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Reuse of Recycled Plastic for Plastic Road Design

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Reuse of Recycled Plastic for Plastic Road Design

by

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ABSTRACT

REUSE OF RECYCLED PLASTIC FOR PLASTIC ROAD DESIGN

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The ever-increasing volume of traffic and the volatility of daily and seasonal temperatures that are being experienced globally make it essential that the quality and durability of roads are improved. Modifiers can improve the engineering properties of the asphalt mix in these situations. Perhaps plastics are the most promising of all types of modifiers. Since the disposal of plastic waste is a major concern for the environment, recycling of these waste materials is highly desired. The recycling and reuse of plastic waste is a prime example of sustainable waste management. It reduces the buildup of waste in landfills, thereby reducing hazards inherent in landfills, increases the landfills' active life, and potentially increases the lifespan and reduces the maintenance of roadways. Current study reflects two-fold major objectives (a) To improve the pavement strength; and (b) To reduce the plastic waste.

This study provides a critical review of the history and process of reusing plastic waste in asphalt, reviews previous studies that have been conducted on the subject, and assesses the potential reuse of plastic waste in asphalt pavement design today to improve the properties of pavement.

Using recycled plastics and recycled asphalt pavement (RAP) materials, this study was carried out to determine if there is a correlation between rutting and cracking. An experimental program was developed to conduct different volumetric and performance tests with different RAP content, plastic types and plastic content. Three types of recycled plastic were utilized – Low density polyethylene (LDPE), High density polyethylene (HDPE) and Polypropylene (PP) mixed with

Superpave SP-C mix to test the applicability of these materials for use in flexible pavement surface layers. According to the study, up to 8 percent recycled shredded plastic can be mixed with 15% RAP mixture, whereas increasing the RAP content by 25% reduces the performance of the mix. The value of indirect tensile strength (IDT) and tensile strength ratio (TSR) increases with an increase in plastic content. Moreover, rutting depth decreases significantly (up to 75% reduction) with the increase of plastic content. The plastic modified mix can double the service life of the pavement according to the findings of Overlay test.

In pavement design, one of the most important parameters is the rutting depth. However, the rutting test is too time consuming, and the equipment is expensive to conduct regularly. However, indirect tensile strength tests are easier to perform. Hence, in order to determine the value of the rutting depth for different combinations of recycled materials, MLR models were developed using the indirect tensile strength (IDT), RAP content, plastic type and plastic content.

This experimental research can be not only a solution to the worldwide waste plastic disposal problem but also can help reducing the depletion of natural resources.

Keywords: Rutting, cracking, modifier, recycle, plastic.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

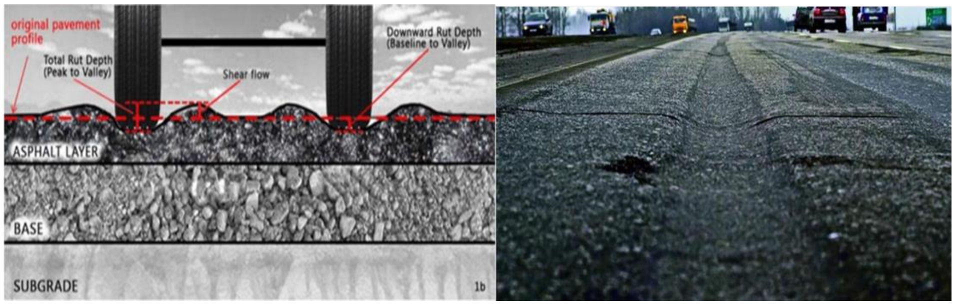
CHAPTER 1

INTRODUCTION

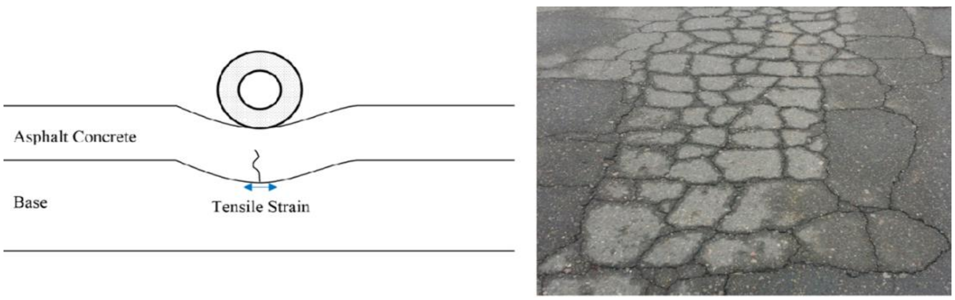
1.1 Background

Road networks represent a vital element of the national infrastructure, and their construction, operation, and maintenance take up a large share of the annual budget. They must fulfil the structural and functional requirements. It is necessary for structural requirements to provide a reasonable level of structural adequacy or durability over a reasonable timeframe. Functional requirement or serviceability is to provide road users a safe, comfortable and robust travel experience during its whole service life. The main purpose of paving structures is to distribute the traffic induced stresses and strains over the load bearing layers to an intense level which the material can withstand. The performance of the pavement primarily depends on material properties, traffic loads, structural layer thicknesses, subgrade material, and climate. In the United States, 94% of the more than 2 million miles of pavements and highways are constructed with asphalt concrete and cost over $30 billion (Sobolev et al., 2013). The number of vehicles on the road increased by 95.4% from 1980 to 2010, while the number of miles of roads only increased by 5.8% (Sobolev et al., 2013). Thus, engineers and researchers are relentlessly trying to achieve better performance from asphalt pavement to meet the rising demand for new and better highways. The performance of pavement is affected by bitumen's properties. Bitumen is a viscoelastic material with good mechanical and rheological properties that are suitable for waterproofing and covering roads and roofs due to its strong adhesion properties. The resistance of bitumen mixture to shoving and rutting under repeated traffic is one of its most important properties (Arbani and Pedram, 2016). Pavement tends to crack and prematurely fail because of increased traffic and high pressure caused by heavy vehicles. Low stability causes fatigue and reflective cracking on the road surface. Thus, the stability of the network should be high enough to handle traffic effectively.

As a result of high traffic loads, road surfaces have been subject to excessive stress, leading to deformation of the pavement (Doh et al., 2006). Permanent deformation occurs when a pavement lacks adequate stability, is improperly compacted, and do not have sufficient strength. Two of the main pavement distresses that the engineers try to avoid are rutting and fatigue cracking. Rutting of flexible pavement at high temperature is one of the main distresses that commonly occurred in flexible pavement due to accumulation of permanent deformation of each layer of the pavement structure under repetitive traffic loading action. Rutting often occurs due to repetitive traffic loads or plastic flow adjacent to the surface of the pavement, which have a significant impact on the pavement's structural and functional performance. (Figure 1.1a). Fatigue cracking is another means of distress mechanism causing degradation of pavements. It is caused by recurrent traffic loadings that result in crack initiation, crack propagation and ultimately catastrophic failure of material due to instability in crack growth. (Figure 1.1b). Estimates show that the state of Texas spends almost 9 billion USD to 15 billion USD annually (Jones and Jefferson,2012) which includes all the construction process, maintenance and repair of the roads. As per the records from "Texas Department of Transportation" it is actually required almost less than $5 billion if the excessive pavement distresses could be in control (Chukka and Carr, 2016).



(a)



(b)

Figure 1.1. (a) Rutting (Inzerillo et al., 2016; uceps.ru) (b) Fatigue cracking (Islam, 2015)

To perform sufficiently under a vast range of temperatures and provide resistance against degradation due to stresses and loads, a bituminous mix should have thermal stability along with load spreading and chemical stability to prevent the pavement from rutting, fatigue and reflective cracking. Over the last few decades, there has been a rapid surge of using additives in asphalt mixtures to ameliorate its thermos mechanical properties, which can offer high resistance to fatigue cracking and permanent deformation of bituminous layers (Gupta and Veeraragavan, 2009). Bitumen modified with recycled plastic provides a novel approach to overcoming new technical demands. The chemical or physical blending of waste plastics into the asphalt mix upgrades the properties of elasticity, strength, and adhesion, and increases the pavement’s lifespan (Kalantar et al. 2012).

Plastics have become an integral part of our lives, and the quantity used annually is gradually increasing due to population growth and rapid developments in industry. The United States produces approximately 35.4 million tons of plastic waste each year, which represents about 13.2%

of the total MSW generation (EPA, 2017). According to a 2017 environmental agency report, 75.5% of plastic waste is sent to landfills, 16% is incinerated, and only 8.5% of total plastic waste is currently recycled. Filling up the landfills with plastic waste has become a severe problem since its non-biodegradable nature means that it will persist in the ecosystem for a long period of time and lead to various environmental problems (Bale, 2011).

Utilizing post-consumed plastics decreases the number and intensity of adverse effects on the environment and reduces the number of products manufactured from new plastic. Reusing plastics has become more convenient since the inception of new mechanical, chemical, and thermal alternatives associated with the recycling process (Al-Salem, 2009; Siddique et al., 2008), and a well-organized collection, sorting, and recycling process facilitates the recycling of plastics to make new products (Jayaraman and Bhattacharya, 2004; Chakraborty and Mehta, 2017). For instance, recycled plastics can be used in wood-plastic composites (WPCs) and provide an additional market for recycled plastics (Najafi, 2013).

Plastic waste can also be used as fillers with virgin resins or other materials like concrete, or as fill material in road construction (Siddique et al., 2008). Many researchers are utilizing it effectively with bitumen, as a binding material in the construction of flexible pavements (Vasudevan et al., 2011; Movilla-Quesada et al., 2019). The benefits of incorporating recycled plastics into the asphalt mix for road construction include better stability, an increased capacity for water resistance, improved rutting and crack resistance, and enhanced binding properties and durability (Venkat, 2017; Moazami et al., 2019). Modifying asphalt with plastic will not only expand the lifespan of the roads but will also make them cost effective and environmentally friendly (Venkat, 2017).

1.2 PROBLEM STATEMENT

Cracks and premature failure of pavements have been attributed to increased traffic loads and high pressures resulting from heavy vehicles. As a result of traffic on the bituminous layer, fatigue cracking is very common. In order to avoid premature cracking of bituminous pavements, fatigue cracking of the bituminous layer must be carefully considered when designing the pavement and selecting materials for the pavement layer. An asphalt mixture lacking stability may also cause raveling and rutting of the road surface (Ahmad, 2014). To minimize the susceptibility of bituminous concrete mixtures to cracking and rutting, different solutions have been proposed. The use of additives such as rubber, latex, polymer etc has increased dramatically in recent years in bituminous concrete mixtures to improve its properties (Ahmad, 2014). Flexible pavements are being constructed using polymer modified bitumen, one of the newest construction materials. Plastic is one kind of polymer and if recycled plastic could be used as bitumen modifier it will represent a sustainable and cost-effective pavement solution.

The versatile use of plastics in our daily lives, such as packaging, building and construction, automotive, electric and electronic applications, creates considerable amounts of solid waste across the world. (Gawande et al., 2012). Having a high decomposition temperature, high resistance to ultraviolet radiation, and being non-biodegradable, they can remain on land and at sea for years, polluting the environment. As plastic usage is increasing day by day due to population growth, urbanization, development activities and frequent changes in the lifestyle, the disposal of plastic has been difficult (Venkat, 2017). Either they are landfilled or incinerated, both of which are not eco-friendly, polluting the land and air (Prasad et al. 2012). The recycling rate of plastic is sinking day by day since China banned importing foreign waste from many countries, including USA from

January 2018. As a result, plastics which were recycled before, are now going to landfills occupying a large volume of space for a long time. Despite the relatively small overall amount of plastics recycled, recycling of some types of plastic containers such as PET, HDPE, and PP is more significant. However, there is very little or no recycling facility for LDPE such as grocery bags, plastic wraps etc and Polystyrene.

Even though recycled plastics have been used for asphalt pavement, is often reported in the literature, the lack of widely accepted mix design remains one of the biggest barriers limiting its use all over the world. There are very limited literatures for investigating the behavior and performance of plastic modified asphalt pavement (Vasudevan et al. 2011; Hınıslıoğlu and Ağar, 2004; Movilla-Quesada et al. 2019). Previous literatures conducted studies using either mixed recycled plastics or only certain types of plastic. Moreover, there is no literature found where the effects of recycled plastic investigated while using recycled aggregate such as recycled asphalt pavement (RAP) instead of virgin aggregate. In USA, the use of recycled plastics for asphalt pavement is very limited. The reasons might be the lack of appropriate mix design and standard; lack of study on the type of plastics that can be used to modify the bitumen and lack of performance evaluation of plastic modified asphalt pavement.

It is necessary for engineers to understand the behavior of plastic modified asphalt pavement to carry out sustainable and economical design and construction. Therefore, it is important to evaluate the effect of different plastic types (e.g. LDPE, HDPE, PP etc.) for effective application in reducing pavement distresses.

1.3 Objectives

The objective of the current research study is to evaluate the potential reuse of recycled plastics for plastic road design. As a part of the research objective, optimum mix design for plastic modified bitumen and asphalt will be determined. The specific task to accomplish the objective of the study include:

a. Collection, Sorting and Cleaning of Recycle Plastic
b. Shredding of recycled plastic
c. Aggregate and bitumen collection
d. Development of an experimental program for optimum mix design
e. Determination of volumetric characteristics for the mix design
f. Evaluation of the performance of the mix design
g. Propose optimum plastic content
h. Development of statistical correlations between the Rutting and IDT so that the rutting depth can be determined from faster, easier, and less expensive tests, such as the indirect tensile strength (IDT) test.

1.4 Dissertation Organization

The dissertation is organized into six chapters. The summary of each chapter is presented as follows:

Chapter 1 presents the background, problem statement, and research objectives of the current study. The contents of each chapter are also summarized.

Chapter 2 presents a literature review on previous studies conducted on recycled asphalt materials and recycled plastic use in pavement. A brief overview of recycled plastic situation in the world. It also provides a glimpse of the performance of plastic modified bitumen mix analyzing different studies and test results of rutting, cracking and moisture susceptibility.

Chapter 3 describes the experimental program and preparation of recycled plastic; several sample preparation and test procedures, such as Bulk density test, Rice gravity test, Hamburg wheel tracker test, Indirect tensile strength test, Overlay test and Moisture susceptibility test.

Chapter 4 presents test results, analysis and discussions of the results.

Chapter 5 provides a description of the multiple linear regression analysis procedure and development of a statistical model to determine the value of rutting depth, using indirect tensile strength, RAP content, Plastic type and plastic content.

Chapter 6 summarizes the major conclusions from laboratory test results and statistical analysis. Finally, recommendations for further studies are presented.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Good road infrastructure is a vital requirement for the social and economic development of any country. The goal of roads is to provide durable and long lasting pavements to improve riding comfort and safety, as well as to reduce maintenance costs. This can be achieved by providing good structural pavement design as well as good asphalt mixture design (Naghawi et al, 2018). Throughout the years, numerous studies have been conducted to improve asphalt mixture design for better performing pavements (Cao, 2007; Onyango, 2015; Jain et al, 2011).

2.2 Pavement structure

The basic function of a pavement is to reduce the stress to a tolerable level for the sub- grade. A typical pavement structure consists of several layers where every layer convey load from the upper layers and distribute them to the lower layers. The ultimate purpose of the upper layers is to ensure that the transmitted stresses due to wheel load do not exceed the bearing capacity of the sub-grade. In general, there are two types of hard-surfaced pavement: flexible and rigid (Figure 2.1). Flexible pavement generally consists of a prepared or stabilized sub-grade, base or sub-base course, and surface course which is surfaced with bituminous (or asphalt) materials. On the other hand, Rigid pavement consists of a prepared sub-grade, base or sub-base course, and a pavement slab. A pavement slab is usually composed of a Portland cement concrete (PCC) settles uniformly under loading. The modulus of elasticity of flexible pavement is very low meaning less strength. In contrast with flexible pavements, rigid pavements have a very high modulus of elasticity due to high strength concrete and more load bearing capacity of the pavement itself. As a result, asphalt concrete pavements must be durable as a basic and important requirement. In asphalt concrete, durability refers to its ability to withstand traffic, temperature changes, and the action of air and water.

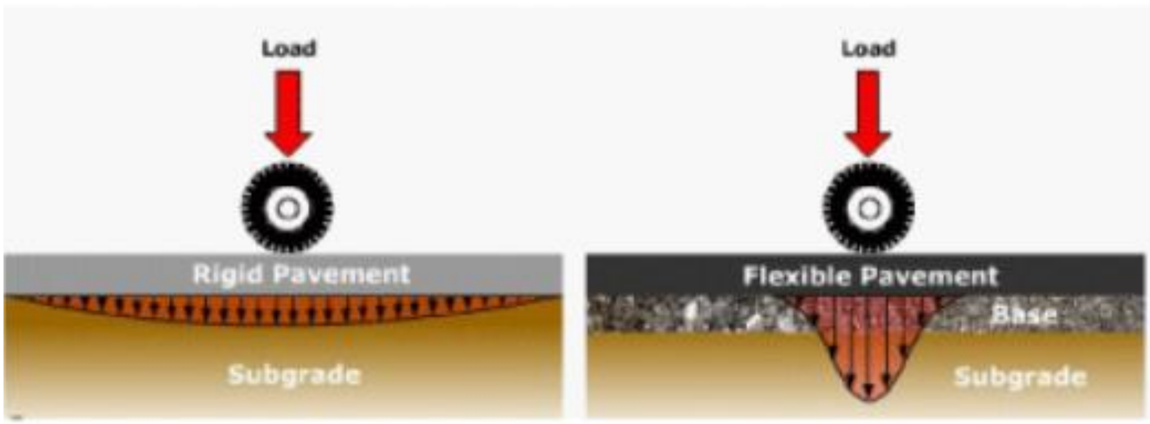


Figure 2.1. Typical stress distribution under a rigid and a flexible pavement (TxDOT Manual)

2.3 Flexible Pavement and Its Structure

The term "flexible pavement" describes a type of pavement that is composed of bituminous mixtures, aggregates (coarse and fine), compacted granular materials, and layered over subgrades of proper quality. The coarse aggregate can be crushed stone, while the fine aggregate is usually sand. Bitumen is made from tar, which is a derivative of fractionally distilled natural oils. In contrast to rigid pavements, these pavements are designed for low-volume traffic loads. Due to the fact that they are spread over an increasingly larger area and carried deep into the ground through successive layers, these pavements have a stress distribution that gradually recedes as the load is transmitted downwards. Several factors influence the performance of flexible pavement, including the properties of the components (binder, aggregate, and additive) and the proportion of each component to the mix. Asphalt mixtures can be improved by incorporating a variety of additives, such as polymers, latex, fibers, and chemical additives (Taih, 2011; Awwad & Shabeeb, 2007). Figure 2.2 shows a typical section of flexible pavement.

