compressed along the vertical diametricplane. Loading strips of 0.5 inches wide are utilized for 4-inch diameter samples with 2.5-inch heights to ensure a uniform load distribution in a direction perpendicular to the load. For a 4-inch diameter sample at 25°C, a deformation rate of 50 mm/min is prescribed, and for a 6-inch diameter sample, the applied deformation rate is 76.2 mm/min. This test measures the tensile strength of HMA mixtures, which affects the cracking behavior of these materials.

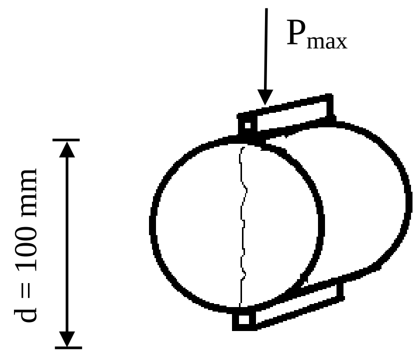


Figure 3.28 Tensile Strength Ratio Schematic Diagram

3.11 Resilient Modulus Test (MR)

The resilient modulus test is carried out after the sample's IDT has been find using the Indirect tensile strength test (ITS) and 5 to 20% of the sample's IDT is used as a peak loading force as an input parameter during the test because this will make the deformationalmost recoverable.

This information can also be used as a substantial input for the process of mechanistic-empirical pavement design, as well. In the context of cyclic loading of a sample, the resilient modulus of a sample is defined as the relationship between applied stress and recovered strain observed during the loading cycle. Additionally, the resilient modulus is a preliminary test that can be used to determine the relative quality of the materials and provide information for pavement design, as well as for evaluation and analysis purposes, in addition to other applications. It is necessary to compare variations inmaterial stiffness as a function of polymer content and temperature to determine the robustmodules. The robust modulus is a critical statistic for anticipating pavement performance and measuring the response of pavements to traffic stress, according to experts. Permanentdeformation has been shown to be more resistant to firmer pavements than temporary deformation. It is vital to remember that at low temperatures, mixes with a high rigidity (higher M_R) break more quickly than combinations with a low rigidity (lower M_R). The test samples must be placed in a temperature-controlled cabinet and brought to the proper testing temperature before the robust modulus test can be carried out successfully. In the following hours, they are placed in an environmental room for a total of at least twelve hours. To ensure that the samples achieved the proper test temperature assoon as possible, they were placed into the loading assembly at two different temperatures: 25 degrees Celsius and 40 degrees Celsius, depending on the application. A cylindrical specimen's resilient modulus must be determined by performing an indirect tension testwith a repeating load over a period.

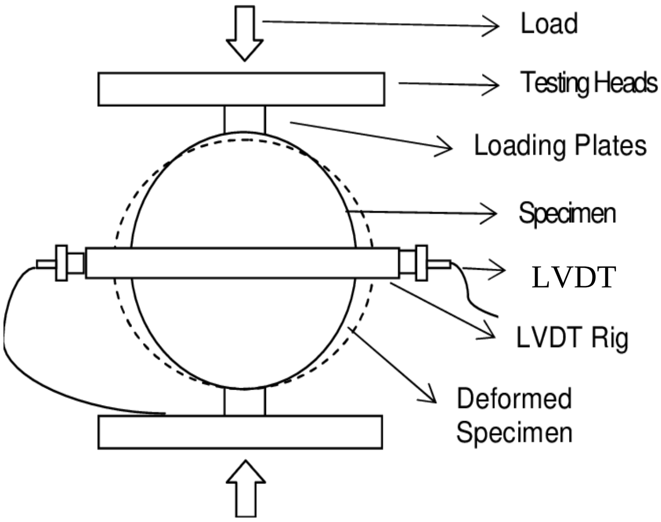


Figure 3.29 Schematic of Resilient Modulus Test

In the vertical diametric plane of the specimen, a haversine waveform is given vertically. The horizontal elastic deformation was used to determine the application of the load and the value of the resilient modulus. To precondition the specimen, it is necessary to subject it to a minimum of 50 to 200 cycles of stress. The modulus of the test machine is determined by the software program that runs on the machine during each load stroke. Also included were results from the average test findings, which were expressed as the specimen's robust modulus at that temperature. The resilient modulus is computed using equations by calculating the actual load, horizontal deformation, and recovered horizontaldeformation for each load pulse and then multiplying these values together.

$$\text{Resilient Modulus, } MR = \frac{P\ (0.27+\mu)}{(\Delta h)\ t}$$

$Where$:

$P = Dynamic\ Load$

$T\ = Specimen\ Thickness$

$\Delta h = Horizontal\ Recoverable\ Deformation$

$u =\ Poisson\ Ratio$

3.12 Hamburg Wheel Tracking Test

Rutting is one of the most prevalent pavement permanent deformations, caused by cyclic traffic loads and characterized by the accumulation of minor pavement material deformations in the form of longitudinal depressions along the wheel paths. The specimens were evaluated using a Double wheel tracker to determine their resistance to persistent deformation in order to investigate rutting propensity. The DWT is an electrically powered device that can move a steel wheel with a diameter of 203.2mm and a width of 47mm across a test specimen. The weight of the steel wheel is 1581.0 lbs, and the average contact stress produced by the wheel contact is 0.73 MPa with a contact area of 970 mm2. Just like the influence of the rear tire of a double axle is produced by the contact pressure of the steel wheel. As the rut depth increases, the contact area expands, and the contact stresses become more varied. In a forward and backward motion, the steel wheel passes over the object. DWT steel wheel must pass the sample roughly 60 times per minute. The highest speed of the wheel over the specimen is nearly 1 ft/sec, which is achieved at the center of the sample. With the help of DWT, rutting tests can be carried out on dry, wet, and air modes. In this research, the dry mode was used to determine the susceptibility of asphalt mixtures to rutting. These three modes can be utilized by adjusting the DWT at anticipated test conditions. Figure 3.35 shows the Double wheel-tracking device used for conducting rutting tests. Before conducting the test, two 2.5-inch-thick specimens were obtained by sawing the samples from the top and bottom surfaces. These specimens were cut using the wheel tracker tray's silicone mold.

The steel tray containing the sample was stowed under the wheel and secured. The wheel tracker system was activated. The sample information was then entered into the software. The wheel's speed was set to 25 ppm (passes per minute). The number of passes was set to 10,000 (5000 cycles) as required for determining the rutting potential of asphalt mixtures, including grade 58 bitumen (ARL 60/70). The wheel tracker was used by selecting a dry mode for the determination of rut damage at 40°C temperature. Finally, the test was run, and the wheel started moving forward and backward on the mounted specimen. The number of passes was shown on the laptop connected with the machine. One complete to and fro movement of the wheel was taken as two passes. The LVDT (Linear Variable Differential Transformer) measures the impression of a rut in millimeters of the unit at the same time as the motion of the wheel. The machine automatically stopped when the required number of passes were achieved. Results were saved for further use. when required number of passes achieved. Results were saved for the further use.

Figure 3.30 Hamburg Wheel Tracking Device

3.12.1 Result of Hamburg Wheel Tracker Test (HWT)

The software generates two different types of information: an Excel sheet of data and a graph that compares the number of passes to the rut depth in millimeters. An application called Wheel Tracker Graph will show graphs and header data for the Wheel Tracking Machine. In order to examine and graph retrospective data, it includes the option to choose a database during startup. The program includes the ability to save the header data and graphs to a file.

The graph will be presented as an image, and you can retrieve the rut depth at each number of passes by creating a report and then importing it into an MS Excel file.

3.12.2 A novel quantitative analysis of HWT test results for Moisture Susceptibility

It is generally accepted that post-compaction, visco-plastic deformation, and moisture degradation all played a part in the rutting that occurred during the HWT test (stripping). This work proposes a novel analysis method to separate these three behaviors and assess the influence of moisture damage exclusively. The steps are illustrated as follows. Fitting the 6-order polynomial to the raw rutting curve:

Equation below illustrates how the curve is initially fitted with a 6-order polynomial, similar to the Iowa technique:

$R(N) = p6N^6 + p5N^5 + p4N^4 + p3N^3 + p2N^2 + p1N + p0$ (1) (Lv et al., 2022)

Where Pi (i = 0, 1, 2, 3, 4, 5, 6) are the regression constants, N is the number of loading passes, and R is the fitted rutting depth. To avoid the random noise of the actual rutting curve, the subsequent analysis will be based on the polynomial curve. When the polynomial's first derivative reaches a local minimum, an inflection point is added (where the creep slope is inserted in the Iowa DOT method). It is thought that the inflection point marks the beginning of moisture damage (stripping stage). In other words, only visco-plastic and post-compaction deformation can be blamed for the deformation from zero pass to the inflection point. separating the visco-plastic, moisture, and post-compaction deformation:

First, we distinguish between visco-plastic deformation and post-compaction deformation. According to Texas DOT's study (Lv et al., 2022), it is anticipated that the post-compaction is the only factor responsible for the deformation that occurred within 1000 passes. After that, the deformation is entirely visco-plastic, lasting from 1000 passes to the inflection point. As a result, the rutting depth after 1000 passes is referred to as the post-compaction rutting depth (Rp), and the curve from 1000 passes to the inflection point can be utilised to model and forecast the development of visco-plastic deformation.

3.13 Summary

This chapter discusses the laboratory testing of aggregate and binders to prepare bituminous paving mixtures in a controlled environment. To produce the bituminous mix, only those materials have been utilized that met or exceeded the required criteria. Volumetric characteristics of the bituminous mix were computed, and the Optimum bindercontent (OBC) was established. Furthermore, super pave gyratory and Marshall sample preparation and testing technique that was used for the rutting resistance, ITS, M_R, and TSR testing of HMA samples have been described in greater detail.

CHAPTER 4

DATA ANALYSIS AND RESULTS

4.1 Introduction

The analysis and findings for modified and unmodified asphalt concrete mixtures are presented in this chapter. Aggregates from Babuzai, Mardan, and bitumen penetration grade 60/70 from ARL made up unaltered mixes. By weight of the ideal binder contents, modified mixtures consisted of Nano-clay (2 %, 4 %, 6 %, and 8 %) and SBS (4.5 %) that were utilized to modify asphalt concrete. Performance testing was carried out using the usual sample preparation described in the preceding chapter. Three performance tests were conducted: the Hamburg wheel tracking test using the DWT device to measure the rut depth of modified and unmodified asphalt concrete mixtures, the ITS with UTM-25 test to assess moisture resistance, and the Resilient Modulus test using UTM-25 to assess stiffness.

4.2 Bitumen Physical Properties Result

In the investigation, the physical characteristics of bitumen obtained from ARL 60/70 penetration grade were used. According to test results, the bitumen results met the requirements. Table 4.1 provides an overview of the tests that were performed.

Table 4.1: Summary of Bitumen Physical Properties

Type of Test	Asphalt ARL 60 / 70						
	Test	Binder					
	Standards	Base Binder	4.5%SBS				
			0%NC	2%NC	4%NC	6%NC	8%NC
Penetration (dm)	ASTM D5 /AASHTO T49	68.33	56.27	53.47	46.51	41.92	37.51
Flash & Fire Point(°C)	ASTM D92 /AASHTO T53	233 & 278	257 & 271	260 & 273	277 & 297	254 & 280	251 & 277
Softening Point (°C)	ASTM D36 /AASHTO T53	49.1	53.3	55.1	58.5	65.4	68.1

4.3 Aggregates Physical Properties Result

Crushed aggregate was used in the study from the Babuzai, Mardan quarry site. Results from common tests on aggregates show that the values are within the usual range and the aggregate is suitable for use. Table 4.2 provides a summary of the tests performed on aggregates.