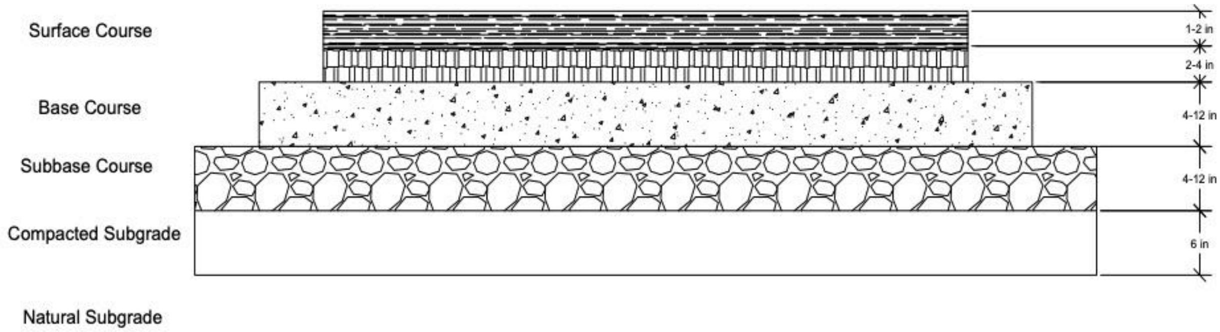


Figure 2.2. Typical section for a flexible pavement (Imtiaz, 2021)

2.4 Types Of Bituminous Mix

Dense-Graded Mixes

Bituminous concrete with a dense mix is composed of all constituents in good proportion. It provides durability by offering offers good compressive strength as well as some tensile strength.

Gap-graded mix

There are some large coarse aggregates missing but they are very strong and have a good fatigue and tensile strength.

Open-graded mix

It lacks fine aggregates and fillers, is porous, provides good friction and has low strength.

Hot mix asphalt concrete

The most widely used paving material in the world is hot-mix asphalt (HMA). The material is also known as asphaltic concrete, plant mix, bituminous mix, or bituminous concrete. Aggregates and asphalt binder are its two primary components. Aggregates consist of coarse and fine materials, usually a mix of different sizes of rock and sand. By weight, aggregates account for approximately 95% of a mixture. HMA is produced by mixing them with about 5% asphalt binder (Speight, 2016). Prior to mixing the asphalt binder with the aggregate, the asphalt binder must be heated to reduce its viscosity and the aggregate must be dried to remove moisture. In most cases, virgin asphalt is mixed at 150°C with the aggregate.

Warm mix asphalt

Zeolite wax, asphalt emulsions, or sometimes even water are added to the asphalt binder prior to mixing to make it. Thus, mixing and laying temperatures can be lowered significantly, which reduces the consumption of fossil fuels, releasing less carbon dioxide, aerosols, and vapours.

Cold mix asphalt

It is manufactured by emulsifying asphalt in water before mixing with aggregates. In addition, the mixture has a lower viscosity and is more easily compacted and convenient to work with. Cold mix asphalt acts like cold HMA after evaporation of water breaks the emulsion.

Cut-back asphalt concrete

The binder is dissolved in kerosene or some other lighter fraction of petroleum, making the asphalt less viscous and easier to work and compact. The lighter fraction evaporates after laying of the mix. Due to concerns about pollution caused by volatile organic compounds in the lighter fraction, cut-back asphalt has been largely replaced by asphalt emulsion.

Mastic asphalt concrete

In order to produce mastic asphalt, hard grade blown bitumen (oxidation) is heated until it has transformed into a viscous liquid in a green cooker (mixer) before it is mixed with aggregates. Following that, the bitumen aggregate mixture is cooked (matured) for approximately 6-8 hours and then the mastic asphalt mixer is transported to the job site where it is generally laid with a thickness of between 3⁄4–13⁄16inches (20-30 mm) for footpaths and roads and around 3⁄8 of an inch (10 mm) for flooring or roofs.

2.5 Constituents of a Mix

A bituminous mix is formed by mixing aggregates continuously grading in size, usually less than 25 mm, up to 0.075 mm fine filler. Adding enough bitumen to the mix ensures that the compacted mix is effectively impermeable, dissipative, and elastic. Designing a bituminous mix involves determining what proportions of bitumen, filler, fine aggregates, and coarse aggregates are best for a mixture that is workable, durable, and economical.

2.5.1 Aggregate

Hard, inert materials such as sand, gravel, crushed rock, slag, or rock dust are called aggregates (or mineral aggregates). HMA pavements are made of properly graded and selected aggregates mixed with an asphalt binder. Aggregates are the primary load-bearing component of HMA pavements. Since dense-graded HMA pavements are 95% built up of aggregate, the characteristics of the aggregates have a major impact on a pavement's performance HMA aggregates can be classified into three types based on their size: coarse aggregate, fine aggregate, and mineral filler. In general, coarse aggregates are those retained on the 2.36-mm sieve. An ideal coarse aggregate is screened crushed rock that is angular, free of dust particles, clay, vegetation, and organic matters, has good compressive and shear strengths, and is stable under load. Fine aggregates are those that pass through the 2.36-mm sieve and are retained on the 0.075-mm sieve. A fine aggregate should be screened quarry dusts that are free of clay, loam, vegetation, and organic matter. It fills the voids in the coarse aggregate and stiffens the binder. Mineral filler is the portion of aggregate that passes through a sieve of 0.075 mm. Mineral filler material, also known as mineral dust or rock dust, is a very fine, inert mineral that is added to hot mix asphalt in order to enhance its density and strength. The gradation of the combined aggregate shall be incorporated into it (Chen, 2009; Transportation research board committee, 2011). Filling voids, stiffening the binder and allowing permeability are its main functions. According to Vasudevan et al. (2011), aggregates are chosen based on their strength, porosity, and moisture absorption capabilities.

2.5.2 Asphalt Binder (Bitumen)

Asphalt binder (bitumen) acts as a binder for aggregates, fines, and stabilizers in bituminous mixtures. Bitumen is a viscous material extracted from crude petroleum, which is used in the paving of roads to bind aggregates together. Asphalt is mostly composed of hydrocarbon molecules (hydrogen and carbon). Small amounts of oxygen, sulfur, and nitrogen also contribute to the binder (Hadidy and Yu, 2008). Since bitumen exhibits both viscous and elastic properties at normal pavement temperatures, it must be treated as a visco-elastic material. Temperature affects the physical properties of asphalt binder significantly. Asphalt binder is a fluid with a low consistency similar to oil at high temperatures A majority of asphalt binders will have the consistency of soft rubber when stored at room temperature. Asphalt binder can become extremely brittle at subzero temperatures. In many asphalt binders, small percentages of polymer are used to improve their physical properties; these materials are known as polymer modified binders. Asphalt binder specifications were primarily developed to maintain consistency with temperature (Transportation research board committee, 2011).

2.5.3 Reclaimed Asphalt Material

In today's asphalt industry, reusing reclaimed asphalt materials like reclaimed asphalt pavements (RAP) and reclaimed asphalt shingles (RAS) to create new pavements is essential RAP and RAS are used in hot-mix asphalt (HMA) because they provide economic and environmental benefits. As asphalt binder costs have increased (because of the global rise of oil price) and high quality virgin aggregates are in short supply, demand for RAP and RAS has steadily increased in recent years. More than 99 percent of RAP is being reused in new pavements (NAPA, 2011). By using RAP and RAS in 2010, approximately 20.5 million barrels of asphalt binder were conserved (NAPA, 2011). Use of RAP and RAS in 2010 conserved approximately 20.5 million barrels of asphalt binder (NAPA, 2011). RAP, which contains an average of 5 percent asphalt binder, is an excellent source of asphalt binder and high quality aggregates for new HMA.

Recycled Asphalt Pavement (RAP)

As RAP has become more commonly used by the asphalt industry, DOTs have realized they need to update their specifications and testing protocols since more laboratory and field data are needed on asphalt mixes containing RAP. Jones (2008) states that more than twenty DOTs, including Texas, Louisiana and Arkansas, allow RAP of 30 percent or more in base courses and 10 percent or more in surface courses. In many other DOTs, the RAP is limited to 25 percent in the base course and none in the surface course (FHWA, 2009; ODOT, 2009 Jones (2008) found through a survey that stockpile management issues, binder issues, and mix issues are the leading barriers to increasing RAP percentages in asphalt mixes. Managing stockpiles is a challenging task due to unknown original quality, gradation control challenges, and processing requirements. Issues with binding substances consist of bumping binder grades, unknown properties of final blends, and compaction problems. There are unknown performance and durability characteristics of mix materials, additional testing requirements, variability of RAP mixes, and concerns about early failure of mixes. Therefore, the performance of asphalt mixes containing RAP needs to be extensively investigated in the laboratory and in the field..

Recycled Asphalt Shingles (RAS)

The use of RAS in HMA is both economically and environmentally beneficial. By incorporating RAS in HMA, the need for virgin materials, such as asphalt and aggregates, will be reduced (FVD, 2006; Sengoz and Topal, 2005; Foo et al., 1999). In RAS, a source of fine aggregate is found in an amount from 19 to 36 percent, whereas asphalt binder is found in an amount from 20 to 38 percent (CIWMB, 2007; NAHB, 1998). As far as the environment is concerned, RAS will reduce landfill usage and virgin material consumption (Sengoz and Topal, 2005 As per the results of a survey conducted by NAPA (2011), between 2009 and 2010, manufacturers' waste and tear-offs went from 702,000 to 1.1 million tons, an increase of 57 percent. This would represent 234,000 tons of asphalt binder conservation (1.5 million barrels) if 20 percent of the binder was contributed by the shingles (NAPA, 2011).

In several studies, RAS has been shown to be technically feasible in HMA (Sengoz and Topal, 2005; Rajib et al., 2000; Foo et al., 1999; NAHB, 1998; Ali et al., 1995; Button et al., 1995; Grzybowski, 1993). The use of RAS in HMA has also been shown to improve the mechanical properties of pavements, in addition to its economic and environmental benefits. Studies have shown that mixes containing RAS are more rutting resistant, fatigue-resistant, and perform better overall than conventional asphalt mixes, even when moisture is accounted for (Baaj, 2007; Ali et al., 1995; Grzybowksi, 1993). Due to its potential benefits, RAS is expected to become a significant part of recycling in the asphalt industry.

Although reclaimed asphalt and reclaimed aggregate content are reported to improve rutting performance of pavements, contradictory findings have been reported regarding fatigue life and thermal cracking of mixes with reclaimed asphalt and reclaimed aggregate (Huang et al., 2004; McDaniel and Shah, 2003; McDaniel et al., 2000). In this respect, there is a need to investigate the effects of using RAS on dynamic modulus, fatigue life, and thermal cracking of mixes containing local aggregates.

2.6 Desirable Properties of Asphalt Mixes

HMA product designs are aimed at achieving a set of properties. In addition to these properties, the aggregate characteristics include gradation, texture, shape, and chemical composition, as well as asphalt binder content, asphalt binder characteristics, and compaction degree. It is important to understand the properties of HMA constituents individually, however HMA mixture behavior is best explained by considering how asphalt cement and mineral aggregate act together. In order to prevent permanent deformation within the mixture, the HMA must be internally strong and resilient. Additionally, the material must also have sufficient tensile strength to sustain the stress at the bottom of the asphalt layer and must also be resilient enough to withstand a wide range of load applications without fatigue cracking. It must also resist the stresses imposed by rapidly declining temperatures and exceptionally cold temperatures.

A brief description of some desirable properties of asphalt mixes is presented below (Wayne et al., 2006):

a. Resistance to permanent deformation: It is important not to allow the mix to distort or shift when subjected to traffic loads, especially during high temperatures and long loading times.

b. Durability: Weathering (air and water) as well as traffic abrasion must be resistant to the mix. To guarantee adequate film thickness around aggregate particles, asphalt mix must contain enough asphalt cement.

c. Fatigue resistance: In the long run, repeated loads on the mix should not crack it.

d. Skid resistance: Under wet weather conditions, the mix must be sufficiently skid-resistant. Skid resistance is determined by the aggregate properties such as texture, shape, and size.

e. Workability: In order to place and compact the mix to specific density, reasonable effort is required.

f. Moisture damage resistance: Moisture penetration into the HMA mix should not significantly degrade the mix by reducing adhesion between aggregate and asphalt.

g. Low noise and good drainage properties: The pavement structure's wearing layer should possess this property.

h. Resistance to low temperature cracking: It is important for cold climates to have this mix property.

2.7 Asphalt Limitations in Paving Industry

The high summer temperatures soften the asphalt binder, thereby reducing the stiffness of the paving mixture Due to the low temperatures in winter, the asphalt binder becomes stiff and the paving mixture becomes less flexible. As a result, in summer pavement rutting and in winter thermal cracking of the pavement surface may develop and adversely affect the performance of the paving mixture, resulting in frequent and costly repairs. As a result, there can be rutting in summer and thermal cracking in winter, resulting in more frequent and expensive repair work.

2.8 Pavement Mix Design

HMA mix design involves determining how much aggregate, asphalt binder, and what blend of the two should be used. A laboratory simulation is used during mix design. To the extent possible, it attempts to simulate actual HMA manufacturing, construction, and performance. Using this simulation, the type of mix design that would be best for the particular application can be determined (with reasonable certainty).

2.8.1 Hveem Mix Design

Francis Hveem developed the basic concept of the Hveem mix design method during his time as a Resident Engineer for the California Division of Highways in the late 1920s and 1930s. At present, several western states are using the Hveem method. The following three points represent the basic philosophy behind the Hveem method (Vallerga and Lovering, 1985):

a. Adequate asphalt binder is needed to coat each aggregate particle to an optimum film thickness (allowing for the asphalt to penetrate the aggregate).
b. It must be stable enough to resist traffic loads. The stability of the aggregate is achieved through friction between individual particles and cohesion (or tensile strength) provided by the binder.
c. The durability of HMA increases with a thicker asphalt binder film.

This philosophy dictates that the design asphalt content be selected based on the most durable asphalt content which does not fall below a minimum level of stability. Therefore, the minimum stability requirements should be met while using the maximum amount of asphalt binder.

Hveem mix design is mainly conducted according to AASHTO T 246 and AASHTO T 247. The Hveem mix design method consists of 6 basic steps: Aggregate selection, Asphalt binder selection, Sample preparation and compaction with California Kneading compactor (Figure 2.3a), Stability determination using the Hveem Stabilometer (Figure 2.3b), Density and voids calculations, Optimum asphalt binder content selection.

(a) (b)

Figure 2.3. (a) California Kneading Compactor (b) Hveem stabilometer

2.8.2 Marshall Mix Design

Marshall mix design methods were created by Bruce Marshall of the Mississippi Highway Department in 1939 and then refined by the U.S. Army. Approximately 38 states use the Marshall method in some capacity. By using the Marshall method, a suitable asphalt binder content is selected at a density that meets a minimum amount of stability and a range of flow values (White, 1985). Even with its shortcomings, the Marshall method is probably the most widely used mix design method in the world today.It has probably become so widely used because

a. Instead of stressing a portion of the sample, the entire sample was stressed.
b. Minimal effort was required for rapid testing.
c. It was compact, lightweight and portable.
d. It produced densities that are similar to those found in the field.

Marshall mix design procedure is conducted according to AASHTO T 245. The Marshall mix design method consists of 6 basic steps: Aggregate selection, Asphalt binder selection, Sample preparation and compaction with Marshall hammer, Stability determination using the Marshall

Stability testing apparatus (Figure 2.4), Density and voids calculations, Optimum asphalt binder content selection.

Figure 2.4. Marshall Stability apparatus

Using the Marshall stability and flow test, the Marshall mix design method can predict performance. As part of the stability test, the test specimen is loaded at a rate of 50.8 mm per minute (2 inches per minute), to measure the maximum load it can support. This is accomplished by increasing the load until it reaches a maximum, then stopping the loading just as the load begins to decrease to record the maximum load.

The dial gauge connected to the loading device measures the plastic flow caused by the loading. At the same time that the maximum load is recorded, the flow value is recorded in 0.25 mm increments.

2.8.3 Superpave Mix Design

As part of the Strategic Highway Research Program (SHRP), a new method of mix design that accounts for traffic load and environmental conditions was developed, along with new methods for evaluating asphalt binder quality. As part of SHRP, these three developments were called the

Superior Performing Asphalt Pavement System (Superpave). As a replacement for Hveem and Marshall, the Superpave mix design method has been developed. Volumetric analysis is the basis for the Superpave mix design method, which is similar to both Hveem and Marshall methods. A Superpave mix design takes traffic and climate into account as well as asphalt binder and aggregate selection. Unlike the Hveem and Marshall procedures, the compaction devices from the mix design use a gyratory compactor, whose compaction effort varies with expected traffic.

The Superpave mix design method consists of 6 basic steps: Aggregate selection, Asphalt binder selection, Sample preparation and compaction with Superpave Gyratory compactor, Density and voids calculations, Optimum asphalt binder content selection and Moisture susceptibility evaluation.

Aggregate Selection:

Several key HMA parameters are affected by aggregate gradation, including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance and resistance to moisture damage (Roberts et al., 1996). Furthermore, the maximum aggregate size is important when determining compaction and lift thickness. During the aggregate gradation design of superpave mixes, control points are specified through which aggregates must pass. The aggregate specification for Superpave hot-mix asphalt (HMA) mixtures includes a restricted zone that lies along the maximum density gradation between the intermediate size (i.e., either 4.75 or 2.36 mm, depending on the nominal maximum size of the aggregate) and the 0.3-mm size (NCHRP Report). It was recommended that gradations not pass through the restricted zone (Figure 2.5). Superpave adopted the restricted zone requirement in an effort to reduce the incidence of tender or rut-prone HMA mixes.

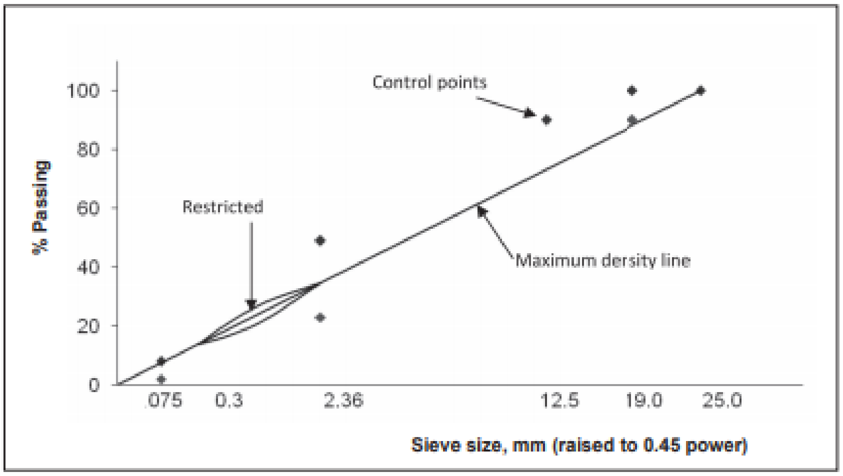


Figure 2.5. Superpave specified gradation for 19.0 mm nominal size (Mampearachchi and Fernando, 2012)

Asphalt Binder Selection:

In contrast to the older system of viscosity-graded binders, which are typically used for surface treatments and aggregate precoating, the Superpave binder specifications are performance-based, hence they are called performance-graded (PG) binders. In the PG binder system, engineering properties believed to be related to the expected performance are measured at temperatures corresponding to the climatic and traffic conditions (maximum 7-day pavement temperature, minimum pavement temperature, loading duration based on truck speed, and traffic volume) of the pavement location. Hence, a binder grade suitable for a particular highway application may be selected. Binder grades are determined by climate parameters, as shown below (Figure 2.6):

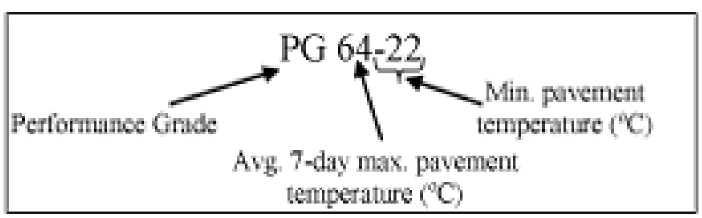


Figure 2.6. Performance Grade Binder (TxDOT manual)

Superpave Gyratory Compaction:

The standard gyratory compactor sample preparation procedure is AASHTO TP4. The Superpave gyratory compactor (Figure 3.14) establishes three different gyration numbers:

a. $N_{initial}$: A measure of the compactability of a mixture during construction is the number of gyrations. Mixes that compact too quickly (air voids at $N_{initial}$ are too low) may be tender during construction and unstable when subjected to traffic. An HMA with excess natural sand will often fail the $N_{initial}$ criteria - it is a good indicator of aggregate quality. A mixture designed for greater than or equal to 3 million ESALs with 4 percent air voids at N_{design} should have at least 11 percent air voids at $N_{initial}$.

b. N_{design}: The design number of gyrations is the number of gyrations required to produce a density that is similar to that of the actual test field after the indicated amount of traffic. When designing the mix, it is desirable to have an air void content of 4 percent at N_{design}.

c. N_{max}: It should never be exceeded in the field the number of gyrations required to produce a laboratory density. If the air voids at N_{max} are too low, then the field mixture may compact too much under traffic resulting in excessively low air voids and potential rutting. N_{max} should never have a void content below 2 percent.

Typically, samples are compacted to N_{design} in order to determine the optimum asphalt binder content followed by compacting additional samples to N_{max} as a check. Previously, samples were compacted to N_{max} and then $N_{initial}$ and N_{design} were back calculated.

Figure 2.7. Superpave Gyratory Compactor

Density and Voids Analysis:

All mix design methods use density and voids to determine basic HMA physical characteristics. Two different measures of densities are typically taken:

a. Bulk specific gravity (G_{mb}).

b. Theoretical maximum specific gravity (TMD, G_{mm}).

These densities are then used to calculate the volumetric parameters of the HMA. Measured void expressions are usually Air voids (V_a), sometimes expressed as voids in the total mix (VTM), Voids in the mineral aggregate (VMA) and Voids filled with asphalt (VFA).

2.9 Asphalt Pavement Distress

Many of the properties of bitumen make it unsuitable for pavements, and it can cause traffic disruptions. Bitumen is derived from fossil fuels, which are disastrous for the environment. The bleeding of roads occurs at high temperatures, reducing surface performance. In some cases, bitumen can crack as a result of chemical reactions, like oxidation. The water repellent properties of bitumen in action with water cause the bitumen to peel off the aggregate on roads, causing potholes. Below are the main types of asphalt pavement distress that engineers endeavor to avoid when designing new roads.

2.9.1 Permanent Deformation/Rutting

Whenever a surface cross section no longer conforms to its design position, it is said to have undergone permanent deformation. Permanent deformation is defined as an accumulation of small amounts of deformation that occur each time a load is applied. This deformation cannot be recovered. Rutting of the wheel path is the most common example of permanent deformation. Although rutting has many causes (e.g., moisture damage, abrasion, and traffic density), there are two primary causes. Occasionally, rutting may occur due to repeated repeated stress applied to the subgrade (or subbase) under the asphalt layer, or it may occur in the asphalt surface course (Figure 2.8). It may also be caused by moisture invading the pavement layer unexpectedly Asphalt layers deformation is the type of rutting that most concerns asphalt designers. Ruthing is caused by a lack of shear strength in the asphalt mixture to withstand repeated heavy loads. Every time a truck passes over a weak mixture, small but permanent deformations occur. Eventually, a rut forms, characterized by downward and lateral movement (Figure 2.9).

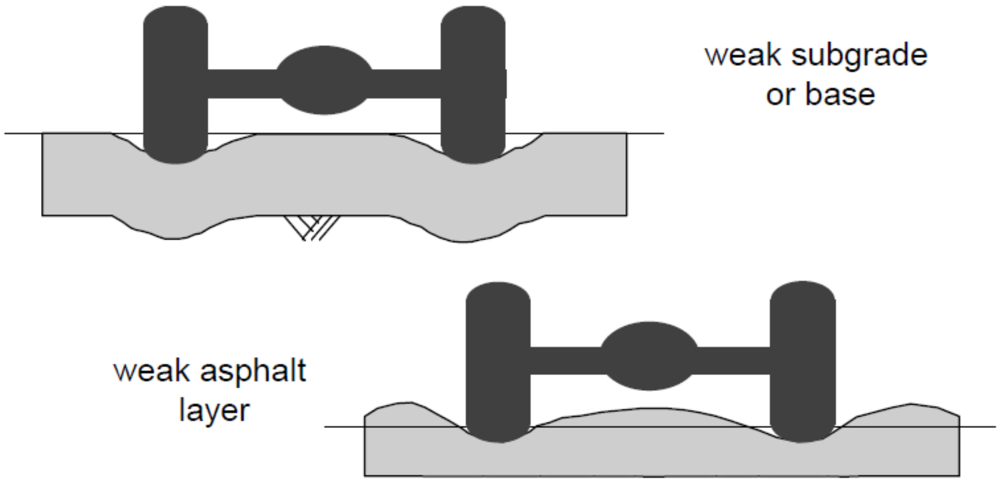


Figure 2.8. Rutting mechanism (FHWA manual)

Figure 2.9. Rutting of pavement (Sarah et al 2017)

2.9.2 Fatigue Cracking

When the asphalt material is overstressed, fatigue cracking occurs. Intermittent longitudinal cracks appear in the wheel path of the vehicle as an early indication of fatigue cracks. In fatigue cracking, the initial cracks join together at some point, leading to the formation of more cracks. Fatigue cracking can also be characterized as alligator cracking, characterized by transverse cracks joining longitudinal cracks (Figure 2.10 During heavy traffic, pavement pieces can become dislodged, causing a pothole to form in extreme cases. Many factors cause fatigue cracking at the same time. The most obvious factor is repeated heavy loads. Weak underlying layers or thin pavements are prone to deflection under heavy vehicle loads. Due to high deflections, horizontal tensile stresses are increased at the base of the asphalt layer, causing fatigue cracks to form. This problem is often caused by poor drainage, faulty construction, or an under-designed pavement.

Figure 2.10. Fatigue cracking

2.9.3 Low Temperature Cracking

In contrast to traffic loads, adverse environmental conditions are responsible for low temperature cracking Interestingly, there are intermittent transverse cracks with very consistent spacing (Figure 2.11). Asphalt pavement layers shrink in cold weather, resulting in low temperature cracks Tensile stresses build within the pavement layer as it shrinks The asphalt layer starts cracking at a certain point along the pavement when the tensile stress exceeds the tensile strength. Most low temperature cracks develop after a single cycle of low temperatures but can also develop after repeated cycles of low temperatures. Low temperature cracks are caused by the asphalt binder. Asphalt binders made of hard materials are more susceptible to cracking at low temperatures than asphalt binders made of soft materials. Low temperature cracking is more common with asphalt binders that are excessively old; they are unduly susceptible to oxidation and/or contained in a mixture constructed with too many air voids. Therefore, in order to prevent low temperature cracking, engineers must use a binder that is soft, does not age rapidly, and controls the air void content and pavement density so as not to excessively oxidize the binder.

Figure 2.11. Low Temperature Cracking (Ahmad and Khawaja, 2018)

2.9.4 Bleeding

The road becomes slippery when water absorption is high for a bituminous surface. During the summer, excessive bleeding of bitumen is another factor that makes roads slippery. Water can cause bleeding by seeping out of joints or cracks, or by penetrating a highly porous HMA layer (Figure 2.12). As a result of the prolonged performance of the road, the softening point of plain bitumen is reduced, which in turn results in over bleeding and smoothening of the road surface, resulting in a higher skid number.

Figure 2.12. Bleeding on pavement (Lawson et al, 2007)

2.9.5 Moisture Damage

Stripping is a term commonly used to describe damage caused by moisture. Asphalt is stripped off of the aggregate surface as a result of the damage. The adhesive bond between aggregate surfaces and asphalt cement is broken during stripping (Fromm, 1974 Since bitumen is water repellent, bitumen strips away from aggregates causing potholes on roads (Figure 2.13). Since bitumen is water repellent, bitumen strips away from aggregates causing potholes on roads (Figure 2.13). Weak adhesive and cohesive properties are the mechanistic result of moisture damage. Due to moisture damage, asphalt pavement will experience shoving, rutting, and fatigue cracking (Ping and Kennedy, 1991).

Figure 2.13. Moisture damage

2.10 Background of asphalt mix modified with waste products

Better service from paving materials is crucial today because of the increasing vehicle loads, heavier volume of traffic, and the need for longer-lasting roads. Some improvements have been made in asphalt properties by tailoring the refinery processes and/or selecting the best petroleum product, and one manufacturer modified the asphalt to enhance its properties. Studies have suggested that there are ways to make roads more durable. These alternatives mostly use industrial waste, which is difficult and expensive to manage. However, reusing them as a modifier of other structures can be a good measure of avoiding the waste problem (Asutosh and Nawari, 2017). For instance, ash, glass, scrap tire, plastic, etc. are used to make new innovations for road construction and rehabilitation.

2.10.1 Ash

Fly ash, bottom ash, pond ash, and magnetherm slag are common industrial wastes that are being used in building and road construction and have gained increased acceptance over a period of many years. Some countries like India have already devised guidelines for using ash in the construction of pavement.

Ali et al. (1996) found that the addition of fly ash enhances the stripping resistance of the mix. Since large quantities of fly ash are produced from the direct combustion of the Jordanian oil shale, which is regarded as a hazardous material to the environment, Asi and Assad, 2005 conducted research to learn how to use it to modify asphalt. They found that because of the pozzolanic cementing properties, asphalt mixed with fly ash has superior properties and is more resistant to water damage. This improvement was proportional to the fly ash content. Moreover, the use of fly ash up to 4% in a dense-graded bitumen mix showed a reduction of 7.5% in the optimum bitumen content compared to the control mix (Mistry and Roy, 2016).

2.10.2 Waste Glass

Studies show that adding fine waste glass to fine aggregates could also produce promising results (Ismail, 2008). It was found that asphalt does not adhere to glass as well as it does to aggregates, which means that glass finer than what can pass through a 3/8 inch sieve can be used up to15% by volume of the total aggregate. A few additives, including hydrated lime, were introduced to reduce the adhesion problem, and the results were promising. To eliminate the adhesion problem for the glass and asphalt, many other anti-stripping agents were also uncovered. This led to the use of waste glass on low volume roads with the help of binders. Glass can be used in both flexible pavements and in rigid pavements, and can be a very efficient substitute in rigid pavements, where concrete is the main element of the roadway. The waste glass shows a lot of promise when used in concrete in numerous forms, including coarse aggregate and fine aggregate (Ahmed, 2011). According to an experiment at the University of Baghdad, the 28-day compressive strength value of a concrete mix made of 20% waste glass fine aggregate had a value of 45.9 MPa, which represents an increase of 4.23% in the compressive strength, compared to the control mix. After 28 days, the pozzolanic effect of waste glass in concrete became more evident. When 20% of waste glass was used, it gave the maximum values of compressive and flexural strengths. Using finely ground waste glass rather than fine aggregate could produce promising results, assuming that the geometry is less heterogeneous (Ismail, 2008). The specific gravity values were found to be approximately 10% lower than the values of natural aggregate reported by Das (2007).

2.10.3 Scrap-tire Rubber

The use of scrap tires in the construction of pavements has been established as an effective measure (David et al. 1992; Costa et al, 2013). Scrap tires are typically used in wet and dry processes where they act as a binder or replacement aggregate in asphalt mixtures, respectively (Moreno et al., 2012). The modification of an asphalt cement binder with 5 - 25% by weight of fine tire rubber crumb modifier (CRM) at an elevated temperature refers to the wet process (Cao, 2007). Souza and Weissman (1994) used 15% rubber content as a binder, which showed an improved performance in dynamic stability, flexural strength, and strain value. Studies have also been performed on modifying the properties of asphalt mixtures with recycled tire rubber, using the dry process, to minimize the pollution caused by waste tires and improve the properties of asphalt mixtures. Given the investigative consequences of the Marshall test, rutting test, and indirect tensile test, the inclusion of 3% (by weight of absolute mix) of tire rubber in asphalt blended by the dry process could enhance the protective properties and reduce pavement distortion and low temperature cracking (Cao, 2007).

2.10.4 Waste/recycled Plastic

The disposal of plastic in landfills is dangerous to the environment, but using it in bituminous road construction would provide a way to eliminate that hazard by recycling. Numerous researchers are analyzing the ecological stability and functional capacity of recycled items in different construction scenarios, and there is no doubt that utilizing the plastic waste in flexible pavement construction would accelerate the removal of huge amounts of plastic from the landfills.

Since Professor Vasudevan's achievement, the process has been practically put to test in the streets of India and has been deemed very successful (DNA India, 2010). The Indian government developed 21,000 miles (33,000 kilometers) of streets, utilizing reused plastic in 2017; however, a large majority of them were built in countryside regions (Louise, 2019).

2.11 PLASTIC GRADES

In accordance with the Society of the Plastics Industry (SPI), there are seven types of plastic. A classification system was created by SPI in 1988 so that consumers and recyclers could distinguish between different types of plastic Each plastic product is equipped with an SPI code, which is usually molded into the bottom. Following is a brief overview of the types of plastics associated with each of the code numbers described in this guide.

Grade 1. Polyethylene terephthalate (PET or PETE)

PET is tough, transparent, and has good barrier properties against gases and moisture. It is usually used in soft drink bottles. Foods and beverages stored inside these containers tend to absorb odors and flavors. This plastic is used for a variety of household appliances and everyday essentials.

Grade 2. High Density Polyethylene (HDPE)

HDPE products are typically recycled. These plastics are used to make milk containers, motor oil containers, shampoo bottles, detergent bottles, and bleach bottles. A HDPE bottle should not be used as a food or drink container if it did not originally contain any edible material. This is because of the risk of contamination.

Grade 3. Polyvinyl Chloride (PVC)

Many everyday objects are made of PVC, though it is primarily used in the plumbing and construction industries. There are major rigid markets for bottles and packaging sheets, as well as in the construction market where it is widely used in pipes and fittings. As a dangerous, toxic chemical, this plastic should not be used for food.

Grade 4. Low Density Polyethylene (LDPE)

Polyethylene is the most common polymer in plastics, since it is made from ethylene monomers. Plastics made from polyethylene are flexible and durable. Food can be stored safely with it because it does not release hazardous chemicals. Many common items made of LDPE include plastic grocery bags, sandwich bags, squeezable bottles, and cling-film.

Grade 5. Polypropylene (PP)

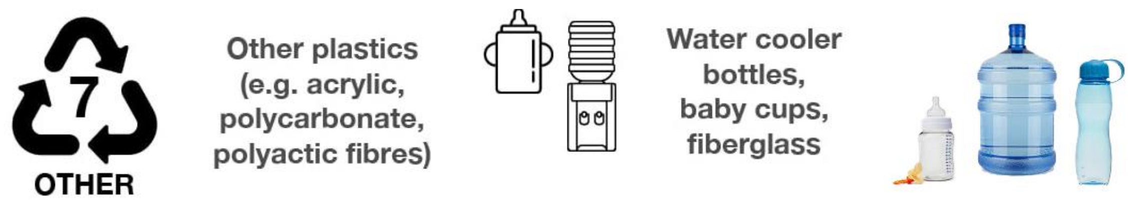
Polypropylene has a high melting point, is chemically resistant, and is strong, making it suitable for liquid hot-filling, as well as packaging for catchups and margarine. There are many uses for it, such as lunch boxes, yogurt pots, syrup bottles, prescription bottles. PP is typically used for plastic bottle caps. PP is a strong plastic that can typically withstand higher temperatures.

Grade 6. Polystyrene (PS)

Polystyrene can be rigid or foamed depending on its structure. Polystyrene is a hard, clear material that is brittle and hard. The melting point is relatively low. Packaging, containers, lids, cups, bottles, trays, and containers can be used as protective packaging.

Grade 7. Other

Miscellaneous plastic types are described with code 7 instead of the other six codes. These include polycarbonate and polylactic acid. Plastics of this type are very difficult to recycle. Polycarbonate (PC) is used in baby bottles, compact discs, and medical storage containers.