Table 4.2: Summary of Aggregate Tests Results

Aggregates Tests Summary				
Test Descriptions	**Specifications Reference**		**Results**	**Limits**
Fractured Particles	ASTM D 5821		**99.00%**	
Elongations Index (EI)	ASTM D 4791		***3.70%***	≤10%
Flakiness Index (FI)	ASTM D 4791		***9.30%***	≤10%
Aggregates Absorption	Fine Aggregate	ASTM C 127	***2.55%***	≤3%
	Coarse Aggregate		***0.81%***	≤3%
Impact Value	BS 812		***17.23%***	≤30%
Crushing Value	BS 812		***20.52%***	≤30%
Los Angles Abrasion	ASTM C 131		***28.00%***	≤30%
Specific Gravity	Fine Aggregate	ASTM C 128	***2.628***	-
	Coarse Aggregate	ASTM C 127	***2.632***	-
Clay particles	Fine Aggregate	ASTM C 142	***5.620%***	-
	Coarse Aggregate	ASTM C 142	***2.812%***	-

4.4 Marshall Mix Design/Job mix formula for OBC

OBC was calculated using bitumen content with 4% air spaces (optimal bitumen content). The OBC was discovered to be 4.3%, which is 4% air spaces. Using the presented graphs, all volumetric characteristics were calculated for the 4.3% binder content. The outcomes were compared to the NHA design specification (Table 4.3). All of the outcomes were within the parameters of the plan. The results are tabulated as shown in Table 4.3.

Table 4.3: Marshall Test Results at Optimum Asphalt Contents (OBC)

Marshall parameters	Measured Value	Criteria	Remarks
OBC (%)	4.30%	At 4% Air Void	----
Unit Weight (g/cm3)	2.38	NA	----
VMA (%)	13.2	13 (Min)	Pass
VFA (%)	70	65-75	Pass
Stability (kN)	10.2	8.006 (Min)	Pass
Flow (mm)	2.6	2.0-3.5	Pass

4.5 Binder Consistency Testing

The penetration value assesses the softness and hardness of the asphalt binder at a moderate temperature. Lower penetration values have an impact on the stiffness of the binder. Figure 4.1 demonstrates that the penetration value fell when the proportion of NC in bitumen with the addition of SBS was raised, showing that the fluency and stiffness of the bitumen decreased and rose, respectively. When 4.5 % of SBS by weight of bitumen was added, penetration value decreased by 17.65 %, whereas at 4.5 % SBS with 6 % NC, penetration value decreased by 38.65%.

The softening point is a common test for estimating the approximate boundary between viscous and viscoelastic bitumen behavior and is used to gauge how resistant bitumen is to deformation in high-temperature environments. The addition of 4.5% SBS by mass of Bitumen raised the softening point by around 4°C. based on figure 4.1.

When 4.5 % SBS and 6 % NC by mass were added, the bitumen softening point increased by 16.3°C. Large surface energy, a high young's modulus, and the existence of contact forces between SBS and NC in the binder make it difficult to be penetrated by most materials. Figure 4.1 illustrates the differences between modified and unmodified bitumen. The penetration value is used to gauge how stiff and how quickly the asphalt binder hardens at a reasonable temperature. A lower penetration number suggests that the binder is stiffer.

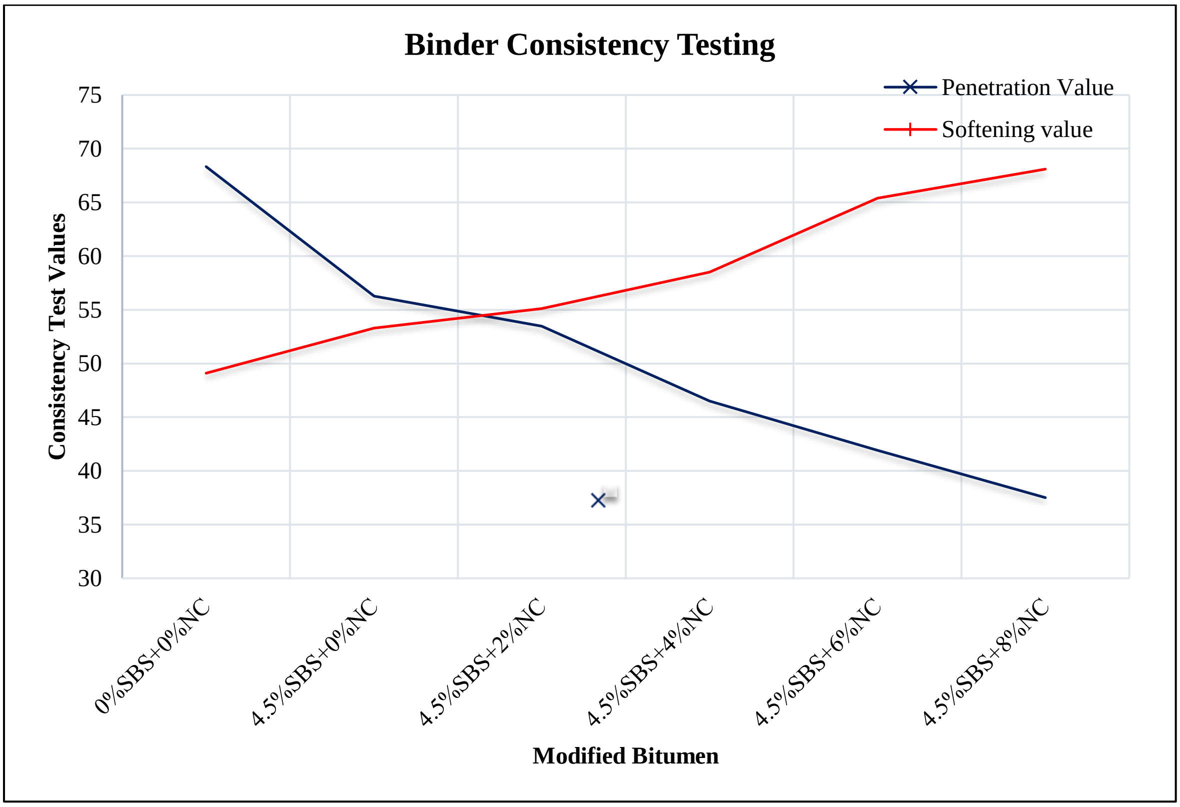


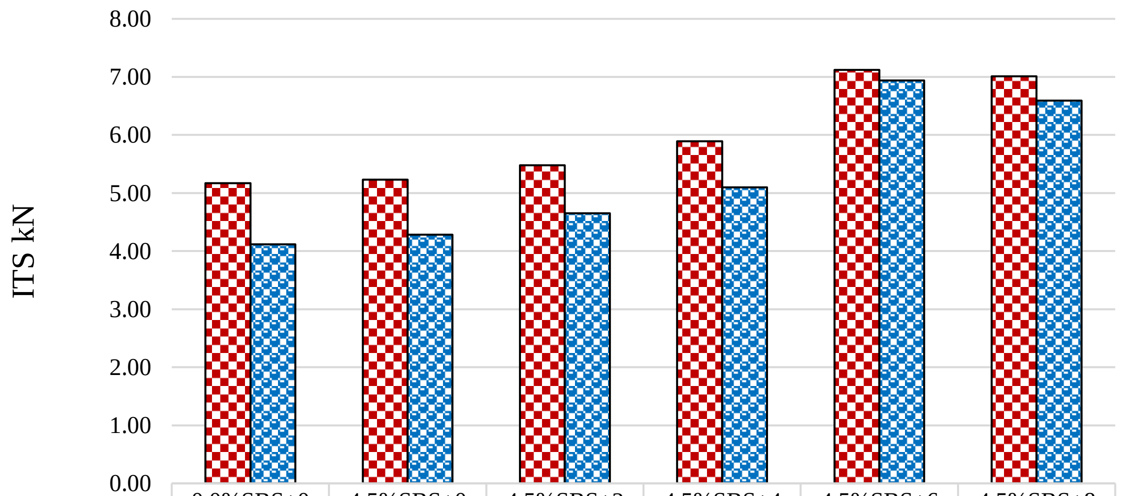
Figure 4.1 Effect on Binder consistency with the addition of additive

4.5 Indirect Tensile Strength Test

Indirect Tensile Strength Test evaluates the tensile qualities of compacted concrete mixtures in accordance with ASTM D 6931-07. The phrase "moisture susceptibility" describes the difference between the tensile strength of unconditioned and conditioned specimens. With ALDOT 361, samples were prepared by being placed in a water bath for 24 hours at 60°C. Each % of the SBS and Nano-clay combination was tested in a total of three Marshall duplicates prior to the tensile strength testing. Both samples that had and didn't have moisture conditioning underwent testing. The samples were tested using a universal testing machine with monotonic loading, and their dimensions were 100 mm in diameter and 65 mm thick. Samples were again condition for one hour at 25°C in UTM after being condition for 24 hours at 60oC. The tested combinations' conditioned and unconditioned strength values are listed in Table 4.4. The monotonic loading schematic diagram utilized for the TSR test is shown in Figure 4.2. Figure 4.3. compares the strengths of the control mixture (which has not been modified in any way) with modified mixtures that contain various amounts of SBS and Nano-clay, both with and without conditioning. Tensile strength ratio is shown in Figure 4.3, and trend in values is shown in Figure 4.5. According to the results, **4.5 % SBS and 6 % NC** content outperforms the control mix with a **17.97 %** improvement in TSR.

Table 4.4: Summary ITS Tests Results

Description	**Average Unconditioned Strength (S1) kN**	**Average Conditioned Strength (S2) kN**	**TSR =S2/S1(%)**
0%SBS+0%NC	5.170	4.110	0.79
4.5%SBS+0%NC	5.228	4.280	0.82
4.5%SBS+2%NC	5.480	4.650	0.85
4.5%SBS+4%NC	5.890	5.100	0.87
4.5%SBS+6%NC	7.120	6.940	0.97
4.5%SBS+8%NC	7.010	6.560	0.94



	0.0%SBS+0 %NC	4.5%SBS+0 %NC	4.5%SBS+2 %NC	4.5%SBS+4 %NC	4.5%SBS+6 %NC	4.5%SBS+8 %NC
ITS Uncond.	5.17	5.23	5.48	5.89	7.12	7.01
ITS Cond.	4.12	4.28	4.65	5.10	6.94	6.59

Figure 4.3: Tensile Strength Values of Specimens

Nano-clay reduces the amount of air spaces in AC mixtures with 4.5 percent SBS to 6 percent and then raises the amount of air voids with additional additions. As the amount of nano-clay is increased, changed mixes become more moisture vulnerable because to an increase in air spaces. However, the addition of 4.5 percent SBS and 6 percent NC improved the moisture susceptibility of AC mixtures, and the results show that this combination outperformed all other SBS Nano-clay combinations.

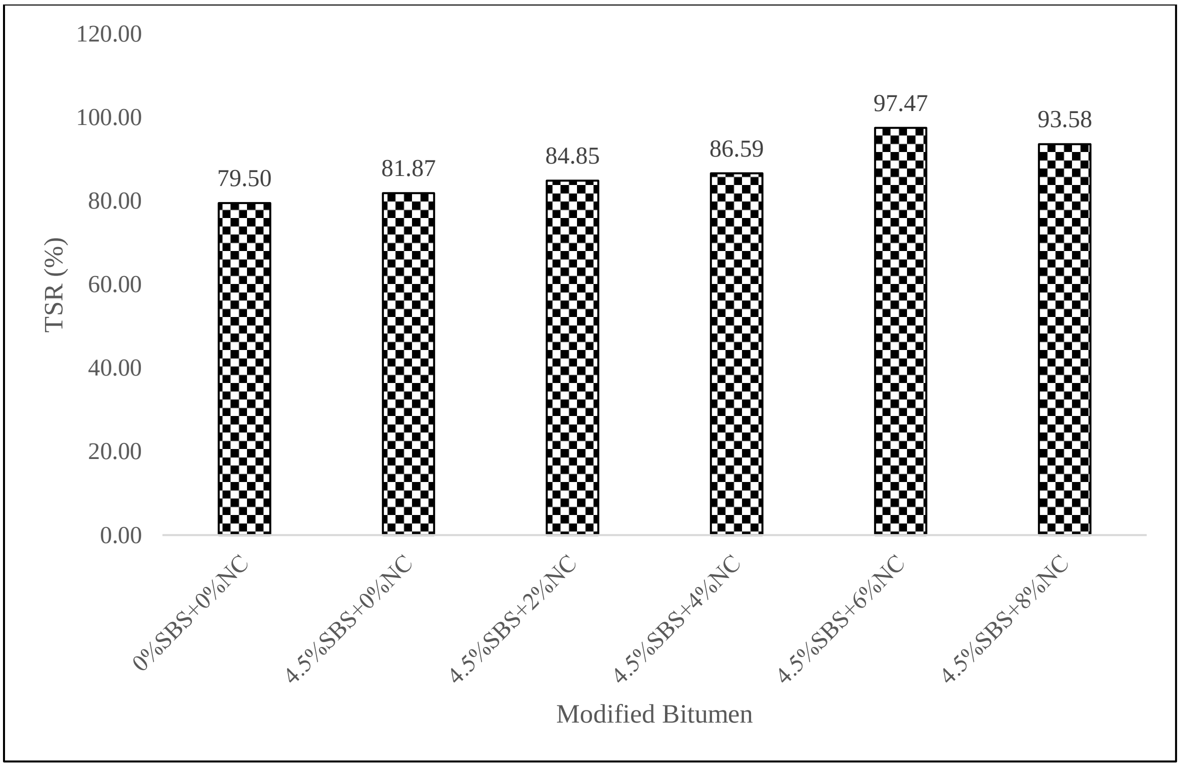


Figure 4.4: Tensile Strength Ratio of Specimens

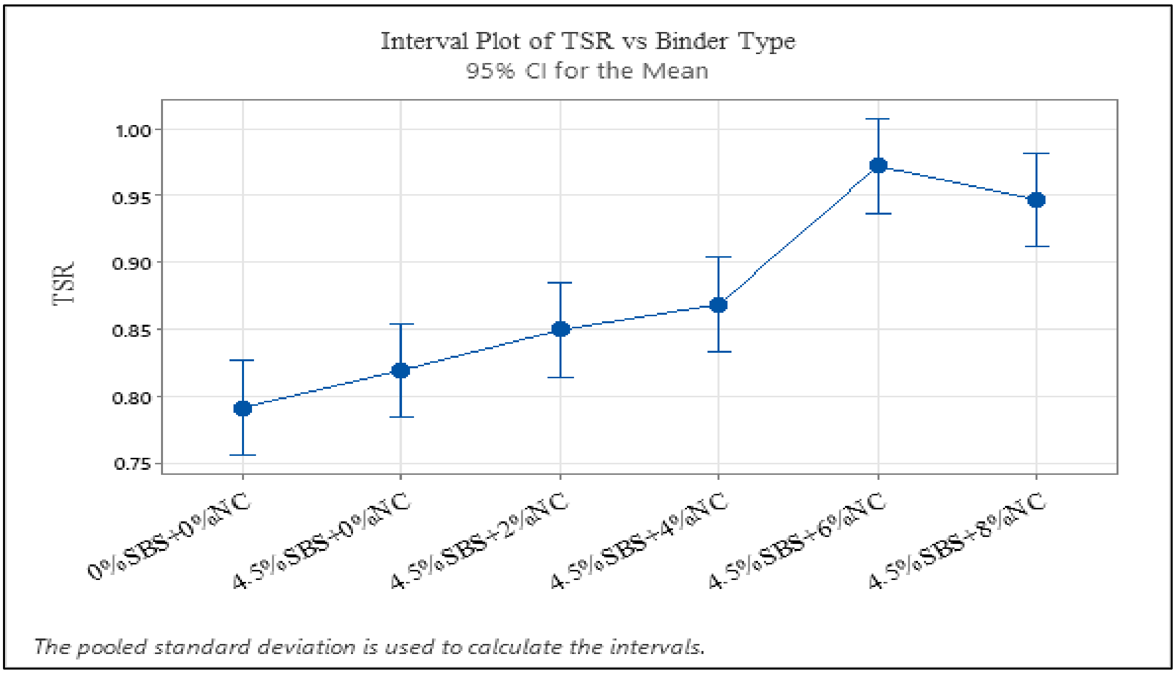


Figure 4.5: Tensile Strength Ratio Trend

4.6 Resilient Modulus

The resilient modulus reading can be used to evaluate how the pavement structure will respond to applied vehicular loads. When a material is subjected to cyclic loading, the ratio of applied stress to recoverable strain is recorded as the resilient modulus, a relative measure of mixture stiffness. A performance test called the resilient modulus can be used to evaluate the quality of materials and collect information for the design of paving. An important statistic for studying pavement response to traffic stress and predicting pavement performance is resilient modulus.

For the stiffness modulus performance test in accordance with ASTM D 4123, three duplicates of each proportion of SBS and Nano-clay combinations are constructed. The software that comes with the test equipment calculates the modulus for each load pulse. Figure 4.5 illustrates how to perform the IDT for resilient modulus on a cylindrical specimen with conventional Marshall specimen parameters (Dia. 100mm and Thickness 65mm) using a haversine waveform and a load applied vertically in the vertical diametric plane. With the help of the load application and horizontal elastic deformation, the resilient modulus value was estimated. The following equation is used to determine the horizontal displacement caused by the real load for each load pulse and to compute the robust modulus:

$$\text{Resilient Modulus, } MR = \frac{P\ (0.27+\mu)}{(\Delta h)\ t}$$

Where:

M_R Resilient Modulus

P Cyclic Load

t Thickness of Specimen

Δh Recoverable horizontal deformation

μ Poisson ratio

Figure 4.6 shows the details of the trend between these observed values of the resilience modulus of the control and SBS and NC modified AC mixes that are shown in Figure 4.5. Results clearly show that the combination of 4.5 % SBS and 6 % NC produces the greatest outcomes. The inclusion of this mixture of modifiers, according to the results, increased the M_R **by 1.39 times** over the initial control mix. After 6 % NC content, the value of the Resilient modulus starts to drop when the modifier content is further increased. Therefore, based on these findings, it is suggested that the optimal combination is **4.5 % SBS and 6 % NC**.