2.12 Plastic waste scenario in USA

The use of plastics in many facets of our lives, including packaging, construction and automotive, electric and electronic devices, and automotive, creates considerable amount of solid waste in the world. These are synthetic materials that mostly consist of hydrogen, carbon, and oxygen and are derived from petroleum or natural gas. In addition to their high decomposition temperature, high resistance to UV radiation, and inability to biodegrade, they are typically not biodegradable. As a result, they can remain on both land and sea for years and cause environmental pollution As the plastic usage has increased due to population growth, urbanization, development activities and frequent changes in life style, it has become more difficult to dispose of plastic (Venkat, 2017). There are two ways to dispose of these materials: land filling or incineration, which pollute the atmosphere and land (Prasad et al 2012). Additionally, plastic tends to break down into smaller fragments called macro, meso, and micro-plastics, which have specific and significant effects on ecosystems and can have negative health effects on people and animals due to their chemical structure (Guru et al, 2014).

In municipal solid waste, EPA tracks generation of plastic materials, recycling, composting, burning with energy recovery, and landfilling. The United States generated 35.7 million tons of plastics in 2018, which accounted for 12.2 percent of MSW (Figure 2.14). The United States in 2018 recycled only 8.7 percent of the plastic generated. So the amount of plastic that went for recycling was 3.1 million tons. There were 5.6 million tons of plastics combusted in MSW in 2018. This represents 16.3 percent of all MSW that was combusted in that year. Moreover, 28 million tons of plastic ended up in landfills in 2018. 18.5% of all MSW was disposed of in this manner. Figure 2.2 shows the plastic waste management in USA.

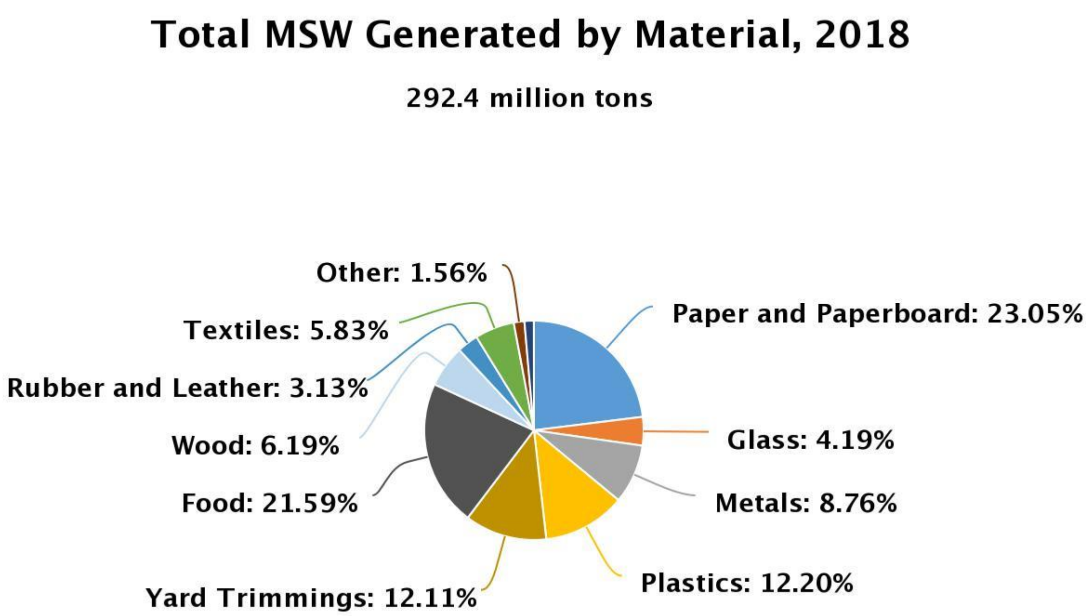


Figure 2.14. Total MSW generation in USA, 2018 (Environmental Protection Agency)

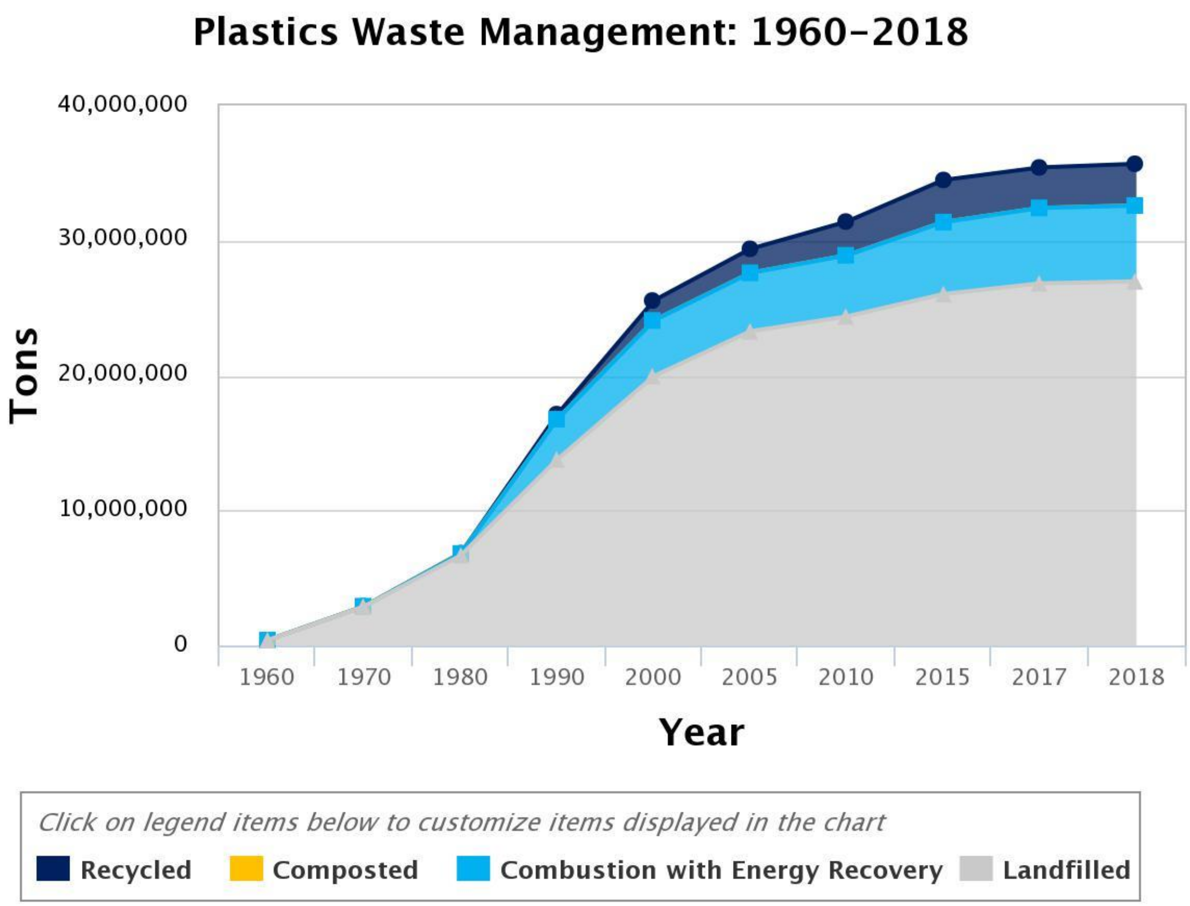


Figure 2.15. Plastic Waste Management (American Chemistry Council)

Waste plastic is one of the fastest growing segments of municipal solid waste (MSW). Containers and packaging, which account for over 14 million tons of plastic in 2015, have the most plastic tonnage of all major MSW categories. This category includes bags, sacks and wraps; other packaging; polyethylene terephthalate (PET) bottles and jars; high-density polyethylene (HDPE) natural bottles; and other containers The volume of recycled plastics is relatively small -- 3.1 million tons for an 8.7 percent rate of recycling in 2018 -- but the amount of plastic containers recycled is substantially higher. For example, PET bottles and jars accounted for 29.1 percent of recycling in 2018, while HDPE natural bottles accounted for 29.3 percent. However, there is very little or no recycling facility for LDPE such as grocery bags, plastic wraps etc and PS (Figure 2.16).

(a) (b)

Figure 2.16. (a) Recyclable plastics (Grade #1, #2 and #5) (b) Non-recyclable plastics (Grade #4, #6 and #7) (Whitehorse Recycling Book, 2020)

Total U.S. plastic waste generation grows 3.8% per year (2015 vs 2014 growth rate from USEPA) resulting this waste generation from 34.5 million tons in 2015 to 38.5 million tons in 2018. Jan Dell, 2018, a chemical engineer, used U.S. Environmental Protection Agency (EPA) data and industry data to estimate the U.S. plastic recycling rate and found that it would sink from 2018 more significantly. Dell, 2018 estimated the recycling rate could drop as low as 2.9 percent in 2019 if plastic waste import bans are adopted by more countries in Asia. Table 2.1 shows the summary of US plastic waste generation and recycling rates.

Plastic pollution is also having a negative impact on our oceans and wildlife health. There have been many instances of marine impacts. By 2050, the oceans will contain more plastic than fish by weight (Jambeck et al, 2015). The United States ranks 20th on the list of countries contributing to plastic pollution in the ocean, with an estimated 88 to 242 million pounds per year of plastic marine debris. The annual International Coastal Cleanup confirmed the evidence of plastic pollution on U.S. coasts in 2017, when more than 3.7 million pounds of trash, the majority of it plastic, was collected by 209,643 people on a single day.

Table 2.1. Summary of US Plastic Waste Generation and Recycling Rates (Dell, 2018)

Plastic Waste	2015 (million tons) USEPA	2015 Actual % USEPA	2018 Projected (million tons)	2018 Projected %	2019 Projected (million tons) (Basel Convention enacted)	2019 Projected % (Basel Convention enacted)
Total Generated	34.5		38.5		40	
Recycled	3.14	9.1	1.68	4.4	1.14	2.9
Composted	0	0	0	0	0	0
Combusted-Energy Recovery	5.35	15.5	5.35	13.9	5.35	13.4
Landfilled	26.0	75.4	31.5	81.7	33.5	83.7

Considering all the adverse effects of plastic it can be concluded that plastics have to be disposed or else it will be hazardous to nature and environment. So one of the best ways of disposal of these plastics is to use in bituminous road construction by melting them. Many researchers are doing various studies on environmental suitability and performance of recycled products in high construction. Use of these waste plastics in bituminous road construction will help in disposal of vast quantities of plastic.

2.13 Plastic Recycling in USA

Plastic usage is increasing gradually and controlling its disposal is very difficult because of the development of urbanization, population growth, and rapid transformations in people's daily lifestyles (Venkat, 2017). They are landfilled or incinerated, neither of which is environmentally friendly, and they pollute our air, land, and water (Prasad et al. 2012). Reusing and recycling plastic would not only reduce the amount of waste in landfills, but also make significant contributions to crude petrochemical savings and energy conservation (EPA, 1991; Solid waste and office water rates, 1990). Rabies and Craft (1995) identified the following technical and financial barriers that might constrain a comprehensive and effective recycling approach to turning plastic waste into new beneficial products: (i) Plastic waste can be contaminated by dirt, dust, and metals that can damage the equipment used in waste recycling; (ii) Plastics are heterogeneous materials, unlike paper and aluminum, and the wide range of types have different melting behaviors, rheology, and thermal stability; (iii) Plastics are generally not soluble in any mixes and form independent phases within a continuous phase; (iv) The raw materials in plastics are not usually identical over time; and (v) Waste plastic has a comparatively low density for which compaction, shredding or grinding are required prior to transportation to decrease shipping and handling expenses. Figure 2.17 presents the most recently reported data for key polymers in the USA, illustrating that PET is the most widely recycled type of polymer.

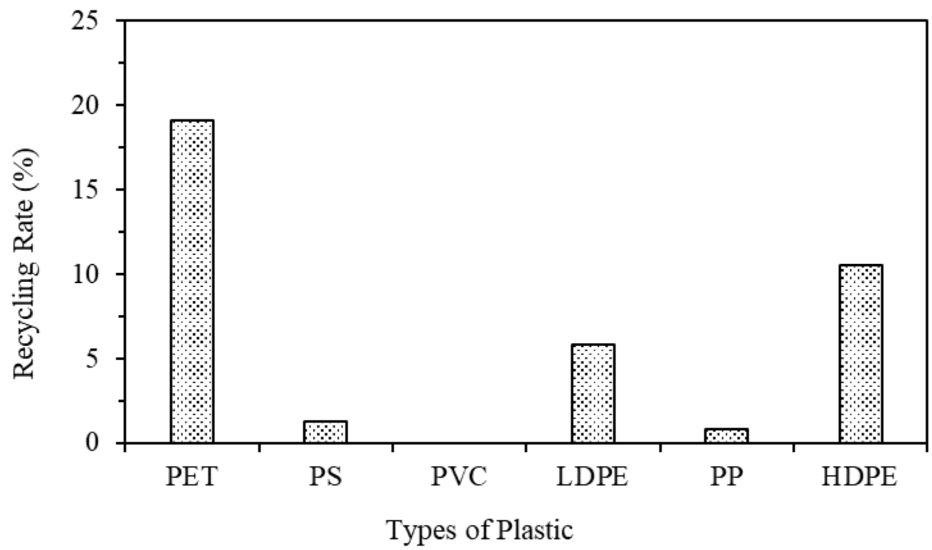


Figure 2.17. Recycling rate of plastics in the USA (Redrawn from Tsakona and Rucevska, 2020)

Based on the previous information, researchers have focused to decrease the amount of plastics that are ended up in landfill by reusing them as a partial substitute of bitumen in the asphalt mix design. Nonetheless, the strength of the asphalt ought not to be undermined.

2.14 Background of Plastic Road

Plastic waste production has been on the rise for a number of years and governments around the world are looking for ways to dispose of the extreme amounts of plastic that litter the streets. Using plastic as a material for the construction of those very streets is a potential solution.

The idea of the plastic road was first devised in early 2000s by an Indian Professor, Dr. Rajagopalan Vasudevan when there were thoughts on banning plastic (Mitra, 2019). He began experimenting with this idea of disposing plastic safely and effectively. During one of his tests he found that plastic can act as a powerful binder, so he decided to do further tests upon this discovery (Jayaraman, 2015). Dr. Vasudevan eventually found that when plastic is combined with stone and bitumen it binds both materials together quickly since both plastic and tar are petroleum products.

This process proved to be very effective, as the plastic infused asphalt is actually much stronger than traditional pavement. Normally when plastic is exposed to heat and light it will break down and give off toxins, but because it is mixed with the bitumen its properties change and it no longer breaks down. This combination of materials improved the strength of the roads by making it more sustainable and flexible (Hassani et al., 2005).

Dr. Vasudevan's discovery was finally put into action when he proceeded to build the Jambulingam Street in Chennai in 2002 as one of the world's first plastic roads (Figure 2.18). The benefits became more apparent once he built this plastic road. After 16 years of use, despite having to withstand a major flood, several monsoons, many heat waves, and non-stop traffic, the street was found to have sustained very little damage, a very impressive feat. The average lifespan of plastic roads is found to be about 10 years, more than double that of a traditional road. This durability helps decrease both time and money spent on road maintenance throughout the years. Some of the additional benefits of building plastic roads are that every kilometer of plastic road uses about one ton of plastic waste, saving that much asphalt, and costs around 8% less than a traditional road.

At least 11 states of India including many cities like Madurai, Chennai, Jamshedpur, Kovilpatti, Kothamangalam, Salem, Wellington, Puducherry, Hindpur (Andhra Pradesh), Kolkata, Goa, Shimla, Thiruvananthapuram, Vadakara, Calicut, and Kochi has been adopted this idea to build more than 33,796 km of roads (World Economic Forum report). Other countries like Indonesia, Australia and UK are trying to adopt this.

Figure 2.18. Jambulingam Street, one of India's first plastic roads (Mizikar et al, 2019)

2.15 Theoretical Explanation of Plastic Road

The shredded plastics on spraying over the hot aggregate get melted and spread over the aggregate giving a thin coating at the surface. When the aggregate temperature is around 140^0–160^0 C the coated plastics remains in the softened state. Over this, hot bitumen (160^0 C) is added. The added bitumen spreads over the aggregate. At this temperature both the coated plastics and bitumen are in the liquid state, capable of easy diffusion at the inter phase. This process is further helped by the increase in the contact area (increased surface area). These observations may be explained as follows. Waste polymers namely PE, PP and PS are hydrocarbons with long chains. The bitumen is a complex mixture of asphaltenes and maltenes which are also long chain hydro carbon. When bitumen was mixed with PCA a portion of bitumen diffuse through the plastic layer and binds with aggregate. The plastic layer has already bonded strongly with aggregate. During this process three dimensional internal cross linked network structure results between polymer molecules and bitumen constitutes. Therefore, the bonding becomes stronger and the removal of bonded bitumen becomes difficult.

Hence, the results of the study done by Vasudevan et al (2011) showed that the bonding between stone aggregate and bitumen is improved due to the presence of polymers. This may be explained by the following structural models (Figure 2.19).

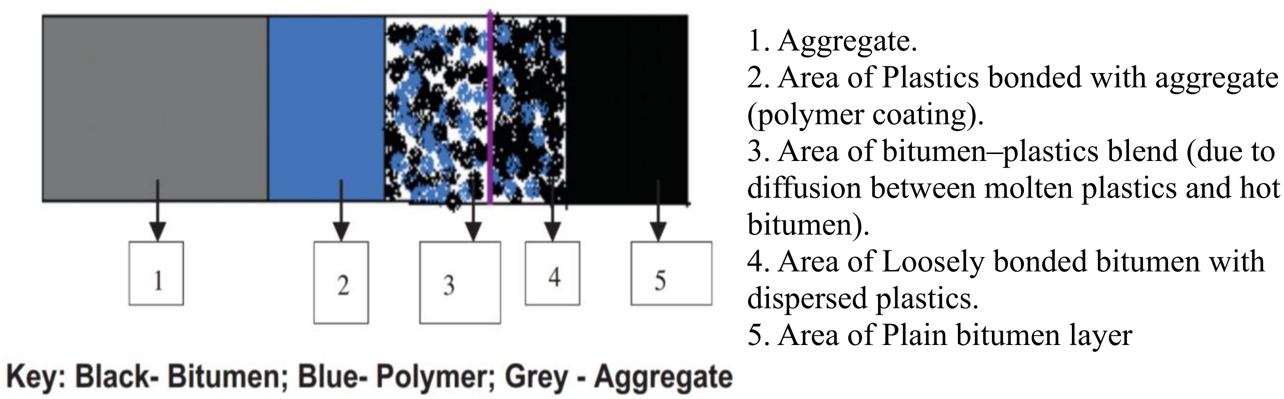


Figure 2.19. A plastic aggregate bitumen interaction model for the Plastics waste coated aggregate bitumen mix (Vasudevan et al, 2011))

2.16 RECYCLED PLASTIC MIXING PROCESS

There are two significant processes by which asphalt mixtures can be changed: the wet process and the dry process (Gawande et al., 2012). If the polymer is first blended with bitumen and aggregate is added later, it is known as a wet process. However, if the polymer is covered with aggregate before the bitumen is added, it is known as a dry process (Shiva et al., 2012). The utilization of recycled plastics in asphalt blends is the most recent widely recognized approach to modifying bituminous mixtures to enhance the thermal behavior. The wet process can be implemented for reusing any kind, size, and state of waste material, such as plastics, rubber, etc. (Huang et al., 2007). The polymers have good outcomes in all tests since the blends delivered are less vulnerable to temperature variations, fatigue cracking, and permanent deformation, which lead to an extended service life (Atta Elmanan et al., 2011; González et al., 2012; Kök and Çolak, 2011; Oliviero Rossi et al., 2015). This strategy has impediments, however, since the polymer must satisfy certain conditions to guarantee the production of a suitably modified binder while maintaining its engineering properties (Presti et al., 2014). Suitable polymers require specific treatment and blending forms (for example, high temperature and rapid shear blending) to improve the properties of the black-top folio and create stable blends with better mechanical attributes and toughness (Al-Adham and Al-Abdul Wahhab, 2018; Montanelli and srl, 2013).