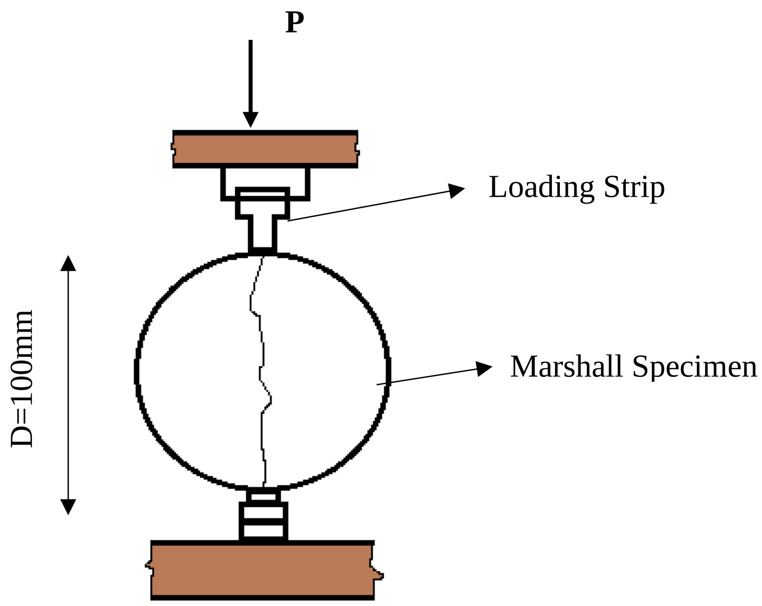


Figure 4.6: Schematic Diagram for Resilient Modulus Testing

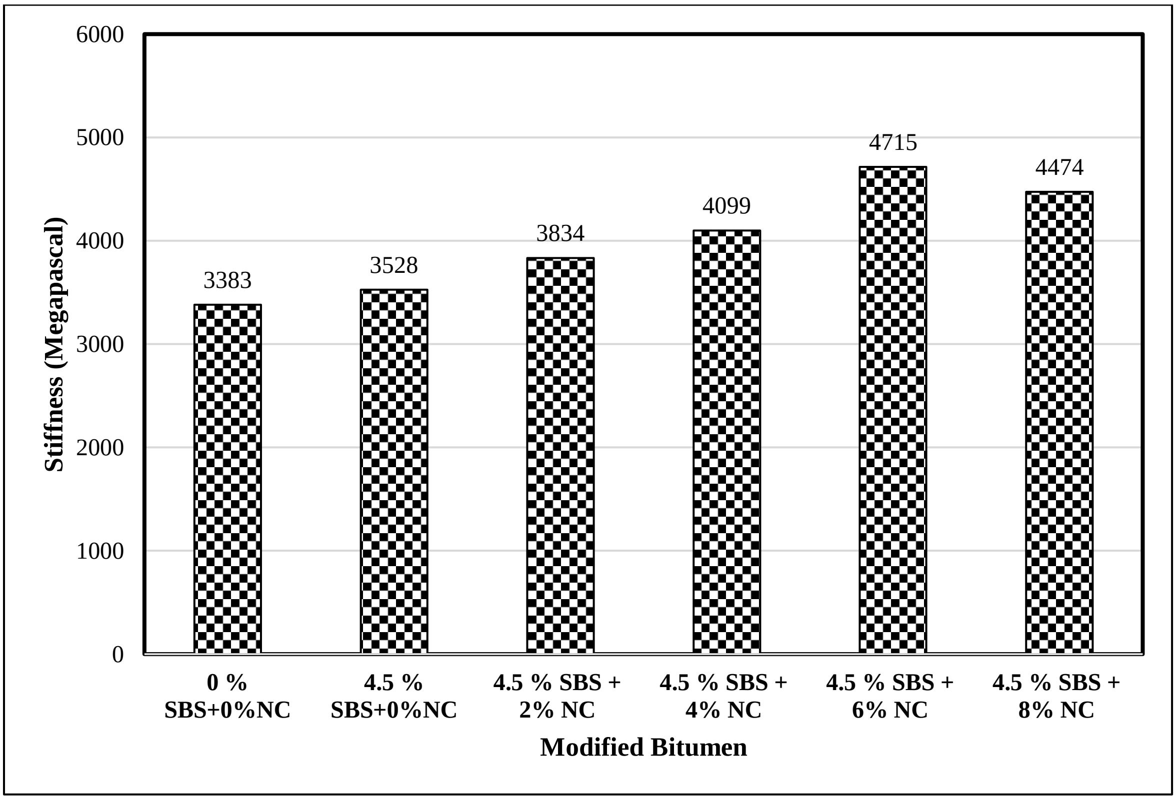


Figure 4.7: Resilient Modulus of modified mixtures at various combinations

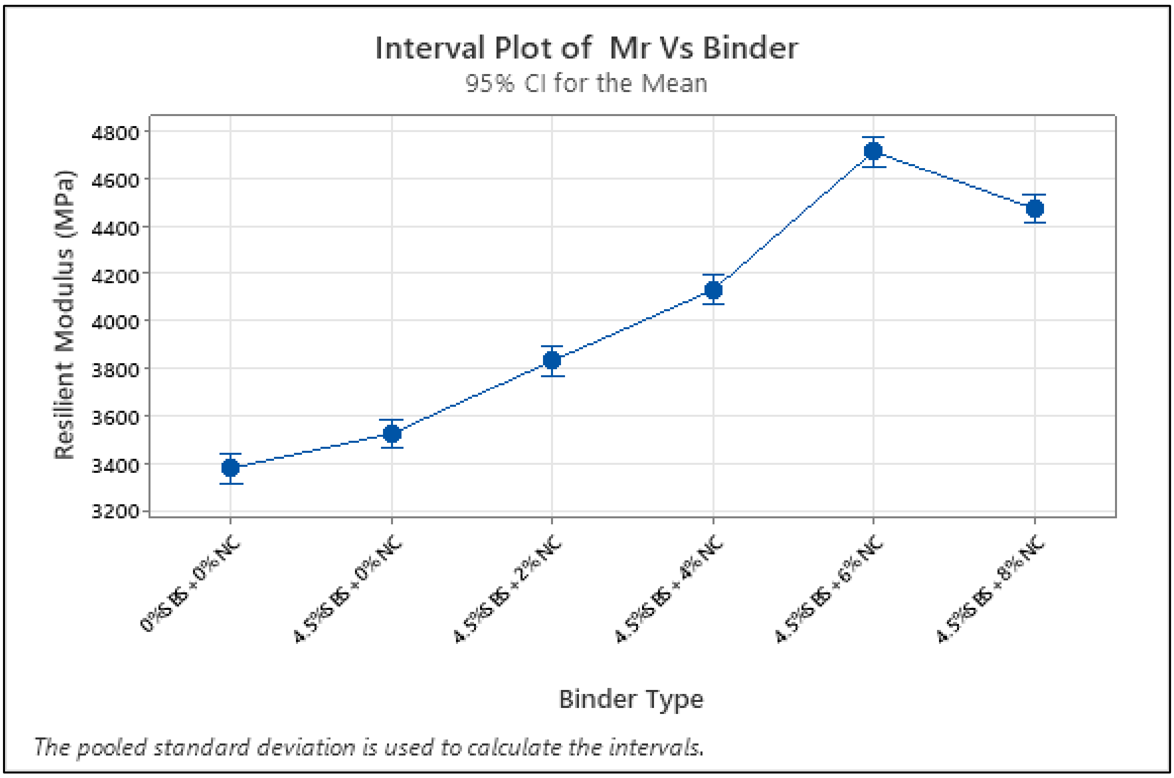


Figure 4.8: Trend line of Resilient Modulus values

4.7 Hamburg Wheel Tracking test

In order to compare the relative rut depth of original and modified HMA samples, wheel tracking tests were conducted using Superpave gyratory compacted samples of dia 6" and height 2.5". The samples (Control 60/70 and modified with 4.5 % SBS and (2 %, 4 %, 6 %, and 8 %) Nano-Clay) were put through 5000 cycles at a rate of 25 rpm, and the software then measured and plotted the rut depth that resulted from the test. Table 4-3 presents the test findings for rut depth for each specimen versus various modifier percentages during the course of 5000 cycles (10000 Passes). The rut depth is plotted against each SBS and NC combination in Figure 4-9.

As shown in Table 4-3, 4.5 % SBS and various percentages of Nano-Clay were used to make a total of 12 superpave gyratory samples of diameter 6” and thickness of 2.5” for the wheel tracker test, which examined the samples' rutting capacity. The specimens were tested pairs of two as par the AASHTO T-324 test procedure in the double wheel tracking test device, on which the wheel reciprocate back and forth on the specimens. The wheel run on the specimens up to 120 mm recording the rut depth at a distance of 20 mm from the start point, at 100 mm and at the center of the specimen. All of the specimens exhibited good rut resistance, however samples with increasing Nano-Clay concentration shown good resistance up to 6%, at which point the rut resistance began to drastically decline. The wheel tracker test parameters of 12.5 mm were met by all specimens.

Table 4.5: Summary of Wheel Tracker Test Results

Wheel Tracker Test (Test Standard AASHTO 324-11)	
Modifier	**Rutting Depth(mm) at 40°C and 5000 cycles**
0.0%SBS + 0%NC	3.23
4.5%SBS + 0%NC	1.63
4.5%SBS + 2%NC	1.48
4.5%SBS + 4%NC	1.34
4.5%SBS + 6%NC	1.28
4.5%SBS + 8%NC	2.50
Rut depth shall be less than 12.5mm	

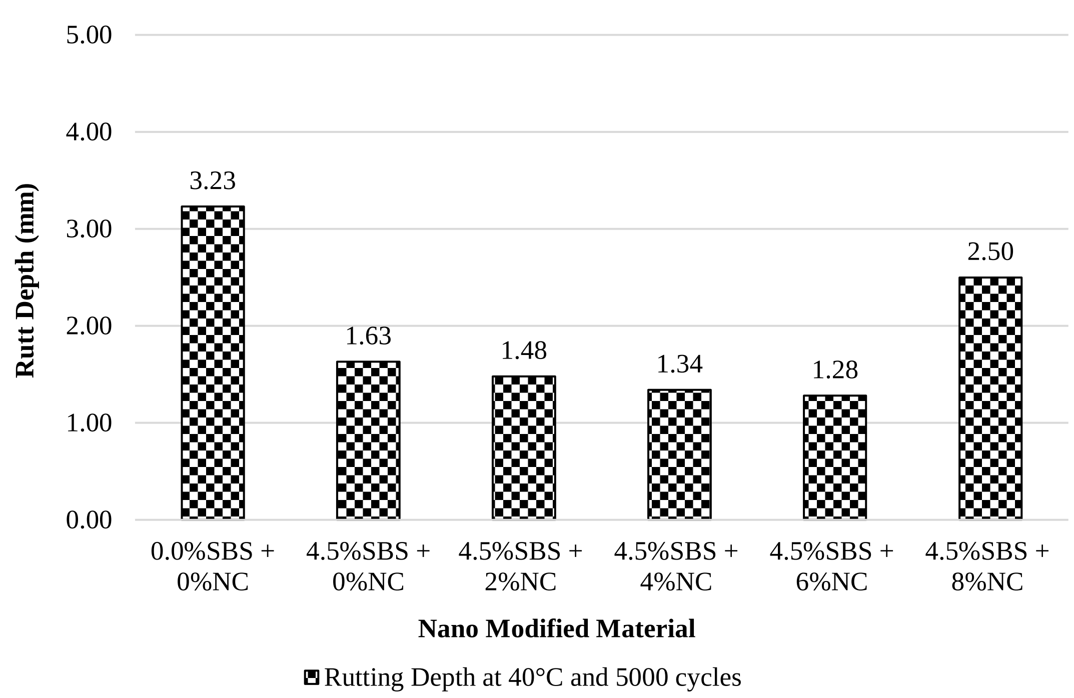


Figure 4.9: Hamburg Wheel Tracking Test Results

4.8 Analysis of variance of test Results for Resilient Modulus

One-way ANOVA was performed to analyze the test results and find the significance of the factors involved. Furthermore, pairwise Tukey analysis was carried out. The Tukey analysis test is used to compare the means of different group and find significance of factors to response factor. Different groups are related to data means and are assigned letters. The results are shown in table is the degree of freedom and the and P-value and F-values. For

95% confidence level the P-value should be less than 0.05 for a significance factor. While the F-value should be more than 10.

The Table 4.6 shows that the modifier is significant for the M_R values as a response factor as the P-value is less than 0.05 and the F-value is more than 10.

Table 4.6 Analysis of Variance M_R

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	5	4151029	830206	345.60	0.000
Error	12	28827	2402		
Total	17	4179856			

Table 4.7 shows the factors which is the modified bitumen and the means and standard deviation of the data.

Table 4.7 Means of M_R with 95% confidence interval

Factor	N	Mean	St. Dev	95% CI
0%SBS+0%NC	3	3382.8	50.0	(3321.2, 3444.5)
4.5%SBS+0%NC	3	3527.7	51.8	(3466.0, 3589.3)
4.5%SBS+2%NC	3	3834.0	51.0	(3772.3, 3895.7)
4.5%SBS+4%NC	3	4131.9	28.3	(4070.3, 4193.6)
4.5%SBS+6%NC	3	4715.0	53.0	(4653.3, 4776.7)
4.5%SBS+8%NC	3	4473.5	55.0	(4411.8, 4535.2)

Pooled St. Dev = 49.0128

Table 4.8 shows the means, and the grouping is done by assigning different letters to each modifier percentages.

Table 4.8 Grouping Information Using Tukey Method and 95% Confidence

Factor	N	Mean	Grouping					
4.5%SBS+6%NC	3	4715.0	A					
4.5%SBS+8%NC	3	4473.5		B				
4.5%SBS+4%NC	3	4131.9			C			
4.5%SBS+2%NC	3	3834.0				D		
4.5%SBS+0%NC	3	3527.7					E	
0%SBS+0%NC	3	3382.8						F

Means that do not share a letter are significantly different.

The following Table 4.9 shows the results of the Tukey simultaneous test for the any possible difference of level. It is seen that there is no insignificance in the results and all the groups are significant for the M_R, which is a response factor. The P-value is less than 0.05 for difference of level. This shows that as the modifier content is increased in the mix relative to the control mix there is a significant change on the results of Mr. Figure shows the distribution of means from a reference zero line which indicate that whichever of the mean contains zero is not significant differently.

Table 4.9 Tukey Simultaneous Tests for Differences of Means

Difference of Levels	Difference of Means	SE of Difference	95% CI	Adjusted P-Value
4.5%SBS+0%NC - 0%SBS+0%NC	144.8	40.0	(10.4, 279.2)	0.032
4.5%SBS+2%NC - 0%SBS+0%NC	451.2	40.0	(316.8, 585.6)	0.000
4.5%SBS+4%NC - 0%SBS+0%NC	749.1	40.0	(614.7, 883.5)	0.000
4.5%SBS+6%NC - 0%SBS+0%NC	1332.2	40.0	(1197.8, 1466.6)	0.000
4.5%SBS+8%NC - 0%SBS+0%NC	1090.7	40.0	(956.3, 1225.1)	0.000
4.5%SBS+2%NC - 4.5%SBS+0%NC	306.3	40.0	(171.9, 440.7)	0.000
4.5%SBS+4%NC - 4.5%SBS+0%NC	604.3	40.0	(469.8, 738.7)	0.000
4.5%SBS+6%NC - 4.5%SBS+0%NC	1187.3	40.0	(1052.9, 1321.7)	0.000
4.5%SBS+8%NC - 4.5%SBS+0%NC	945.8	40.0	(811.4, 1080.2)	0.000
4.5%SBS+4%NC - 4.5%SBS+2%NC	297.9	40.0	(163.5, 432.3)	0.000
4.5%SBS+6%NC - 4.5%SBS+2%NC	881.0	40.0	(746.6, 1015.4)	0.000
4.5%SBS+8%NC - 4.5%SBS+2%NC	639.5	40.0	(505.1, 773.9)	0.000
4.5%SBS+6%NC - 4.5%SBS+4%NC	583.1	40.0	(448.7, 717.5)	0.000
4.5%SBS+8%NC - 4.5%SBS+4%NC	341.6	40.0	(207.2, 476.0)	0.000
4.5%SBS+8%NC - 4.5%SBS+6%NC	-241.5	40.0	(-375.9, -107.1)	0.001

Individual confidence level = 99.43%

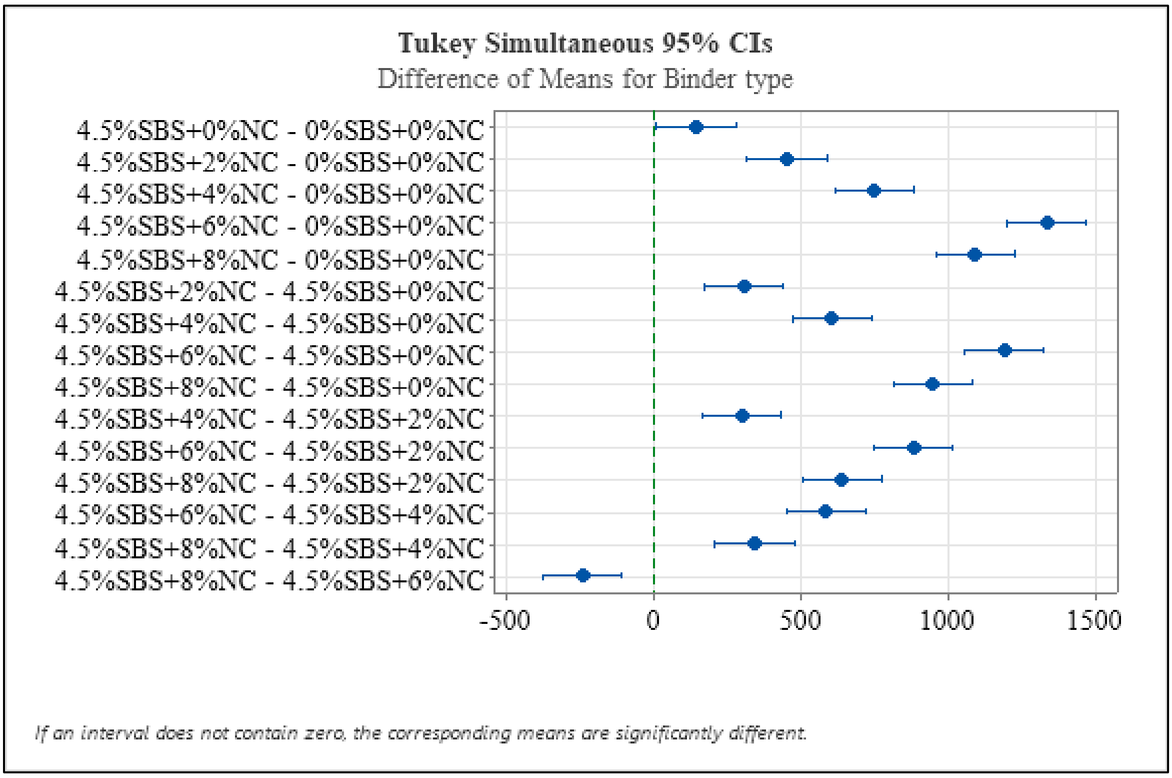


Figure 4-10: Distribution of means of M_R values

4.9 Analysis of variance of test Results for TSR

The Table 4.10 shows that the modifier is significant for the TSR values as a response factor as the P-value is less than 0.05 and the F-value is more than 10.

Table 4.10 Analysis of Variance TSR

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	5	0.075515	0.015103	19.46	0.000
Error	12	0.009312	0.000776		
Total	17	0.084827			

Table 4.11 shows the factors which is the modified bitumen, and the means and standard deviation of the data and Table 4.12 shows the means, and the grouping is done by assigning different letters to each modifier percentages.

Table 4.11 Means 95% CI of TSR with 95% confidence intervals

Factor	N	Mean	St. Dev	95% CI
0%SBS+0%NC	3	0.7917	0.0301	(0.7566, 0.8267)
4.5%SBS+0%NC	3	0.8196	0.0200	(0.7845, 0.8546)
4.5%SBS+2%NC	3	0.8495	0.0200	(0.8145, 0.8846)
4.5%SBS+4%NC	3	0.8686	0.0401	(0.8336, 0.9037)
4.5%SBS+6%NC	3	0.9716	0.0301	(0.9365, 1.0066)
4.5%SBS+8%NC	3	0.9467	0.0208	(0.9116, 0.9817)

Pooled St. Dev = 0.0278567

Table 4.12 Grouping Information Using the Tukey Method and 95% Confidence

Factor	N	Mean	Grouping		
4.5%SBS+6%NC	3	0.9716	A		
4.5%SBS+8%NC	3	0.9467	A		
4.5%SBS+4%NC	3	0.8686		B	
4.5%SBS+2%NC	3	0.8495		B	C
4.5%SBS+0%NC	3	0.8196		B	C
0%SBS+0%NC	3	0.7917			C

Means that do not share a letter are significantly different.

Table 4.13 summarizes the results of the Tukey simultaneous test for the any possible difference of level. It is seen that there is significance in some results but some of the pair's comparisons is insignificant for the TSR, which is a response factor. The P-value is less than 0.05 for difference of level for the significance while P-value is more than 0.05 which shows insignificance of the pair. This shows that as the modifier content is increased in the mix relative to the control mix there is a significant change on the results of TSR. Figure shows the distribution of means from a reference zero line which indicate that whichever of the mean contains zero is not significant differently.

Table 4.13 Tukey Simultaneous Tests for Differences of Means of TSR

Difference of Levels	Difference of Means	SE of Difference	95% CI	Adjusted P-Value
4.5%SBS+0%NC - 0%SBS+0%NC	0.0279	0.0227	(-0.0485, 0.1043)	0.816
4.5%SBS+2%NC - 0%SBS+0%NC	0.0579	0.0227	(-0.0185, 0.1343)	0.186
4.5%SBS+4%NC - 0%SBS+0%NC	0.0770	0.0227	(0.0006, 0.1534)	0.048
4.5%SBS+6%NC - 0%SBS+0%NC	0.1799	0.0227	(0.1035, 0.2563)	0.000
4.5%SBS+8%NC - 0%SBS+0%NC	0.1550	0.0227	(0.0786, 0.2314)	0.000
4.5%SBS+2%NC - 4.5%SBS+0%NC	0.0300	0.0227	(-0.0464, 0.1064)	0.771
4.5%SBS+4%NC - 4.5%SBS+0%NC	0.0491	0.0227	(-0.0273, 0.1255)	0.323
4.5%SBS+6%NC - 4.5%SBS+0%NC	0.1520	0.0227	(0.0756, 0.2284)	0.000
4.5%SBS+8%NC - 4.5%SBS+0%NC	0.1271	0.0227	(0.0507, 0.2035)	0.001
4.5%SBS+4%NC - 4.5%SBS+2%NC	0.0191	0.0227	(-0.0573, 0.0955)	0.954
4.5%SBS+6%NC - 4.5%SBS+2%NC	0.1221	0.0227	(0.0457, 0.1985)	0.002
4.5%SBS+8%NC - 4.5%SBS+2%NC	0.0972	0.0227	(0.0208, 0.1735)	0.011
4.5%SBS+6%NC - 4.5%SBS+4%NC	0.1029	0.0227	(0.0266, 0.1793)	0.007
4.5%SBS+8%NC - 4.5%SBS+4%NC	0.0780	0.0227	(0.0016, 0.1544)	0.044
4.5%SBS+8%NC - 4.5%SBS+6%NC	-0.0249	0.0227	(-0.1013, 0.0515)	0.874

Individual confidence level = 99.43%

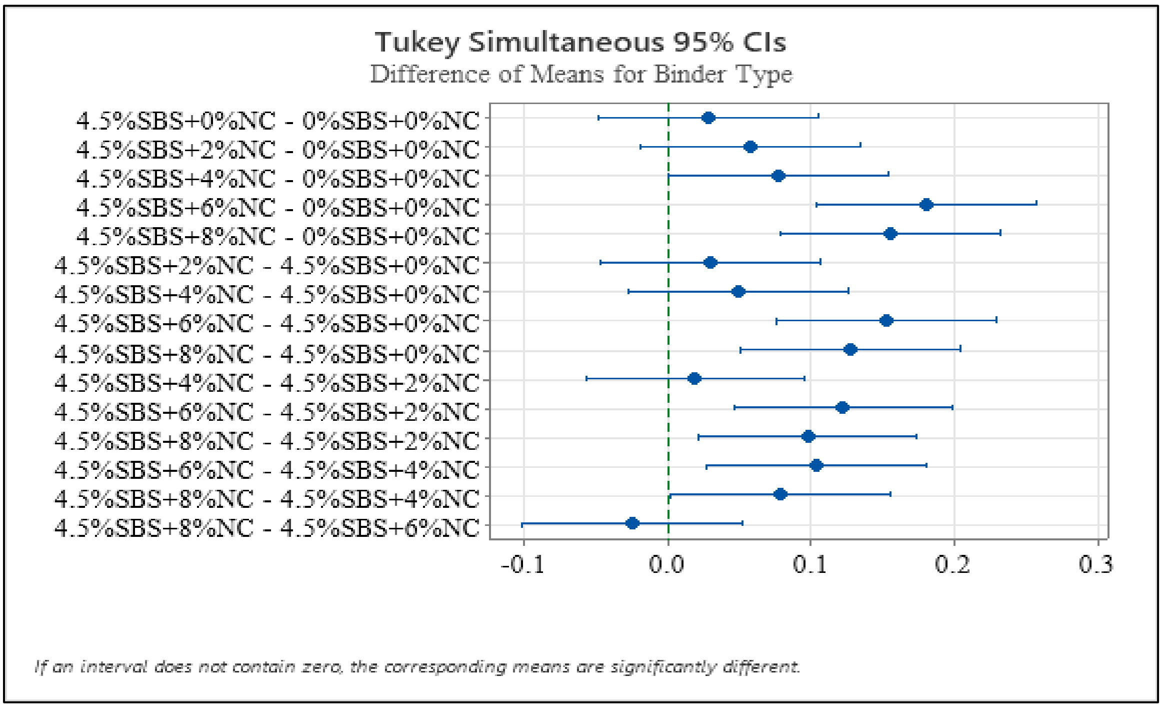


Figure 4-11: Distribution of means of TSR values

4.10 Analysis of variance of test Results for Rut Depth

The Table 4.14 shows that the modifier is significant for the Rut Depth values as a response factor as the P-value is less than 0.05 and the F-value is more than 10.

Table 4.14 Analysis of Variance Rut Depth

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	5	8.4452	1.68905	38.43	0.000
Error	12	0.4684	0.03903		
Total	17	8.9136			

Table 4.15 shows the factors which is the modified bitumen and the means and standard deviation of the data.

Table 4.16 shows the means, and the grouping is done by assigning different letters to each modifier percentages. This shows that the factor which share letter in there grouping are not significant differently.

Table 4.15 Means Rut Depth with 95% confidence intervals

Factor	N	Mean	St. Dev	95% CI
0%SBS+0%NC	3	3.230	0.220	(2.954, 3.506)
4.5%SBS+0%NC	3	1.630	0.180	(1.354, 1.906)
4.5%SBS+2%NC	3	1.480	0.200	(1.204, 1.756)
4.5%SBS+4%NC	3	1.340	0.300	(1.064, 1.616)
4.5%SBS+6%NC	3	1.280	0.220	(1.004, 1.556)
4.5%SBS+8%NC	3	2.5000	0.1700	(2.2244, 2.7756)

Pooled St.Dev = 0.197569

Table 4.16 Grouping Information Using the Tukey Method and 95% Confidence

Factor	N	Mean	Grouping		
0%SBS+0%NC	3	3.230	A		
4.5%SBS+8%NC	3	2.500		B	
4.5%SBS+0%NC	3	1.630			C
4.5%SBS+4%NC	3	1.340			C
4.5%SBS+2%NC	3	1.480			C
4.5%SBS+6%NC	3	1.280			C

Means that do not share a letter are significantly different.

Table 4.17 illustrates the results of the Tukey simultaneous test for any possible difference of level. It is seen that there no insignificance in the results and all the groups are significant for the Rut Depth, which is a response factor. The P-value is more than 0.05 for some pairs of difference of level. This shows that some levels are not significant as compared to the lower mean of the 4.5% SBS+6% NC. The zero line is the reference line for the differences of means of the rut depth values for different modifier materials contents. Figure shows the distribution of means from a reference zero line which indicate that whichever of the mean contains zero is not significant differently. And the one which include zero reference not significant.