In the wet process, a maximum of 8% of waste plastic by weight of bitumen can be incorporated (Gawande et al., 2012, Duggal et al., 2019). What's more, the use of such plastic blends is more complicated and difficult, and not all reused plastics have acceptable conduct during the incorporation process (Fernandes et al., 2017; Lastra-González et al., 2016). Hence a new innovative dry process that coated the aggregate with plastic was developed by Vasudevan et al. (2011), and a mixture of the plastic-coated aggregate and bitumen showed better binding properties and fewer voids.

In the dry process, plastics are mixed with the aggregate before they are mixed with the bitumen (Huang et al., 2007; Angelone et al., 2016), which makes a thin layer of plastic coating over the aggregate. As soon as the aggregate is blended with the bitumen, the performance of the asphalt blend is greatly enhanced (Lastra-González et al., 2016; Liang et al., 2017). The level of bonding between the plastics and the remainder of the materials relies upon the softening and melting temperature of the plastic. The time required for mixing and blending is also significant and can fluctuate from 2 to 15 minutes (Ahmadinia et al. 2012; Huang et al. 2007); the size of the polymer should also be considered. If the plastic particles are small, they can be easily dispersed into the asphalt blend, which permits the plastics to bond strongly to the aggregates and bitumen (Fakhri and Azami, 2017; Santagata et al., 2012). One of the most critical favorable circumstances of this procedure is that it does not expect any kind of adjustments to the asphalt blending plants. The utilization of more than 15% of waste plastic is conceivable by this process.

2.17 Current Research Findings on Use of Recycled Plastic in Pavement

Previous studies on the use of recycled plastic in asphalt concrete have been very encouraging. Rheological properties of plastic-modified bitumen and various properties of asphalt concrete, such as marshal stability and flow, indirect tensile strength, compressive strength, resilient modulus, rutting, etc. were derived by reviewing available literature.

2.17.1 Plastic Types Suitable for Plastic Road

Literature shows that polymer materials that can be used are low-density polyethylene, such as plastic bags, films, foams, high-density polyethylene, polypropylene, and polystyrene. It is impossible to mix PET particles with bitumen because of the high melting point of the PET particles (Modarres and Hamedi, 2014). Vasudevan et al. (2011) stated that polyvinyl chloride should not be used in order to prevent the possibility of chlorine in the system. Examples of different types of plastics usually used in asphalt mixtures and their benefits and drawbacks are given below:

Polyethylene: Previously conducted research on asphalt mixtures with high-density polyethylene (HDPE) and low-density polyethylene (LDPE) has proven that the stiffness modulus decreased, while the Marshall stability (Akinpelu et al., 2013) and indirect tensile strength (ITS) values increased (Ahmadinia et al., 2011; Yin and Wu, 2018; Zoorob and Suparma, 2000). There was also an improved conserved tensile strength ratio (TSR) and an increase in the fatigue resistance (Fazaeli et al., 2016; Lastra-González et al., 2016; Modarres and Hamedi, 2014), offering outstanding impact resistance; lightweight, low moisture absorption; and high tensile strength (Panda and Mazumder, 2002; Awwad and Shbeeb, 2012).

Polypropylene (PP): PP has been used in asphalt pavement and offers good chemical and fatigue resistance. However, some disadvantages like oxidative degradation, high shrinkage, and thermal expansion have also been observed for pavements made of PP mixed asphalt. (Sultana and Prasad, 2012; Ali et al., 2017)

Polystyrene (PS): Problems associated with water percolation and drainage in asphalt surfaces can be rectified by using PS, and the resulting asphalt mixture will have more strength and resistance than the control sample (Motlagh et al., 2012). However, the unstable behavior of the polymer during the production of the mixtures proves that the results with PS are not as favorable, and its fatigue resistance and lifespan decrease in the process (Lastra- González et al., 2016).

2.17.2 Rheological Properties of Plastic-Modified Bitumen

As the content continues to increase (6-10%), the softening point massively increases, and the high-temperature performance improves greatly (Punith and Veeraragavan, 2007; Selvavathi et al., 2007; Fang, 2008; Cheea et al., 2014; Guru et al, 2014). Since harder grades of bitumen are obtained by using plastic modifiers, this indicates improvement in pavement distress. Increasing the percentage of waste PE from 1 to 10% decreases the asphalt penetration that increases the stiffness, which in turn contributes to the positive characteristics of the polymer-modified bitumen composite, i.e., greater resistance to rutting and less vulnerability to temperature (Punith and Veeraragavan, 2007; Fang, 2008; Dixit and Rastogu, 2013; Cheea et al., 2014). Hadidy and Yi-qui, 2009 also found that the sheer temperature of the composite improves in medium-to-high temperatures because the penetration at 25 °C generally decreases as the PP content increases. Viscosity increases with the increase of recycled polyethylene and results in easier mixing, laying, and compaction of the mixture (Habib et al, 2011; Cheea et al., 2014). Binders modified with polyethylene tend to show a higher elastic modulus and reduce the likelihood of thermal cracking during application (Cheea et al., 2014).

2.17.3 Marshall Mix Design

Bindu and Beena (2010) used shredded waste plastic to stabilize a stone mastic asphalt (SMA) mixture in flexible pavement. They used waste plastic bottles, bags, wrappers, etc., with bitumen of 60/70 penetration grade. They conducted Marshall stability tests for the plastic-mixed asphalt concrete with 10% plastic content and found that it yielded an increased stability of about 64%. Sangita et al. (2011) found that a mixture with 8% plastic content performed well. Figure 2.20 depicts that the Marshall quotient or stability of modified blends is higher than other conventional mixes. A 6% to 8% increase in the modifier content increases the stability of modified blends by about 50%, but expanding the modifier content from 6-8% to 12-15% reduces the stability of the modified mixes. That might occur because of the reduced adhesiveness of the mixture.

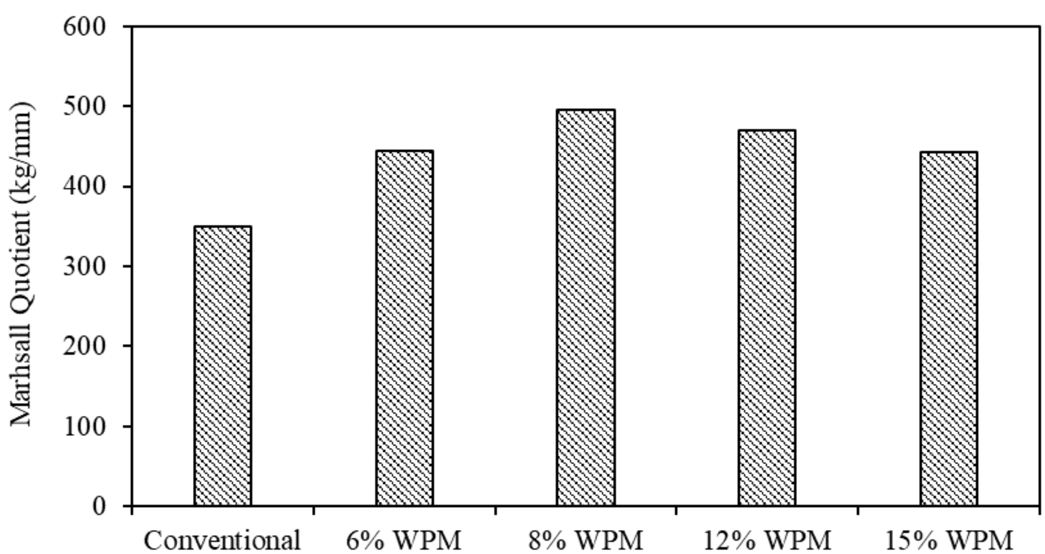


Figure 2.20. Marshall quotient results (Redrawn from Sangita et al. 2011)

Akinpelu et al. (2013) conducted an experiment in Nigeria, where they investigated the effect of polythene on asphalt concrete properties. State polythene was added as a binder modifier in the grinder, and it was introduced to the mixture by melting it in the bitumen mix. The optimal asphalt content was 7%, and six different measures of polyethylene by weight of the optimum binder content were chosen for testing (2.5, 5.0, 7.5, 10, 12.5, and 15%). The results conveyed that 12.5% was the optimum percentage of the modifier by the weight of the optimum bitumen content. It was found to improve the stability of the asphalt concrete specimen from 4440 N to 9180 N, reduce the density, and slightly reduce the flow, from 3.5 to 2.9 mm. This study concluded that the polythene modifier offered excellent engineering properties, as well as better management of the waste hazard created by its utilization as a bitumen modifier.

Rajput and Yadav (2016) conducted a study using shredded plastic waste (e.g., plastic bags, polyethene, etc.). Plastic-modified mix specimens with different percentages of plastic contents (6%, 8%, 10%, 12%, and 14%) by weight of bitumen content were prepared through a dry process in which plastic was added over the heated aggregates. The Marshall stability value increments drastically as the percentage of waste plastic in the mix was increased (Figure 2.21). The maximum stability was achieved in the mix that contained 12% plastic by weight of the bitumen (the optimum plastic content). The accumulation of plastic waste in the mix decreased the percentage of air voids continuously, and the VFB increased continuously as more plastic filled more voids. Hence, it can be concluded that the percentage of air voids decreases with the increase of plastic.

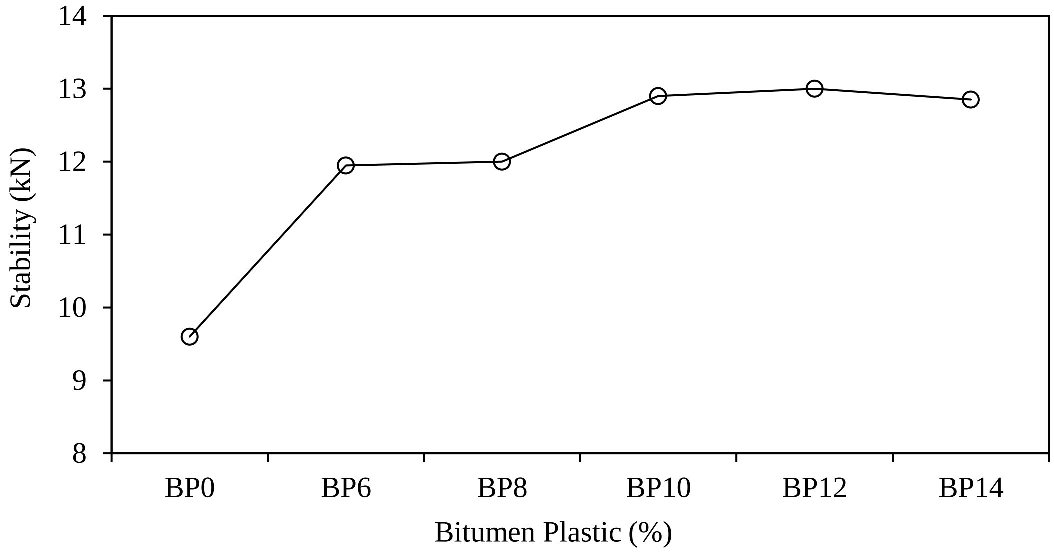


Figure 2.21. Marshall stability results (Redrawn from Rajput and Yadav, 2016)

Plastic waste (LDPE) was used in Rokande's research in 2012. The dry process was used for mixing the LDPE, and the Marshall method of bituminous mix design was executed to determine the different mix design characteristics and different percentages of LDPE. Five percent (5%) of bitumen content was optimum. The stability values increased with the percentage increase in the modifier (LDPE). For 3%, 6% and 9% stability, the values were 1050 kg, 1120 kg and 1185 kg, respectively.

Hınıslıoğlu and Ağar (2003) concluded that the highest stability, the smallest flow, and consequently the highest Marshall quotient, were obtained when the specimens were prepared at a 165 °C mixing temperature and 30 minutes mixing time for 4% HDPE with AC-20 bitumen (Figure 2.22).

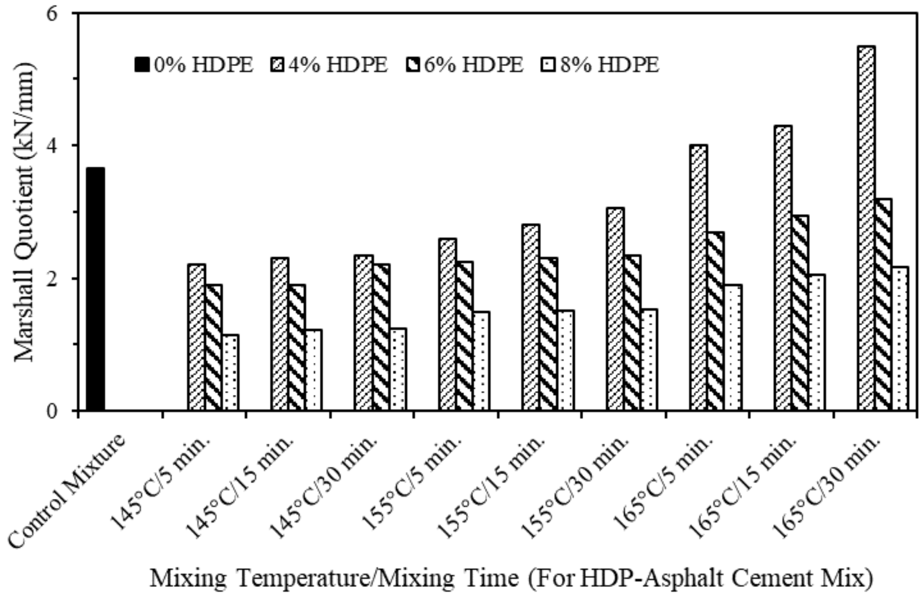


Figure 2.22. Mixing temperature/mixing time vs. Marshall quotient (Redrawn from Hınıslıoğlu and Ağar, 2003)

Kofteci (2016) used HDPE-based waste materials as a modifier in the amount of 1%, 2%, 3%, and 4% to investigate the performance of an asphalt mixture. The performance of the specimen was first measured by the stability and flow value. The HDPE modifier did not affect the sample at low rates since stability values of 1% and 2% HDPE were very close to the control mix. The best performance was obtained with 4% HDPE content, with an increase of stability value from 960 kg to 1080 kg (Figure 2.23).

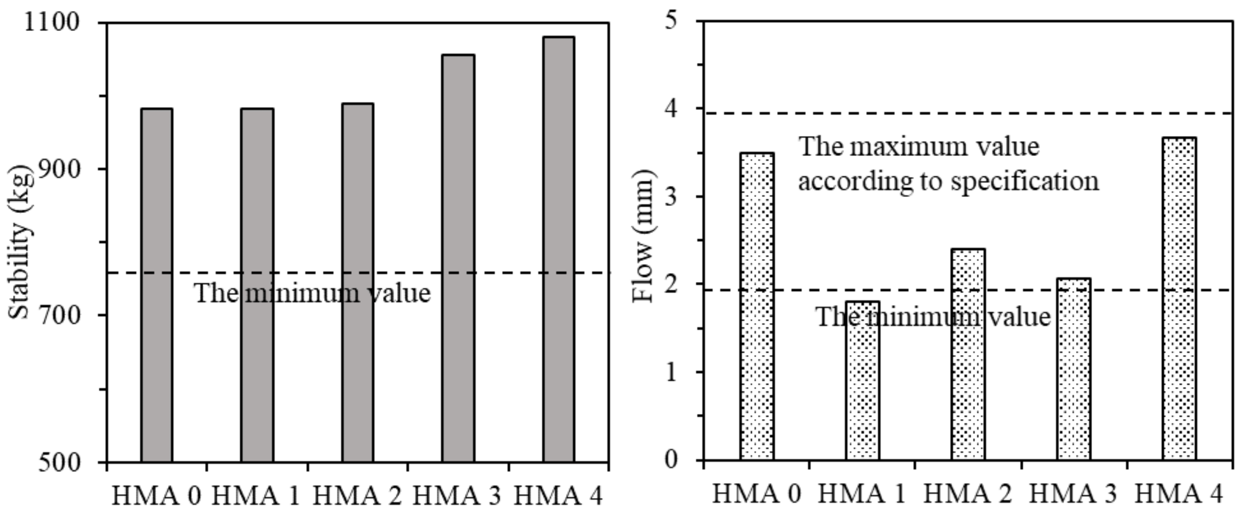


Figure 2.23. Marshall stability and flow results (Kofteci, 2016)

Awwad and Shbeeb (2007) conducted a study to determine the best type and proportion of polyethylene to use. HDPE and LDPE were added through the dry process to coat the agglomeration. The optimum asphalt content was 5.4%, and the polymers were introduced to the mixture in two states (ground and not ground). For the testing process, seven proportions of polyethylene (6, 8, 10, 12, 14, 16, and 18%) by weight of the optimum binder content were selected. The optimum modifier content was found to be 12%. Not all of the individual aggregates could be coated since some of them were not ground. Ground polyethylene was used to strengthen the asphalt mixture's engineering properties, as it provides a better coating for the aggregate and a rougher surface texture. It was concluded that ground HDPE polyethylene modifiers improve the engineering properties, and it was recommended that the modifier proportion be 12% by the weight of bitumen content (Figure 2.24). It was observed that the inclusion of HDPE could reduce the density and increase the stability of the air voids and the voids of the mineral aggregate by a smidgen.