Table 4.17 Tukey Simultaneous Tests for Differences of Means of Rut Depth

Difference of Levels	Difference of Means	SE of Difference	95% CI	Adjusted P-Value
4.5%SBS+0%NC - 0%SBS+0%NC	-1.600	0.179	(-2.201, -0.999)	0.000
4.5%SBS+2%NC - 0%SBS+0%NC	-1.750	0.179	(-2.351, -1.149)	0.000
4.5%SBS+4%NC - 0%SBS+0%NC	-1.890	0.179	(-2.491, -1.289)	0.000
4.5%SBS+6%NC - 0%SBS+0%NC	-1.950	0.179	(-2.551, -1.349)	0.000
4.5%SBS+8%NC - 0%SBS+0%NC	-0.730	0.179	(-1.331, -0.129)	0.015
4.5%SBS+2%NC - 4.5%SBS+0%NC	-0.150	0.179	(-0.751, 0.451)	0.954
4.5%SBS+4%NC - 4.5%SBS+0%NC	-0.290	0.179	(-0.891, 0.311)	0.601
4.5%SBS+6%NC - 4.5%SBS+0%NC	-0.350	0.179	(-0.951, 0.251)	0.417
4.5%SBS+8%NC - 4.5%SBS+0%NC	0.870	0.179	(0.269, 1.471)	0.004
4.5%SBS+4%NC - 4.5%SBS+2%NC	-0.140	0.179	(-0.741, 0.461)	0.965
4.5%SBS+6%NC - 4.5%SBS+2%NC	-0.200	0.179	(-0.801, 0.401)	0.865
4.5%SBS+8%NC - 4.5%SBS+2%NC	1.020	0.179	(0.419, 1.621)	0.001
4.5%SBS+6%NC - 4.5%SBS+4%NC	-0.060	0.179	(-0.661, 0.541)	0.999
4.5%SBS+8%NC - 4.5%SBS+4%NC	1.160	0.179	(0.559, 1.761)	0.000
4.5%SBS+8%NC - 4.5%SBS+6%NC	1.220	0.179	(0.619, 1.821)	0.000

Individual confidence level = 99.43%

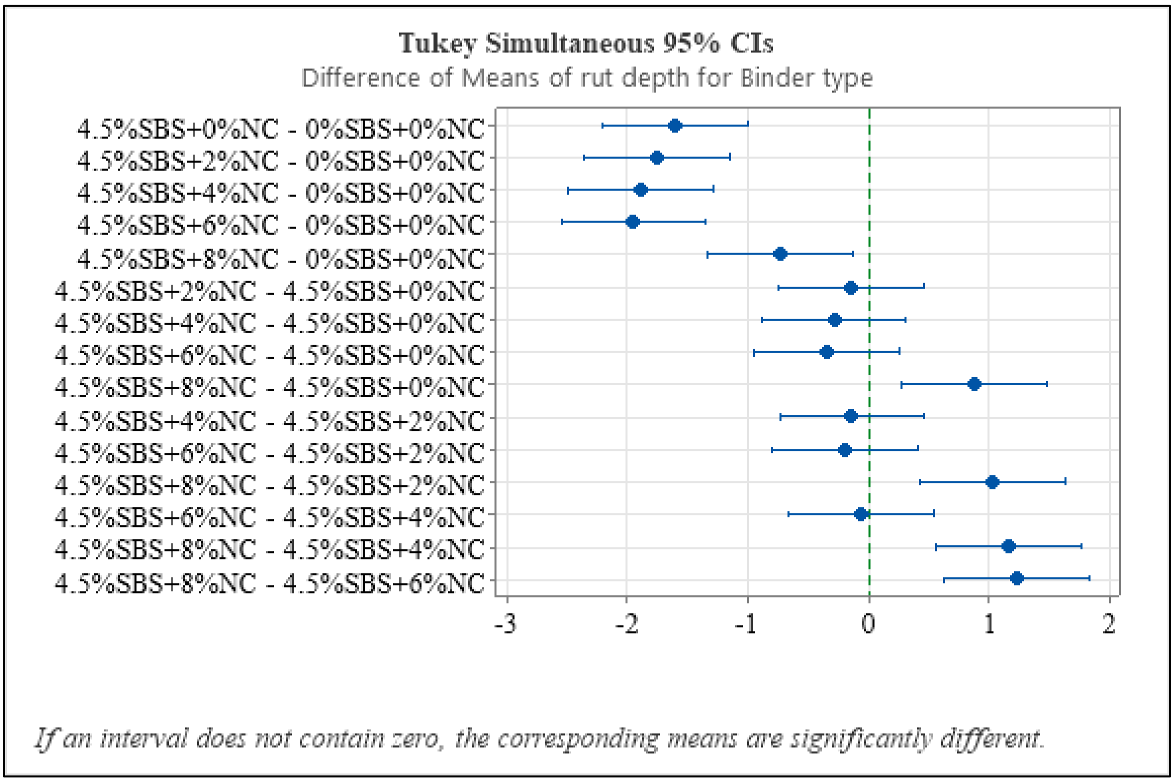


Figure 4-12: Distribution of difference of means of Rut Depth

4.11 Moisture Susceptibility from HWT test results

The results of rut test that is Hamburg Wheel Tracking Test (HWTT) are used to find the stripping inflection point (SIP), which is meant to be the starting point of stripping phase in the rutting. Stripping is the peeling of bitumen cover from the aggregate in HMA mixes. The data collected are plotted against the cycles, then a 6^{th} degree polynomial fitted line is incorporated as regression line. After that from the equation of the that trend/regression line its first derivative is taken, and the values are again plotted against the number of cycles. The lowest first point in the curve is the stripping inflection point. The Figure 4.13 shows the plot of the rut depth value against cycles having a fitted curve, the equation is also shown.

The following Figure 4.13 explains rutting behavior during the wheel tracking test, which shows that the rutting is the process of three stages, the first stage is the post compaction stage, which is the consolidation of the specimen at the start of the test passes and according to the IOWA DOT the post compaction continues to first 1000 passes of the tracking wheel. The second stage is the visco-plastic deformation stage in which the deformation is due to the plastic behavior of binder in the mix and the flow of the bitumen started, this stage start after the post compaction ended and last up to the stripping inflection point. The third and last stage is the moisture damage in which the stripping starts and the aggregates loses the binder coating, this starts at the stripping inflection point(SIP) and ends at the failure of the specimen for rutting which is 12.5mm(Lv et al., 2022).

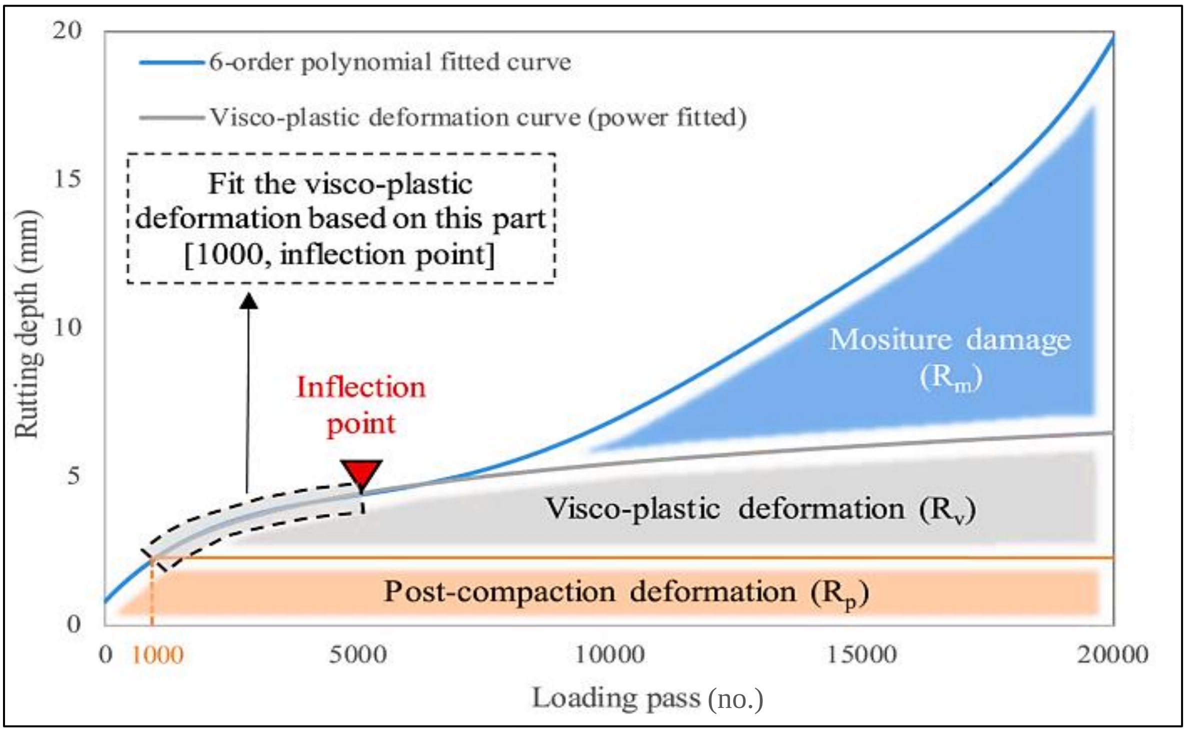


Figure 4.13 Stages of rutting behavior of HMA mixes

The IOWA DOT method is used to find the stripping inflection point (SIP). In this method a 6th order polynomial fitted curve is adjusted on the plotted curve of rut depth and cycles. Then the equation is generated from the regression line and its first derivative was calculated and plotted against the cycles. The first lowest point or value is the stripping inflection point. The stripping inflection point is believed to be the starting point of moisture damage. The equation is

$$R(N) = p6N^6 + p5N^5 + p4N^4 + p3N^3 + p2N^2 + p1N + p0$$

Where,

R(N) is the rut depth at N cycles

P (0,1,2,3,4,5,6) are the regression coefficients

N is the number of cycles

Figure 4.15 also shows plot of the first derivatives of the 6th order polynomial equation shown in Figure. These plots predicts that the samples qualify the minimum criteria of the IOWA DOT method, which says that the minimum passes for the stripping inflection point is 1000 passes showing the lowest point is at about 900 cycles (1800 passes). These plots are for base binder values. The figure 4.15 also shows that the moisture resistance increases as we add the modifier materials and gives an optimum value of 3000 cycles or 6000 passes for the mix having 6% NC with 4.5%SBS.

Figure 4.15 illustrates that all of the specimens are passing the IOWA DOT minimum criteria that is, the stripping inflection point (SIP) should occur after the first 1000 passes of the wheel tracking test. The figure 4.14 is the rutting data plotted against the cycles of the Hamburg wheel tracking Test (HWTT). And the regression line applied to all the modified mixes that are used in the experiment. The different color shows different modified mixes having 4.5%SBS and different percentages of NC.

Figure 4.15 shows the plots for the different mixes of different percentages of modified materials, having the first derivatives values of the regression line in figure 4.14 against the cycles of the Hamburg Wheel Tracking Test (HWTT). According to the IOWA DOT method the first lowest point or derivative is the point from where the moisture damage starts. This point is called the stripping inflection point (SIP). From this point the moisture damage starts in the rutting process of the bituminous pavements from where the stripping of the aggregates starts which is the loss of adhesion between aggregate and bitumen coating.

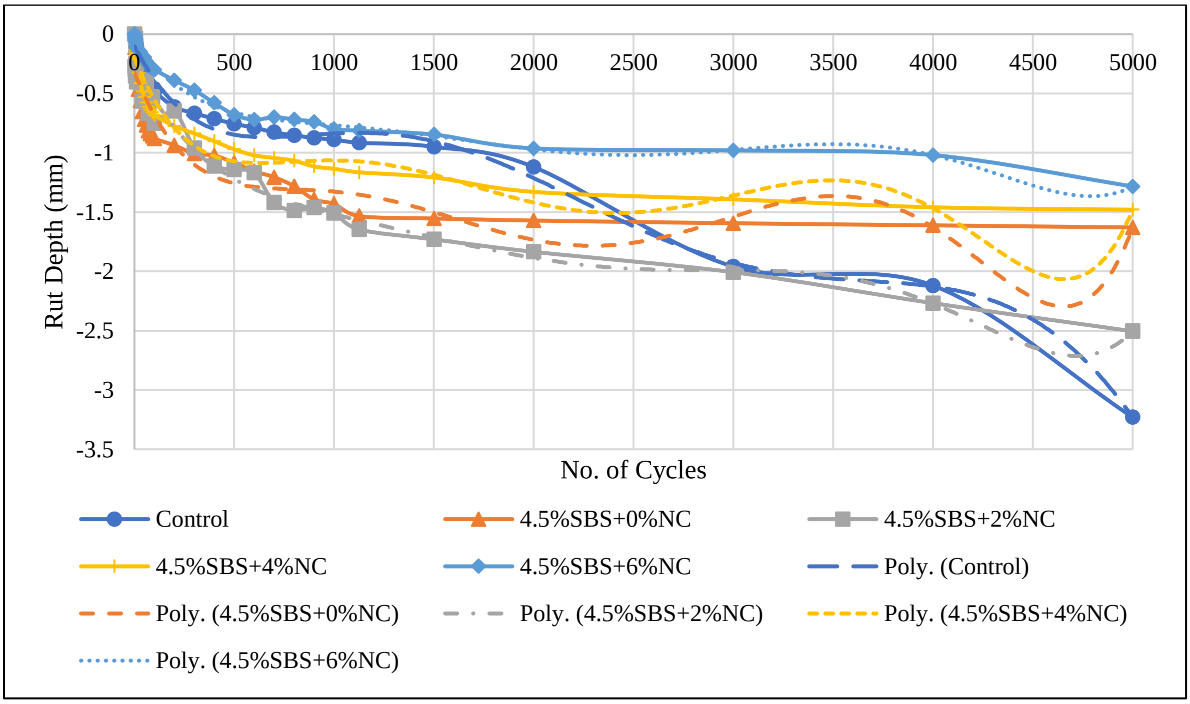


Figure 4.14 Rutting curve with 6-order polynomial regression line of different mixes

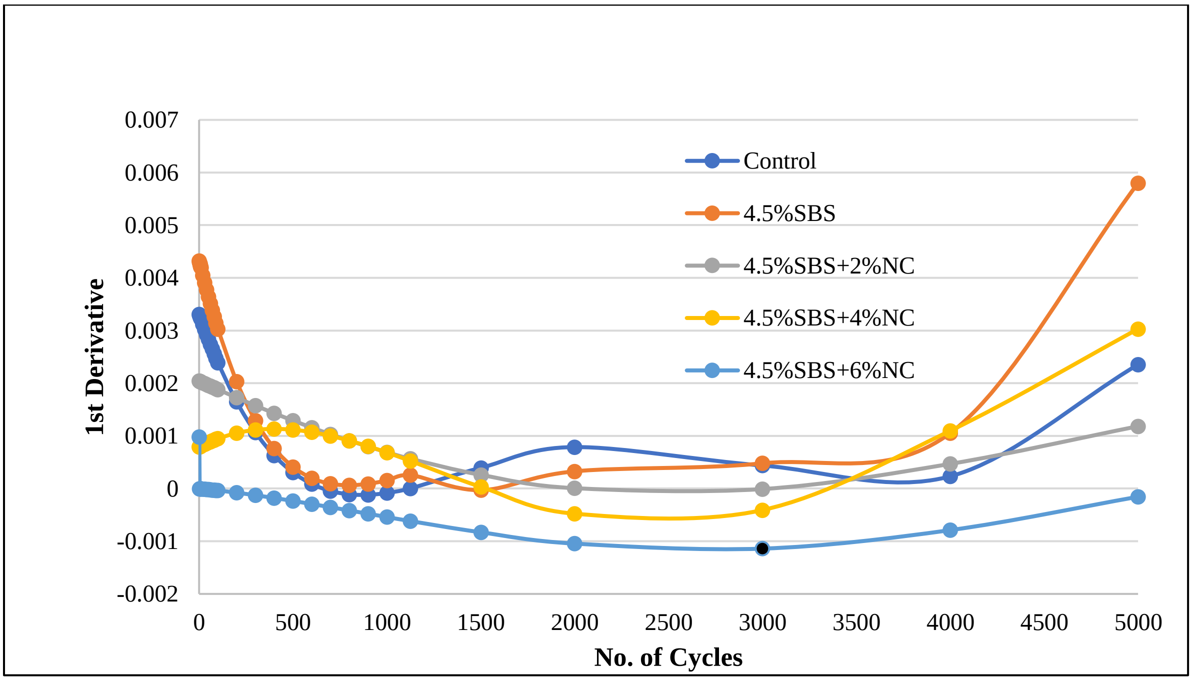


Figure 4.15 First derivative values of the 6th order polynomial for different mixes

4.12 Summary

It is evident from the study that adding SBS and Nano-clay combination to asphalt concrete mixtures can improve the properties of the asphalt. It has been observed that the addition of 4.5%SBS and 6%NC outperforms other combinations. Percentage of Nano-clay 6% with 4.5%SBS content significantly increases the mechanical properties of asphalt mixtures. It has been observed that by addition of SBS and Nano-Clay, have resulted in the enhancement of stiffness, rutting resistance and moisture susceptibility. Moisture susceptibility is **increased by 22.61%** of modified mixes. Moreover, the inclusion of SBS and NC also improves the *stiffness response* of asphalt mixes up to **1.39 times** of conventional mixes. Furthermore, the *Rut resistance* of asphalt mixes has been **enhanced by 39%.** HWT results also shows that resistance to stripping is increased **3 times.** Overall, the asphalt mix with **4.5%SBS and 6%NC** has the **best** results.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

The study's primary goal was to evaluate performance of Nanomaterials and polymers-modified asphalt mixtures to determine the efficacy of modifiers in hot mix asphalt. Conventional mixtures were composed of aggregate procured from Babuzai, Katlang, and bitumen penetration grade 60/70 obtained from ARL for comparison. Modified mixtures were composed of Styrene Butadiene Styrene (SBS) obtained from Shijiazhuang Tuya Tech Co., Ltd, Shijiazhuang, China, and Nano Clay (NC) procured from Miz Builders, a private company in Lahore. Marshall Mix design was adopted to determine optimum binder content (OBC-4.3%) and performance tests specimens were compacted using Gyratory compactor.

SBS 4.5% with five variations of NC (0%, 2%, 4%, 6%, 8%) to the weight of bitumen were used for preparation of Marshall and Superpave gyratory modified samples to establish optimal combination of modifiers. After the preparation of samples in compliance with respective standards, performance testing was conducted. Two performance tests were executed; Rutting using Hamburg Wheel Tracking Test (HWTT), Indirect Tensile Strength Test (ITS) for determining moisture resistance, and Resilient Modulus test to determine the Stiffness response of mixtures using UTM-25.

5.2 Conclusions

Based on the above performance testing conducted on conventional and SBS/ NC modified asphalt mixture specimens, following conclusions have been drawn from this research study:

a) Research study results verify the positive effect of the polymer and nanomaterials such as SBS and NC respectively, as a modifier in asphalt mixtures. Both materials as a modifier have enhanced the rut resistance, resilient modulus and decreased moisture susceptibility of AC mixtures.

b) SBS is one of the most widely used modifier materials in asphalt, however its combined effect with nano-clay has evaluated in this study are found beneficial. Polymers and nanomaterials reinforce the asphalt mixture by providing three-dimensional reinforcement. Research proves that SBS and NC can successfully be incorporated into asphalt concrete and can enhance its mechanical properties.

c) At higher percentages of NC exceeding 6% with 4.5% SBS tend to cause a decline in the performance of the modified bitumen due to the agglomeration and segregation of the nano particle of NC.

d) Specimens with 4.5% SBS and 6% NC content by weight of bitumen demonstrated the best results in performance tests as under:

   i. **39%** improvement in Rut resistance.
   ii. **22.61%** increase in moisture resistance.
   iii. Stripping resistance improvement up to **3 times**
   iv. Resilient modulus was enhanced by **1.39 times**.

e) One-way ANOVA was conducted with Tukey type comparisons of means for the tests results against the modifier materials. The comparisons show the significance of the modifier materials for all test results.

5.3 Recommendations

Study findings conclude that **4.5% SBS with 6% NC** content by weight of bitumen has outperformed as compared to other polymer and Nanomaterial (SBS and NC) tested percentages. Experimentally measured reporting optimum response of **stripping resistance improved by 3 times** while rut **resistance enhanced by 39%**, **enhancement in moisture resistance by 22.61%**, and **resilient modulus by 1.39 times, respectively**. Similarly, the ANOVA results also shows the significance of the modifiers. Therefore, the study recommends utilizing 4.5% SBS with 6% NC in asphalt mixture in relatively hot climates and pavements subjected to heavy/ slow moving traffic. Academia and Industry should collaborate on developing durable pavements with the inclusion of polymer and nanomaterials technology such as Styrene Butadiene Styrene and Nano clay which can save the national exchequer in the form of long-lasting pavements.

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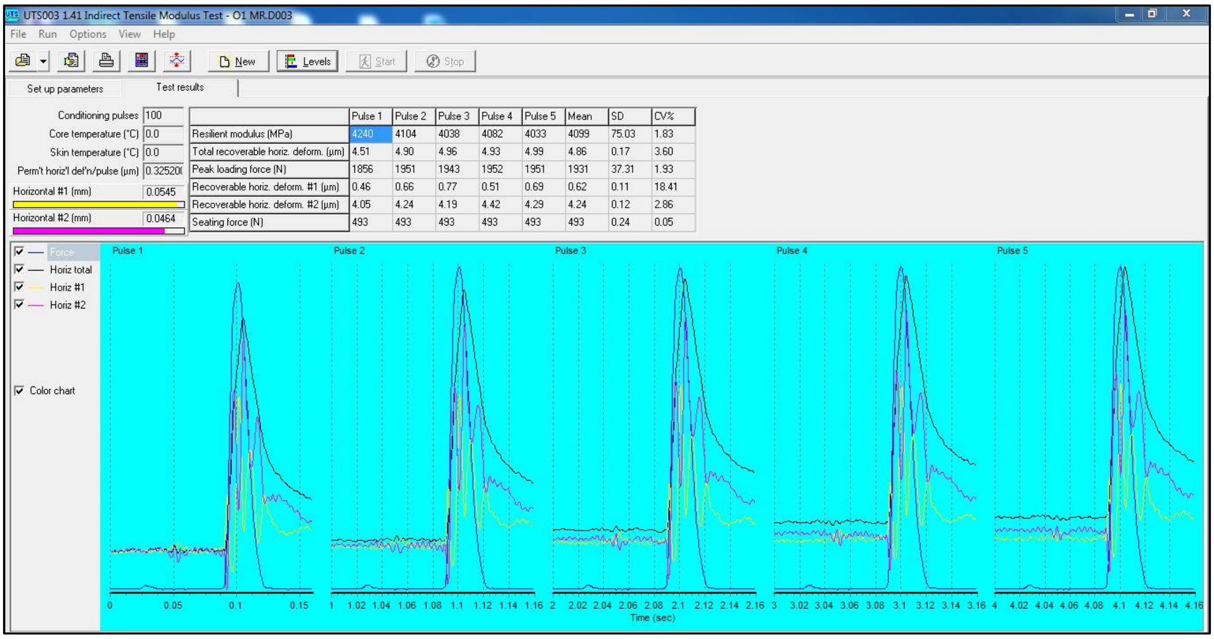
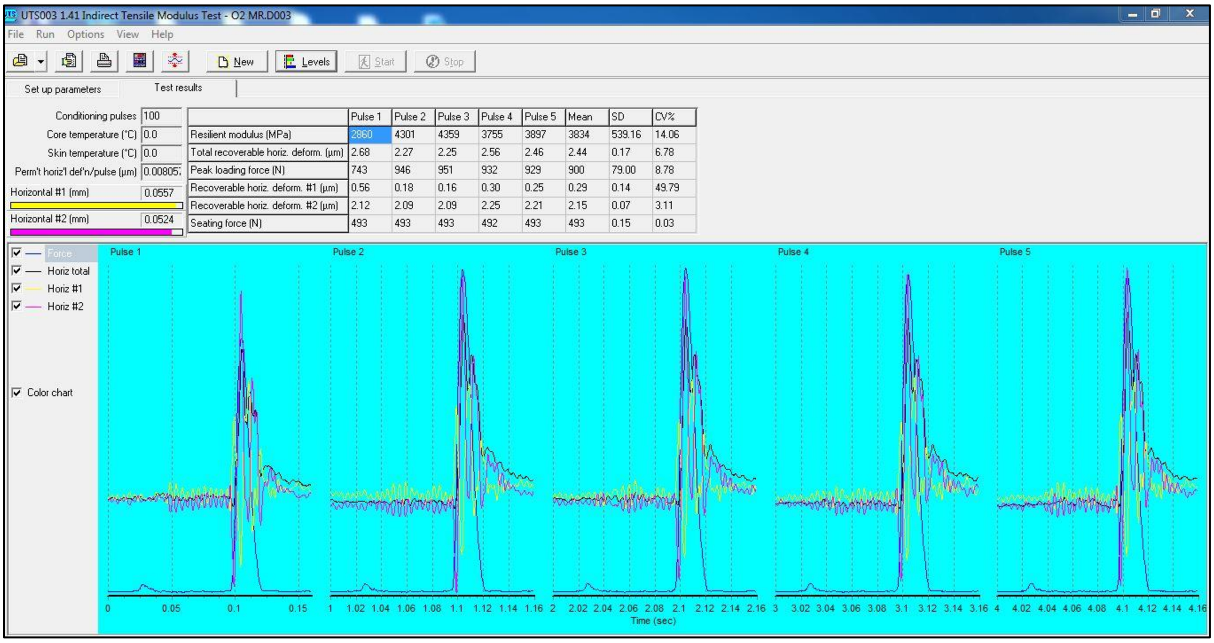
APPENDICES