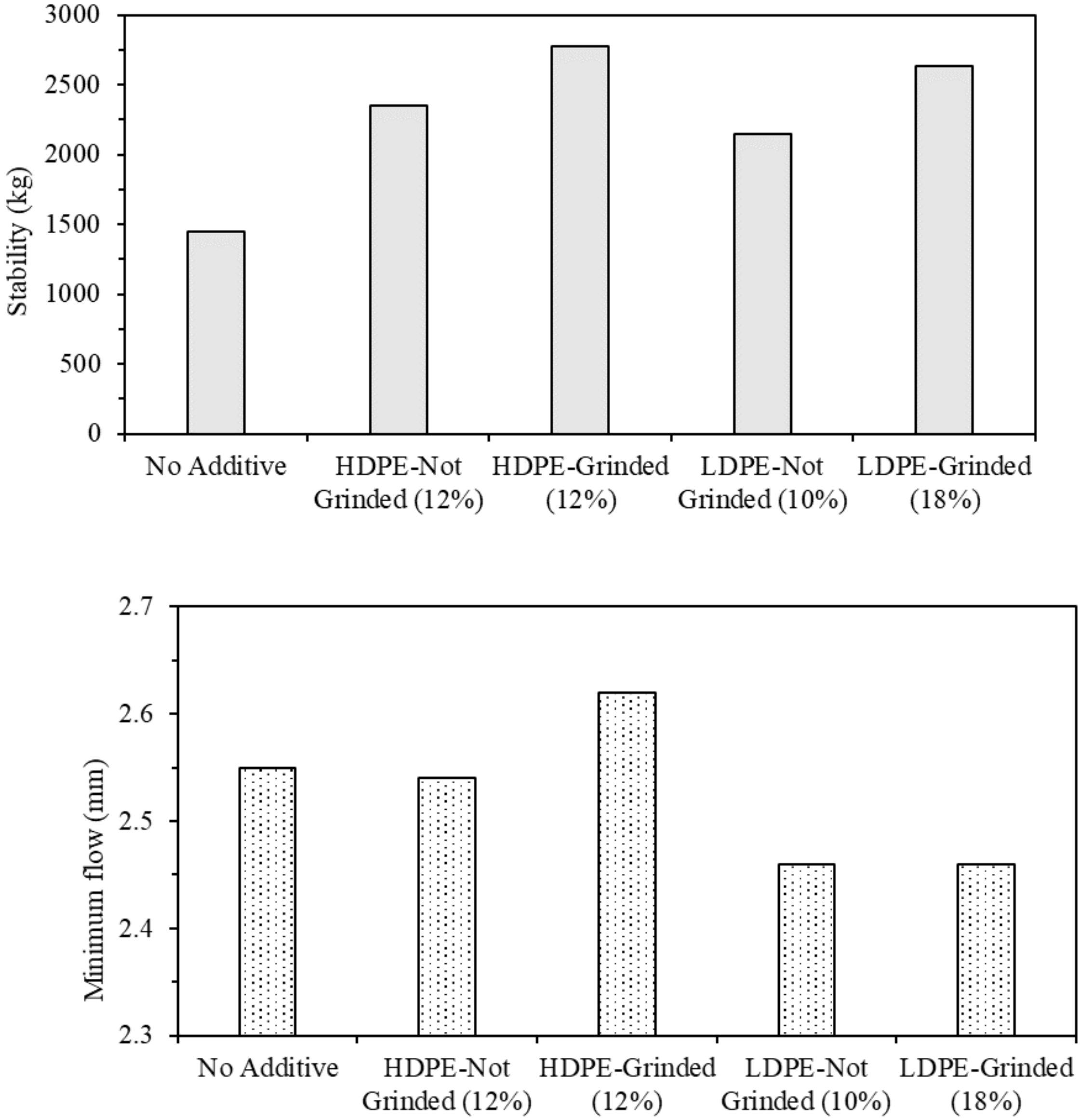


Figure 2.24. Marshall stability and flow results (Redrawn from Awwad and Shbeeb, 2007)

Hadidy and Yi-qui (2009) selected four proportions of pyrolysis polypropylene (PP) to continue their testing. Rheological and homogeneity tests were conducted on unmodified and modified asphalt binders, and the optimum asphalt content obtained from their study was 5.82%. As the stability value was 10.876 KN, 5% PP content by weight of asphalt was recommended to improve the performance of the asphalt concrete mixtures. The addition of PP helped to fill the voids between the particles as well as enhance the interlocking, consequently increasing stability and decreasing flow as the PP content stretched beyond 5% the flow increases and the stability decreases (Figure 2.25).

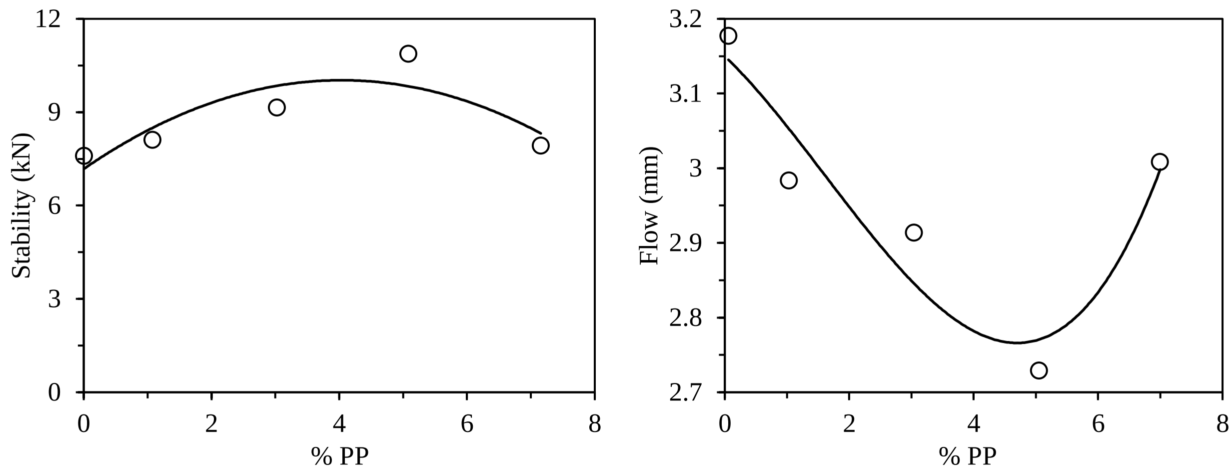


Figure 2.25. Marshall stability and flow Results (Redrawn from Hadidy and Yi-qui, 2009)

2.17.4 Indirect tensile strength

In Punith and Veeraragavan’s (2007) study, various percentages of PE (2.5, 5.0, 7.5, and 10% by weight of bitumen content) were mixed with 80/100 paving grade asphalt. LDPE plastic bags were added to the asphalt by the wet process. PE ratios greater than 10% posed problems, as blending them with the asphalt cement became difficult due to increased viscosity of the binder. They also conducted indirect tensile strength tests and found that the indirect tensile strength was 38 KN, while it was 29 KN with no PE content (Figure 2.26).

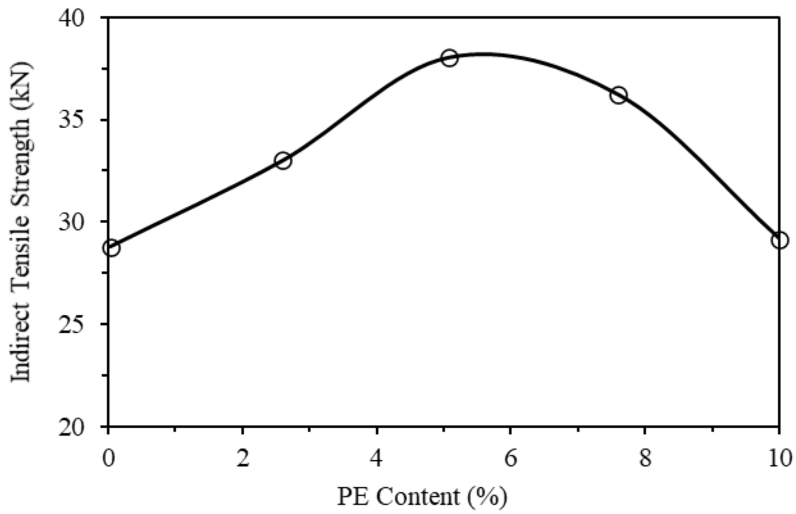


Figure 2.26. IDT results (Redrawn from Punith and Veeraragavan, 2007)

Sangita et al (2011) showed that when the WPMB mix contained 8% WPM, its indirect tensile strength (ITS) results were higher (12 kg/sq.cm) than the conventional mix (6 kg/sq.cm), as shown in Figure 2.27. This shows that the WPMB mix can withstand high tensile strains before it reaches its cracking state.

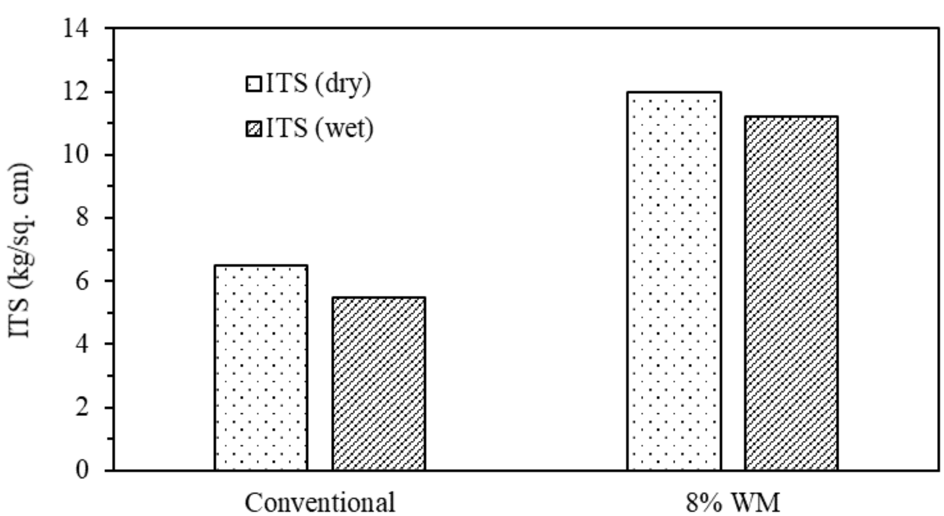


Figure 2.27. Indirect tensile strength result (Redrawn from Sangita et al., 2011)

Attaelman et al. (2011) used 0-7% HDPE with 80/100 penetration-grade bitumen. The mixtures that contained HDPE had a tensile strength ratio greater than 85%. The tensile strength increased up to a 5% addition of HDPE, then it began to decrease (Figure 2.28). The high stability made way for increased resistance against permanent deformation.

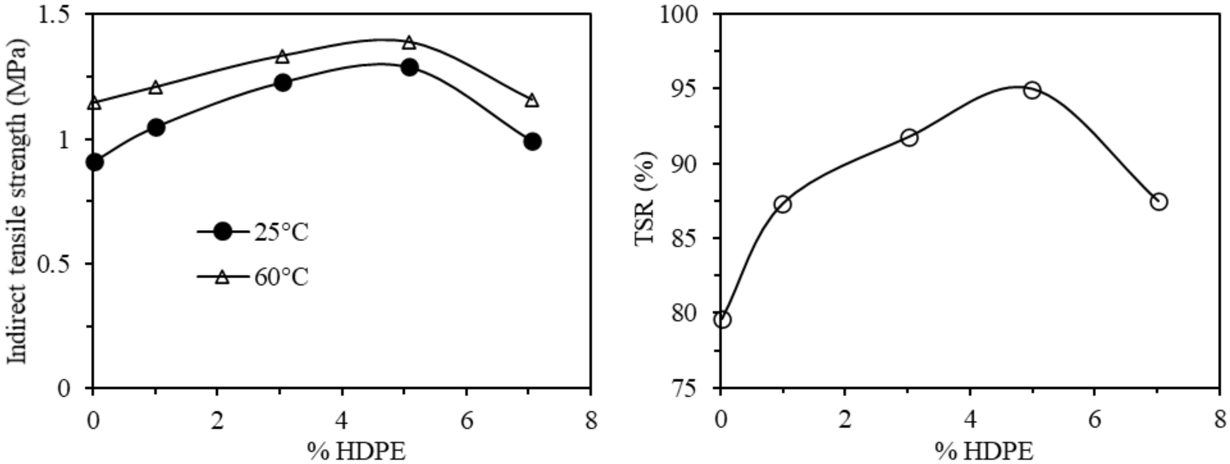


Figure 2.28. IDT and TSR results (Redrawn from Attaelman et al., 2011)

Anurag and Rao (2018) used the dry and wet processes for their mixes with waste plastic for their experimentations. They conducted the indirect tensile strength test since a decent tensile strength indicates a better resistance to cracking, and found that the indirect tensile strength (ITS)) of the sample increased up to 8% for the dry process and 6% for the wet process when using LDPE and HDPE types of waste plastic (Figure 2.29).

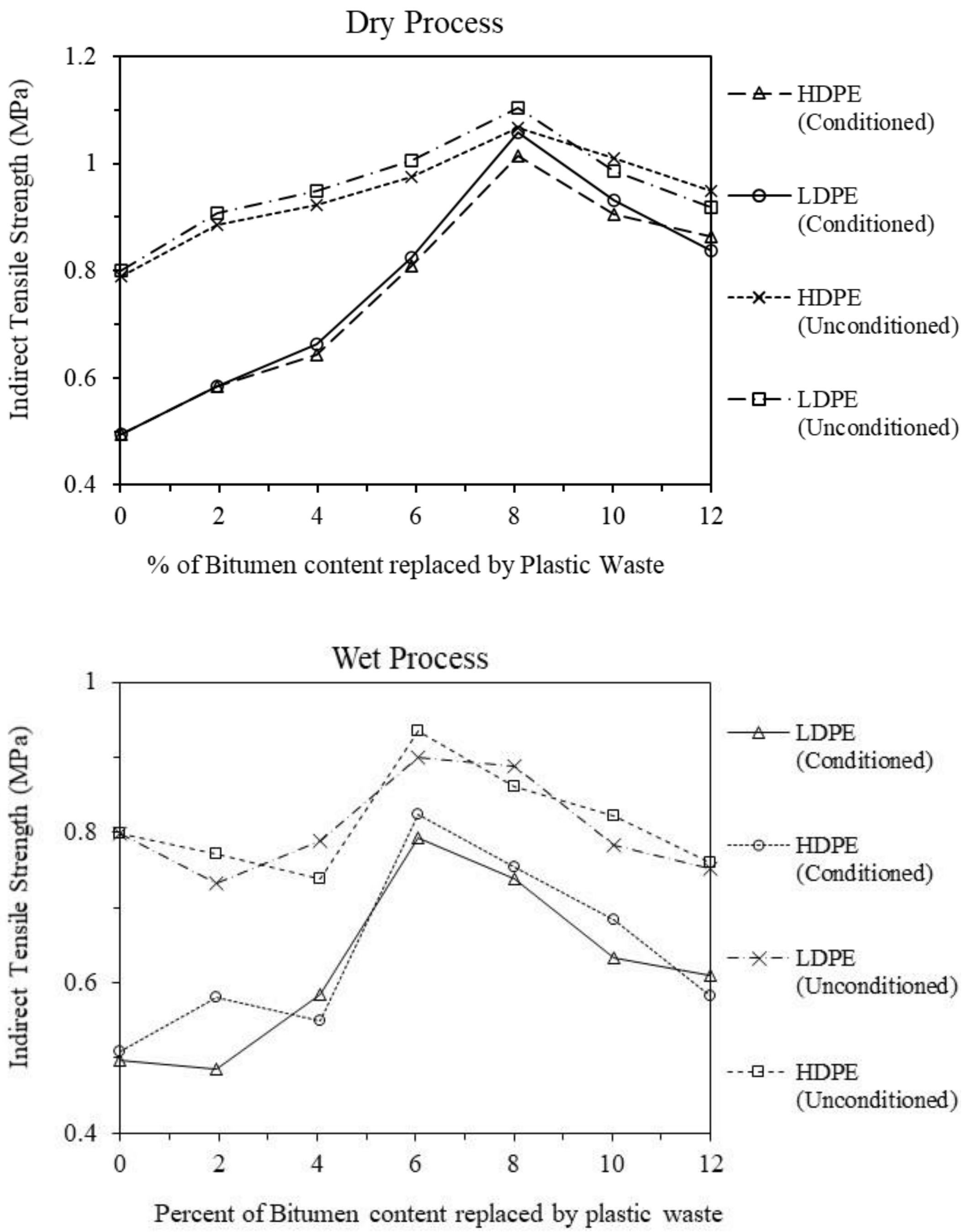


Figure 2.29. IDT test Results (Redrawn from Anurag and Rao, 2018)

Hadidy and Yi-qui (2009) selected both polyethylene (PE) and polypropylene (PP) and found that the indirect tensile strength for PE and PP-modified asphalt mixtures was slightly higher than those of traditional asphalt mixes (Figure 2.30). This was attributed to the modified asphalt having a higher viscosity than that of conventional asphalt.

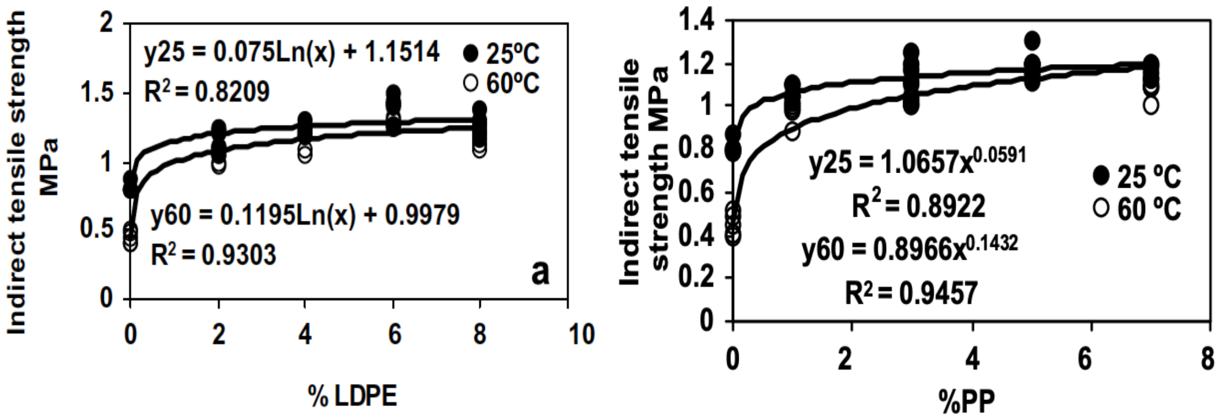


Figure 2.30. Indirect tensile strength test results (Hadidy and Yi-qui, 2009)

According to Pamungkas et al. (2018), PP has the highest tensile strength (6738 KPA) than other types of plastic mixtures like LDPE (1211 kPa) and PET (4703 kPa). In the ITS test, PP mix objects turned out to be 56.3% stronger than PET and 397% stronger than LDPE. PP mixed samples had higher tensile strength due to the nature of the PP plastic. It can change shape and become softer in heat conditions, the aggregate attachment becomes stronger, and the object becomes denser after compaction, resulting in higher tensile strength.