Appendix-I

RESILIENT MODULUS (M_R) RESULTS SAMPLE

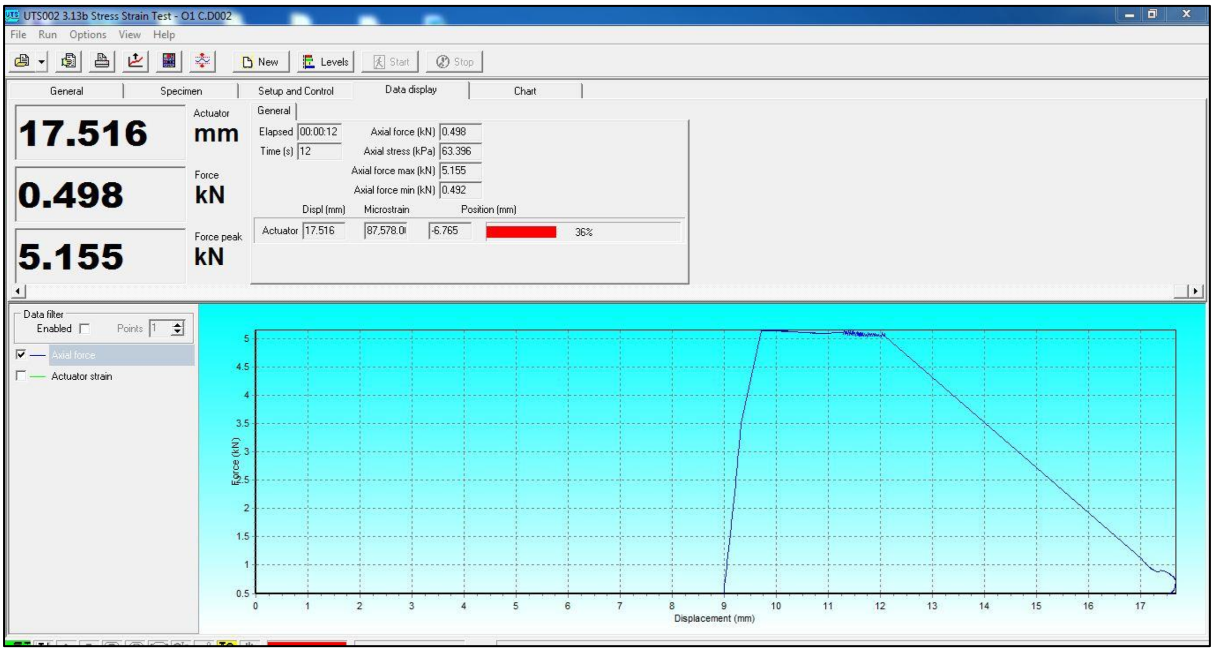


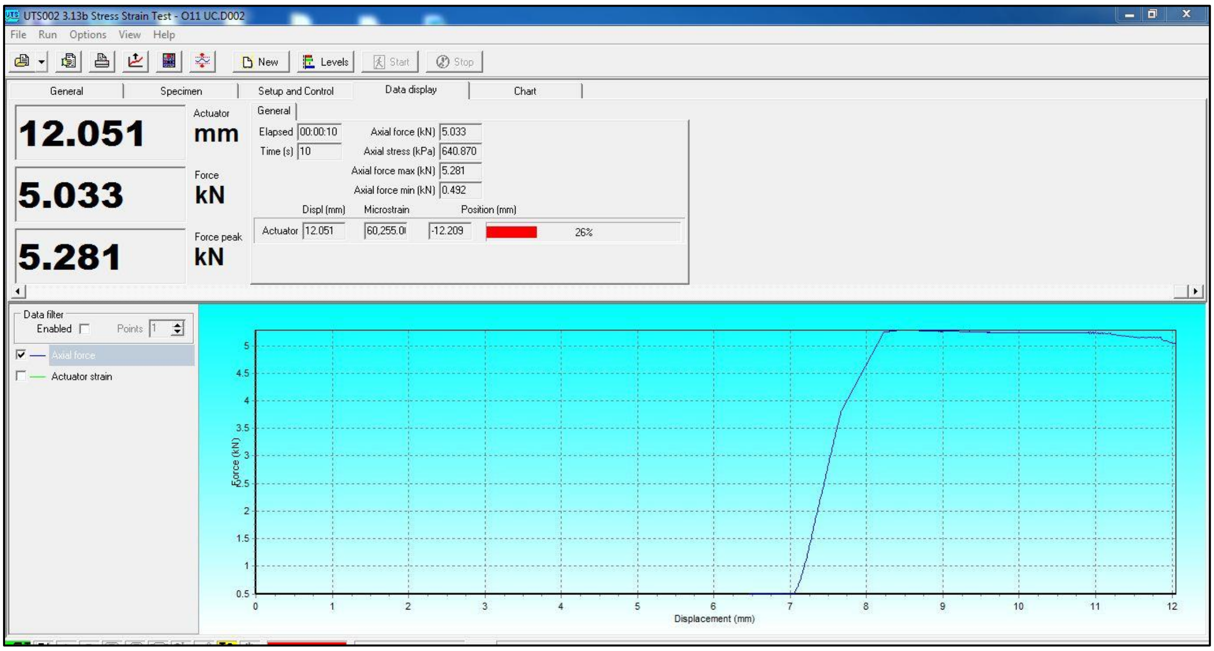
	Pulse 1	Pulse 2	Pulse 3	Pulse 4	Pulse 5	Mean	SD	CV%
Resilient modulus (MPa)	4240	4104	4038	4082	4033	4099	75.03	1.83
Total recoverable horiz. deform. (µm)	4.51	4.90	4.96	4.93	4.99	4.86	0.17	3.60
Peak loading force (N)	1856	1951	1943	1952	1951	1931	37.31	1.93
Recoverable horiz. deform. #1 (µm)	0.46	0.66	0.77	0.51	0.69	0.62	0.11	18.41
Recoverable horiz. deform. #2 (µm)	4.05	4.24	4.19	4.42	4.29	4.24	0.12	2.86
Seating force (N)	493	493	493	493	493	493	0.24	0.05



	Pulse 1	Pulse 2	Pulse 3	Pulse 4	Pulse 5	Mean	SD	CV%
Resilient modulus (MPa)	2860	4301	4359	3755	3897	3834	539.16	14.06
Total recoverable horiz. deform. (µm)	2.68	2.27	2.25	2.56	2.46	2.44	0.17	6.78
Peak loading force (N)	743	946	951	932	929	900	79.00	8.78
Recoverable horiz. deform. #1 (µm)	0.56	0.18	0.16	0.30	0.25	0.29	0.14	49.79
Recoverable horiz. deform. #2 (µm)	2.12	2.09	2.09	2.25	2.21	2.15	0.07	3.11
Seating force (N)	493	493	493	492	493	493	0.15	0.03

INDIRECT TENSILE STRENGTH (ITS) TEST RESULTS OUTPUT





Appendix-III

HAMBURG WHEEL TRACKING TEST RESULTS OUTPUT

CONTROLS PAVELAB SYSTEMS
EXCELLENCE IN PAVEMENT TESTING SOLUTIONS

Single test report

General Data

Final rut depth	**1.63 mm**		
Failure test	NO	End	Cycle number
Density		Void Percentage	
Type of thermal medium	Air	Feedback used	In chamber
Max Temp	34.8 °C	in Cycles	4969
Min Temp	31.3 °C	in Cycles	81
Customer	<Generic>		

Mixture

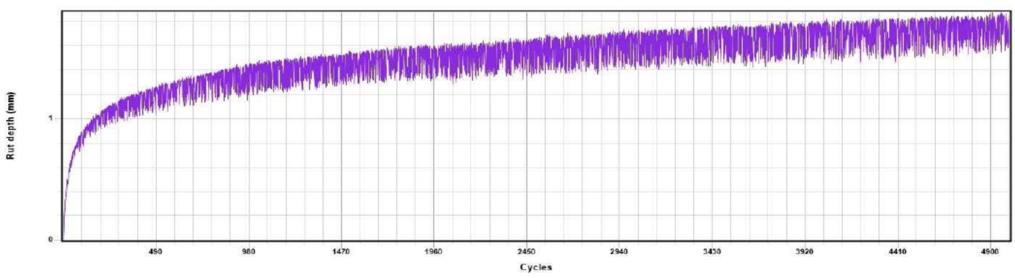
Mixture	<Generic>		
	Type	Weight (%)	Spec. weight (kg/m3)
Aggregate	-	0.00	0
Filler	-	0.00	0
Bitumen	-	0.00	0
Calculated Max Density	0 Kg/m^3	Production Type	-
Production Date	23/05/2022		
Compaction Type	-	Time conditioning	-

Start data test

Sample on test	2	Sample Number	2
ID Sample	0b	Sample Name	o222
Date	23/05/2022 12:42:44	Sample Type	Double Cores
Lenght		Width	
Diameter	150.00 mm	Thikness	63.00 mm
Weight	4.500 Kg	Age	5 dd
Max Rut depth	12.00	Max Number cycles	5000
Test Temp	40.0 °C	Wheels speed	25.0 cycle/min
Time to start	3 min	Operator	Asmat Khan
Cond. cycles	3 Cycles	Temp Limit.	3.0 °C

Test processing

Processing type	No Procedure



Single test report

General Data

Final rut depth	1.28 mm		
Failure test	NO	End	Cycle number
Density		Void Percentage	
Type of thermal medium	Air	Feedback used	In chamber
Max Temp	34.9 °C	in Cycles	4920
Min Temp	31.3 °C	in Cycles	34
Customer	<Generic>		

Mixture

Mixture	<Generic>		
	Type	Weight (%)	Spec. weight (kg/m3)
Aggregate	-	0.00	
Filler	-	0.00	0
Bitumen	-	0.00	0
Calculated Max Density	0 Kg/m^3	Production Type	-
Production Date	23/05/2022		
Compaction Type	-	Time conditioning	-

Start data test

Sample on test	1	Sample Number	1
ID Sample	0a	Sample Name	o111
Date	23/05/2022 12:42:43	Sample Type	Double Cores
Lenght		Width	
Diameter	150.00 mm	Thikness	63.00 mm
Weight	4.820 Kg	Age	5 dd
Max Rut depth	12.00	Max Number cycles	5000
Test Temp	40.0 °C	Wheels speed	25.0 cycle/min
Time to start	3 min	Operator	Asmat Khan
Cond. cycles	3 Cycles	Temp Limit.	3.0 °C

Test processing

Processing type	No Procedure