2.17.5 Compressive strength

The relation of the compressive strength with the variations of the ratio of PP content was shown by Hadidy and Yi-qui (2009). In Figure 2.31, it can be observed that at 5% PP content, the compressive strength (6 MPa) was the highest in this mix. The study revealed that the addition of 5% PP in asphalt increased the percentage of the compressive strength value, which was found to be 20.9% and 49.2% at 25 °C and 60 °C, respectively.

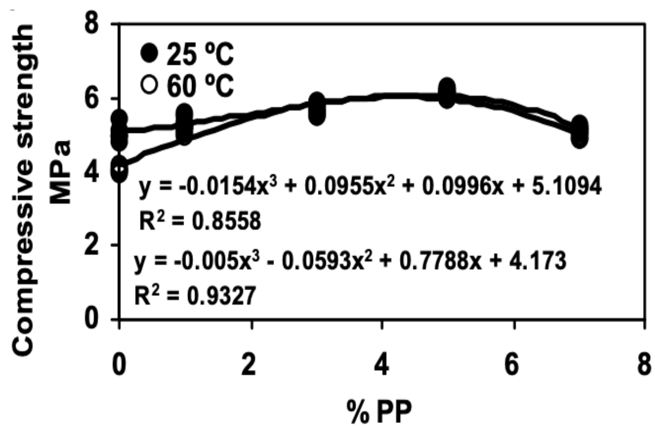


Figure 2.31. Compressive strength result (Hadidy and Yi-qui, 2009)

Pamungkas et al. (2018) investigated the effects of adding different types of plastic on the compressive strength of asphalt concrete. They concluded that the mixtures that contained PP had higher compressive strength (11840 KPa) than the other types of mixtures (Figure 2.32). When the UCS test was conducted, it showed that the PP mix sample was able to hold the vertical pressure of 253% and 399% stronger than PET and LDPE, respectively.

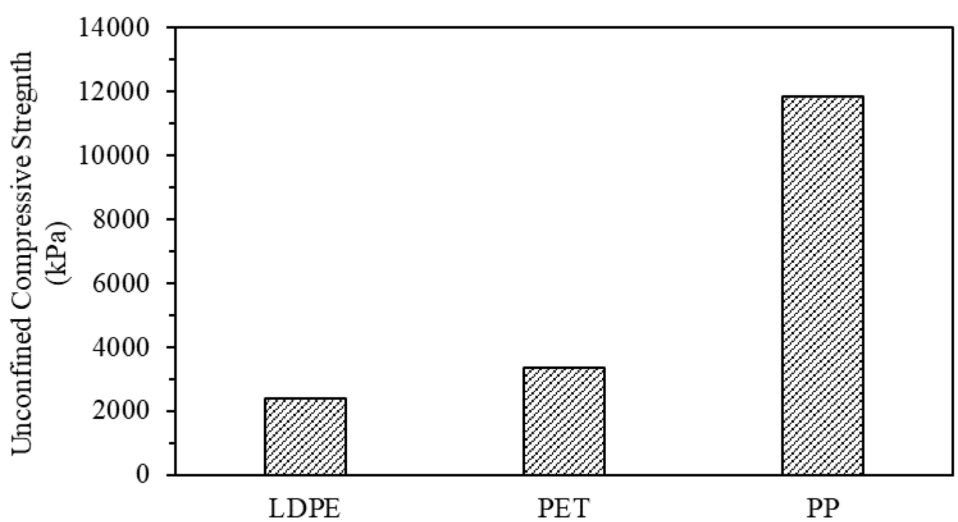


Figure 2.32. Compressive strength result (Pamungkas et al. 2018)

2.17.6 Resilient Modulus

Punith and Veeraragavan's investigation in 2007 showed that 5% of PE content by weight of asphalt improves the performance of asphalt concrete mixtures, as the resilient modulus value increased by 28.9% (from 2040 MPA to 2630 MPa) at 25 ^{0}C (Figure 2.33).

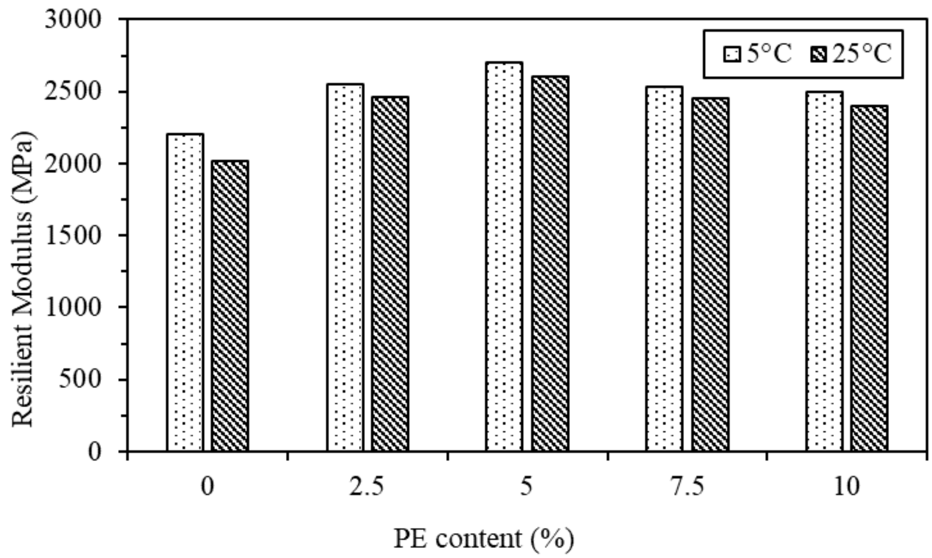


Figure 2.33. Resilient modulus results (Redrawn from Punith and Veeraragavan, 2007)

Attaelman et al. (2011) reported an increase in resilient modulus values at a high (25 °C) temperature when HDPE was used in asphalt concrete mixtures. The modifiers do not weaken the mixture, even when it is exposed to moisture, but just 5% HDPE can result in a flexible, great performing, durable, economical pavement (Figure 2.34).

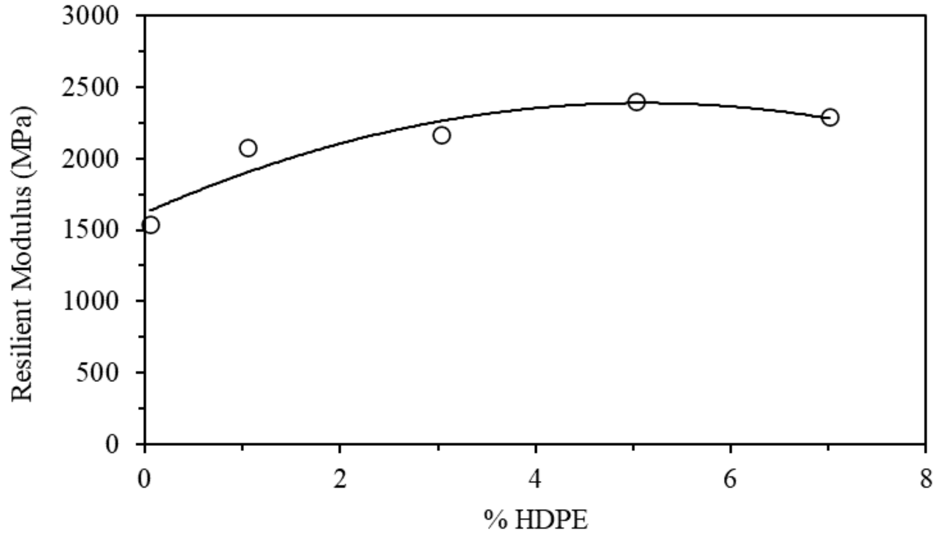


Figure 2.34. Resilient Modulus results (Redrawn from Attaelman et al., 2011)

2.17.7 Rutting

Use of waste plastic in asphalt mixtures increases the resistance against deformation compared to the regular reference mixture. Also, the plastic mixed asphalt can be used in roads associated with high and heavy volume of traffic as well as severe climatic conditions (Lastra- González et al., 2016). The Hamburg wheel tracking tests were conducted by Sangita et al. (2011), and it was found that the conventional bituminous concrete mixes performed poorly and were more susceptible (up to 6.44 mm) to rut deformation than the modified mix consisting of 8% waste polymer modifier, which performed a lot better and was less susceptible (3.68 mm) to rut deformation (Figure 2.35).

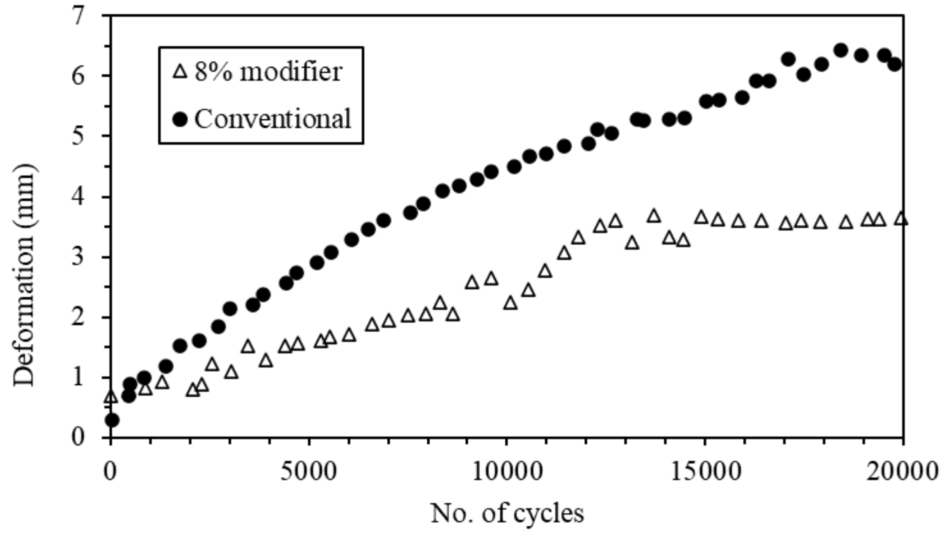


Figure 2.35. Hamburg wheel tracking test results (Sangita et al., 2011)

Napiah et al. (2013) performed research to investigate the deformation behavior of well-graded bitumen and calculate the rut depth. Figure 2.36 shows that the rut depth corresponding to one million axle wheel loads of unmodified, 1%, and 2% LLDPE mix is higher compared to the 3% LLDPE modified bituminous mix which offers the lowest rut depth.

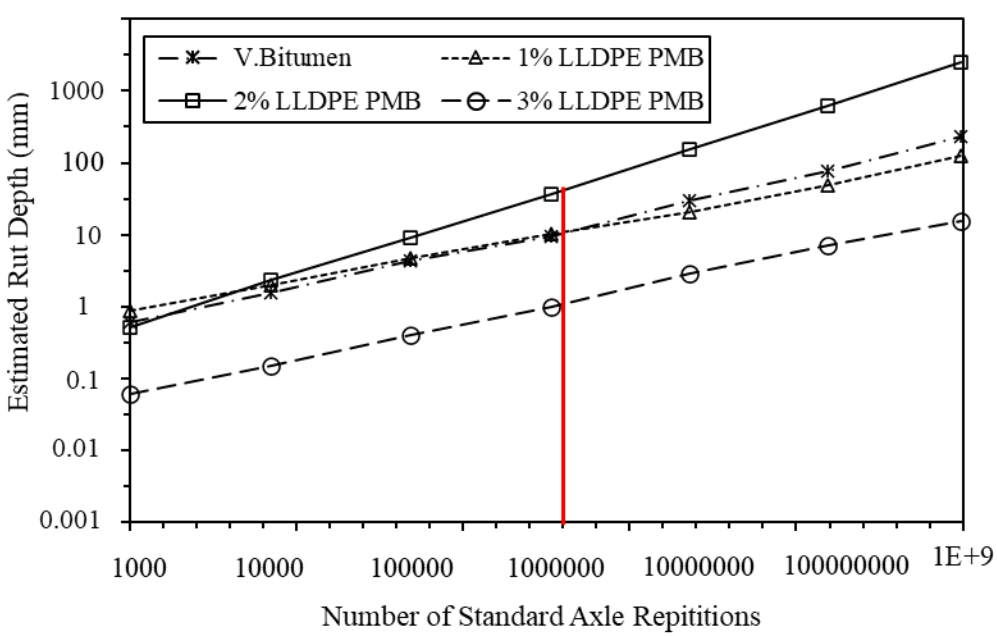


Figure 2.36. Rutting results (Redrawn from Napiah et al., 2013)

2.18 Salient Features of Plastic Road

When bitumen is added to plastic waste coated aggregate, a better adhesion formed between bitumen and plastic waste coated aggregate due to strong inter molecular bonding. These inter-molecular attractions enhanced strength of bitumen concrete mix, which in turn helped in enhancing durability and stability of mixes. Plastic-bitumen composite roads have better wear resistance than standard asphalt concrete roads. They do not absorb water, have better flexibility which results in less rutting and less need for repair. It is unaffected by corrosion and weather. The road structure can handle temperatures of -40^{0}F to temps as high as 176^{0}F with no negative effects. As this road can handle excessive seasonal temperature variation, it causes less pavement distress. Thus plastic waste modified bitumen concrete mixes are expected to be more durable, less susceptible to moisture and temperature in actual field conditions with improved performance (Sabina et al, 2009).

Different advantages of plastic road over conventional one and reasons behind this are briefly discussed below:

Durability and Stability

Visco elastic property of the mix become unhanged over time due to its strong intermolecular bonding between bitumen and plastic coated aggregate which enhanced the durability and stability of pavement. Hence the plastic tar road, on prolonged exposure to the atmosphere and in different environmental conditions does not show any change in the visco elastic nature. This would help in increasing the stability of the mix, reduces the stripping of bitumen which results in raveling and loosening of the surface layers. The aging or the oxidation level of the bitumen present in the plastic coated aggregate bitumen mix is also very low due to the dual binding of bitumen to the aggregate with the plastic and also the slight modification of bitumen properties due to its mixing with polymers (Vasudevan et al, 2011).

Rutting Resistance

Pavement deflection and unevenness can be within tolerance level if using plastic with bitumen. The reason is upon coating the polymers over the aggregate and mixing it with bitumen the polymer will help in maintaining the visco elastic property of the mix due to its strong bonding and a small modification in the structure of bitumen (Vasudevan et al, 2011). Plastic tar road absorbs less moisture, have better flexibility which results in less rutting and less need for repair.

Skid Resistance

In the case of plastic tar road, it absorbs less moisture and reduces slippery. Hence there is no bleeding. Plastics will increase the melting point of the bitumen due to a small modification in the structure of bitumen which is best during high temperatures (i.e. summer season) and it also reduce the bleeding and skidding (Vasudevan et al, 2011).

2.19 Actual Road Performance

Results of field tests that were conducted systematically from May 2007 to May 2008 on six sites by Vasudevan et al. (2011) are depicted in Table 2 . Site 6 indicates the performance of the referenced plain bitumen road, and sites 1 to 5 portray the performance of the plastic road. The roughness (unevenness) of the surface course; skid resistance; texture depth; field density of the road; rebound deflection; and physical examination of road conditions like cracks, raveling, potholes, rutting, and corrugated edge breaks were carried out according to the standard specifications. The results were compared with standard results and are shown in Table 2.2.

From the table, it can be safely concluded that from 2002 to 2006, the plastic roads that were laid showed properties of decent roads. The roads built in 2002 and 2003 reflect better results than the plain bitumen road, which shows a value higher than the tolerance value. There were no pothole formations, cracking, deformation, rutting, raveling, or edge flow observed when the physical surface condition survey was carried out for the plastic road. Hence, Vasudevan et al. (2011) concluded that when compared side-by-side with the plain bitumen road, the plastic roads showed good skid resistance value, good surface evenness, good texture value, and decent strength.

Table 2.2. Summary of the field results of the plastic road (Vasudevan et al., 2011)

Road	**Year laid**	**Unevenness (mm/km)**	**Skid number**	**Texture depth (mm)**	**Field density (kg/m3)**	**Rebound deflection (mm)**
Jambulingam street	2002	2700	41	0.63	2.55	0.85
Veerabhadra street	2003	3785	45	0.70	2.62	0.6
Vandiyur road	2004	3005	41	0.66	2.75	0.84
Vilachery road, Mai	2005	3891	45	0.5	2.89	0.86
Canteen road, TCE	2006	3100	45	0.65	2.86	0.86
Plain bitumen road	2002	5200	76	0.83	2.33	1.55
Tolerance value	-	4000	<65	0.6-0.8	2.86	0.5-1

2.20 Environmental Effect of Using Recycled Plastic

When plastic is exposed to heat and light, it usually breaks down and gives off toxins, but when it is mixed with the bitumen, its properties change, it no longer breaks down, and gas is not generated at temperatures from 130 to 180 °C. Since it has a unique binding property, plastic can be used as a binder and can also be blended with a binder like bitumen to improve the binding property (Gawande, 2012).

Plastic also helps to reduce carbon dioxide emissions during the laying of roads. When littered waste plastics are burned, they produce greenhouse gases. In the dry process, waste plastics acquire a surface coating of aggregates by softening the plastic with moderate temperatures, not by burning. Hence, there is no possibility of a gas like carbon dioxide being generated. Three tons of carbon dioxide can be reduced while using a minimum of one ton of plastic waste to build one-kilometer single lane roads. PP, PE, and PS have softening points at 100-160 C with no gas release, but PVS has a softening point at 200-220 ^{0}C with HCl emission, hence cannot be used in this process.

Application of PS, PE, and PP do not liberate dioxin even on burning at 700 °C

$$PS, PE, PP \longrightarrow C, CO, CO_2 \text{ (but no } Cl_2)$$

Use of only PVC plastic on heating may result in dioxin formation (300-350 °C)

$$PVC \longrightarrow CO, CO_2, Cl_2 \text{ and } HCl$$

Vasudevan et al. (2011) studied the thermal behavior of PE, PP, and PS polymers and used a thermo gravimetry analyzer and differential scanning calorimeter to evaluate the temperature at which the polymers softened and decomposed.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This experimental program was developed and conducted to determine the performance of bitumen mix under modified conditions using recycled plastics. Bulk density test, Rice gravity test, Hamburg wheel tracker test, Indiresct tensile strength test, overlay test and moisture susceptibility test were determined for different combinations of RAP and recycled plastic materials, and were compared to achieve the strength and stiffness required by various guidelines. The test methods, specifications, and testing equipment are described in the following sections.

3.2 Sample Collection

For this experimental program aggregate, bitumen and three types of recycled plastics have been collected.

3.2.1 Aggregate and Bitumen Collection

For this project different types of aggregates are collected from TxDOT approved locations. For Surface course, a Superpave SP-C aggregate gradation and performance-graded (PG) binders are chosen to conduct this research. For base and subbase course recycled crushed concrete aggregates are collected as well.

Hot mix asphalt (HMA) mix design is the process of determining what aggregate to use, what asphalt binder to use and what the optimum combination of these two ingredients ought to be. Mix design is a laboratory simulation. It is meant to simulate actual HMA manufacturing, construction and performance to the extent possible. Then, from this simulation it can be predicted (with reasonable certainty) what type of mix design is best for the particular application in question and how it will perform. Under the Strategic Highway Research Program (SHRP), an initiative was undertaken to improve materials selection and mixture design by developing a new mix design method that accounts for traffic loading and environmental conditions and a new method of asphalt binder evaluation which is called the Superior Performing Asphalt Pavement System (Superpave). The Superpave mix design method was designed to replace the Hveem and Marshall methods. The Superpave system ties asphalt binder and aggregate selection into the mix design process and considers traffic and climate as well. So, in this research all the mix design will be conducted according to Superpave mix design. Samples will be compacted using Superpave Gyratory Compactor.

To meet Superpave SP-C aggregate gradation Type C rock, Type D rock, Man sand and recycled asphalt pavement (RAP) are collected (Figure 3.1). The Superpave binder specifications are performance-based; therefore, these binders are known as performance-graded (PG) binders in contrast to the older system of viscosity graded (AC) binders, which are typically used for surface treatments and aggregate precoating. In this research, PG 64-22 binder having a specific gravity of 1.032 and flash point of 313^0C is being used. The aggregates and bitumen are collected from Austin Paving Co, Dallas.

Figure 3.1. Aggregate & Bitumen collection from Austin Paving Co., Goodnight Lane, Dallas

3.2.2 Collection of Plastic

High density polyethylene (HDPE), polypropylene (PP) and Polyethylene terephthalate (PET) plastic were collected from the Republic Services Material Recovery Facility (MRF), Fort Worth, Texas. The MRF usually collects the waste from curbside trash of nearby cities and conduct a thorough sorting of the collected waste plastics according to the seven categories mentioned earlier. One bale of three kinds of plastics (HDPE, PP, PET) each having around 1000 t0 1500-lb plastic was collected from there for the further test purpose.

Again, Low density polyethylene (LDPE) such as grocery bags, plastic wraps, thin film etc. are also needed to proceed the tests. As these kinds of plastics are not recyclable, from the Republic Services it could not be managed. So, these plastics were collected from the surrounding households and campus dustbins 3.2 shows the collection of different types of plastic.

Figure 3.2. Collection of different types of plastic

3.3 Plastic Processing

In order to use plastic in pavement construction, some preparations should be made before mixing it in surface and base course. The steps are mainly to choose the plastic that can be used in this particular project based on their availability, cost and melting point, sorting, cleaning, drying and shredding. Among the seven grades of plastics mentioned earlier, three types of plastics: Low density polyethylene (LDPE), High density polyethylene (HDPE), polypropylene (PP) are chosen, collected and sorted accordingly. PET, PS and PVC plastic are not considered to use since they have high melting point. Moreover, PVC releases toxic elements while melting.

3.3.1 Sorting and Cleaning of Plastic

Each plastic container and bottle are manually selected and sorted. The process of sorting large bales is crucial to avoid contamination with other materials like paper, dirt, or other types of plastic. As soon as plastics were collected from the bale, they were taken for cleaning (Figure 3.3). All the plastics were submerged in the water for two hours for deep cleansing (Figure 3.4). Clean water was then used to rinse the plastics (Figure 3.5).

Figure 3.3. Collection of plastic from bales

Figure 3.4. Submerging plastic into water

Figure 3.5. Washing of plastic

3.3.2 Drying of Plastic

All the plastics were then taken outside to dry under the sun after being cleaned. They were left to dry under the sun for 24 hours (Figure 3.6).

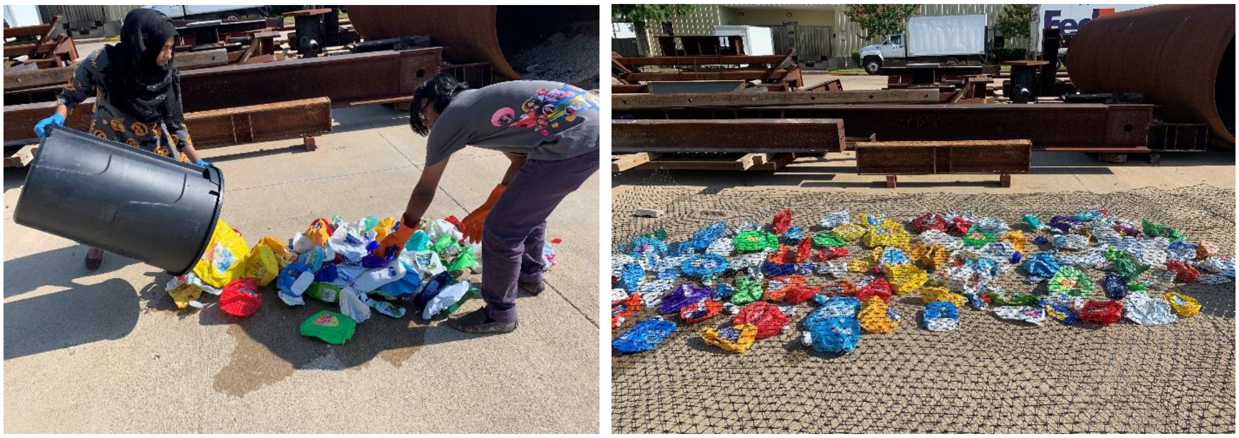


Figure 3.6. Drying of plastic

3.3.3 Shredding of Plastic

Cleaned and dried plastics were transported to a shredding facility for first stage shredding (Figure 3.7). Balcones Shred, Dallas shredded the plastics into heterogeneous mesh size ranges from 1 inch to 3 inch (Figure 3.8). The HDPE and PP are required to shred into small pieces of 3mm-6mm to use the plastic for further use. Using a small-scale shredder, the second stage was done in the Civil Engineering Laboratory Building. For this research study INTBUYING 220V Heavy Duty Plastic Grinder/Granulator is used to shred the plastics into smaller size.

Figure 3.7. Taking plastic for shredding

Figure 3.8. Shredding and collection of plastic

Figure 3.9. Storing of shredded plastic

The LDPE plastic bags are also required to cut into small pieces. As it can’t be possible to shred these in that shredding machine, the plastics are cut manually. Figure 3.10 shows the shredded plastic.

Figure 3.10. Shredded plastic

3.4 Experimental Program

As a part of this ongoing research, different plastic combinations have been used, to replace up to 16% by weight of asphalt binder from the mixture. Important to note, all of the asphalt mixture contained two different percentage (15% and 25%) of reclaimed asphalt pavement (RAP), which is the conventional mixture used by the TxDOT. This research aims at evaluating the potential reuse of recycled plastics for plastic road design and their performance with the presence of reclaimed asphalt pavement (RAP). The experimental program undertaken in this research is the key to the entire assessment. At the beginning of the research, it is necessary to determine the optimum bitumen content of the control mix. Once this optimum bitumen content is fixed, further testing will be run by taking out some percentage of the optimum bitumen content and add the same amount of plastic into it to replace the bitumen. The optimum plastic content can be determined after conducting the test program shown in Figure 3.11. These mixes will be tested to determine volumetric properties by conducting Bulk density test and Rice gravity test. After finding these volumetric properties, the Indirect tensile strength test (IDT), Moisture susceptibility test, Hamburg rutting test and Overlay test will be performed to evaluate strength and deformation properties of the pavement surface mix.

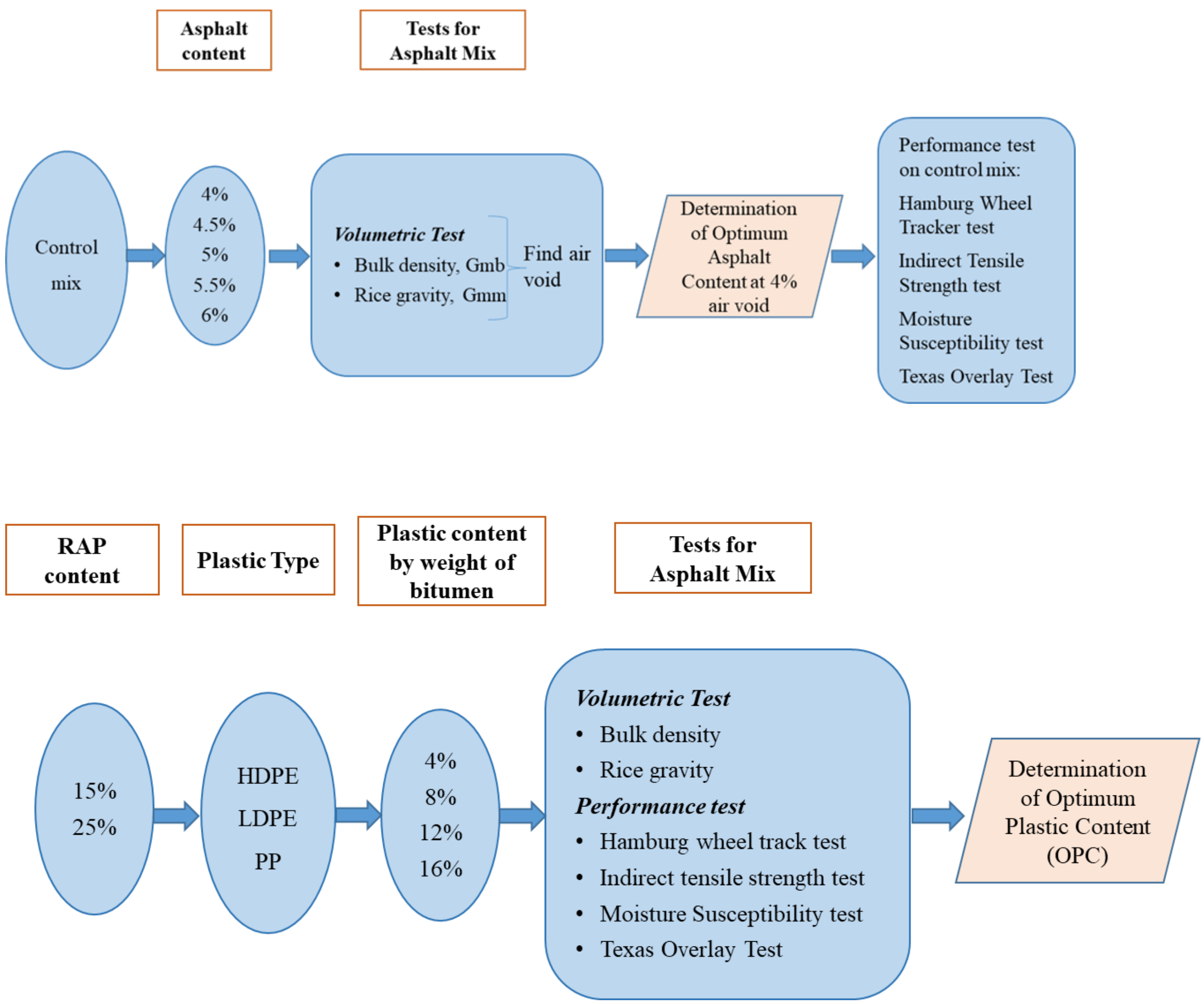


Figure 3.11. Experimental program

Total 360 tests are conducted in this study where more than one samples are taken for each test to ensure the repeatablilty of the tests. For this reason, two samples were tested for bulk density, rice gravity and rutting tests while three samples were tested for IDT, overlay and moisture susceptibility tests for each set of combination (Table 3.1).

Table 3.1. Total number of test done in this study

RAP content	Plastic type	Plastic Content (%)	Bulk density	Rice density	Rutting	IDT	Moisture susceptibility	Overlay	Total Tests
15% RAP	Control	0	2	2	2	3	3	3	15
	LDPE	4,8,12,16	8	8	8	12	12	12	60
	LDPE +0.5%PP (As aggregate replacement)	4,8,12,16	8	8	8	12	12	12	60
	HDPE	4,8,12,16	8	8	8	12	12	12	60
	PP	4,8,12,16	8	8	8	12	12	12	60
25% RAP	Control	0	2	2	2	3	3	3	15
	LDPE +0.5%PP (As aggregate replacement)	4,8	4	4	4	6	6	6	30
	HDPE	4,8	4	4	4	6	6	6	30
	PP	4,8	4	4	4	6	6	6	30
Total number of tests=									**360**

3.5 Aggregate Gradation

Particle size distribution for all types of materials was done by sieve analysis as per standard test method specified in TxDOT guidelines (Tex- 110E) for particle size analysis of aggregates. Recycled base materials were subjected to gradation to check its conformity with TxDOT standard. According to the Tex- 110E guidelines, gradation of the base materials was determined. According to Texas Department of Transportation (TxDOT) specification Item 276, no Hydrometer analysis is required if percent passing on No. 200 sieve is less than 1%. In this case, the amount of percent passing through the No. 200 sieve was less than 1%, so a hydrometer analysis was not necessary. The amount of material retained in each sieve was weighed, and the percent passing through the sieve was calculated. The material retained in each sieve was divided by the weight of the total sample and then subtracted from the total percentage of material. The percent of material passing through each sieve was plotted against the sieve size on semi-log graph paper. Atterberg limits were not also done because of the same reason.

3.6 Mixing Procedure of LDPE

Dry mixing was adapted to continue the test procedure. The following procedure was followed for mixing the ingredients (Figure 3.12).

- The specified amount of coarse aggregate and fine aggregate were placed in a pan and retained in an oven at a temperature of 170°C for 2 hours. Bitumen was kept in the oven at the same time. As aggregates, plastic and bitumen are to be mixed at a heated temperature, preheating is required.
- The required amount of shredded LDPE was weighed and kept in a separate container.
- LDPE is sprinkled over hot aggregate and continue to mix until all the plastic gets melted coated the aggregate.
- Now required amount of bitumen was added to this mix and the whole mix was stirred uniformly and homogenously. This was continued for 15-20 minutes till they were properly mixed which was evident from the uniform color throughout the mix.

Measuring aggregate

Placing in oven

Mixing plastic

Adding bitumen

Superpave Gyratory compactor

Compacted sample

Figure 3.12. Procedures of sample preparation

- Due to the fact that RAP material also contained some binder, the amount of binder to add was adjusted. The weight of new binder added to the mixture was calculated as follows:

$$\textbf{Percent binder}(\textbf{total}) \times \textbf{Total weight}/100 - (\textbf{weight of binder in RAP}) \qquad 3.1$$

where, weight of binder in RAP= (percent binder in RAP) × (weight of RAP)

- Then the mix was kept in the oven again for 2 hours at 150℃ compaction temperature for short-term aging that simulates asphalt mixture production in the plant. After two hours the molds are made using Superpave gyratory compactor (SGC). Table 3.2 is showing the compacting parameters for Superpave Gyratory Compactor.

Table 3.2. Compacting Parameters for Superpave Gyratory Compactor

Parameter	**Value**
Diameter	150 mm
Pressure	600±18 kPa
Angle of gyration	1.16° ± 0.02°
Number of gyration	N_{design}= 50
Speed of rotation	30±0.5 gyrations per minute

- Then each sample was marked and kept separately.

3.7 Mixing HDPE and PP

According to the Thermogravimetric Analysis (TGA) test, HDPE and PP has melting point ranged 170^0 to 190^0 C. As we are using manual mixing, it is difficult to keep constant temperature while mixing. As a result, to mix HDPE and PP, first we mix the HDPE/PP with the aggregate and keep them in the oven at 185°C overnight. The later procedures are same as LDPE mixing. Since it seemed difficult to mix hard plastic like HDPE and PP manually, a temperature controlled automated mixture was bought and used later in this study. This mixture can mix the samples in a controlled temperature, time and rpm setting which makes the mixing quite convenient and uniform (Figure 3.13).

(a)

(b) (c)

Figure 3.13. (a) Temperature controlled automated mixer (b) HDPE and (c)PP mix with aggregate using automated mixer

3.8 Volumetric Test

In this section two tests will be done which are bulk density test and rice test/ theoretical maximum specific gravity test. These tests are needed in order to calculate air void. Percent air voids is calculated by comparing a test specimen's bulk specific gravity (G_{mb}) with its theoretical maximum specific gravity (G_{mm}) and assuming the difference is due to air.

3.8.1 Bulk Density Test

The design of superpave mixes is a volumetric process; key properties are expressed as volumetric values. Due to the difficulty of direct volume measurements, weight measurements are generally taken and then converted to volume using material-specific gravities. Therefore, Specific gravity is a measure of a material's density (mass per unit volume) as compared to the density of water at 73.4°F (23°C). By definition, water at 73.4°F (23°C) has a specific gravity of 1. In addition to air voids, VMA and indirectly VFA, bulk specific gravity is used in most key mix design calculations. Mix design must be based on the correct and accurate determination of bulk specific gravity. The most common method (AASHTO T 166 or Tex-207 Part 1: Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens), calculates the specimen volume by subtracting the mass of the specimen in water (Figure 3.14) from the mass of a SSD specimen. SSD refers to a specimen condition in which all the internal air voids are filled with water, while both the surface and the air voids connected to the surface are dry. The samples for this bulk density are prepared with 50 gyrations having 150 mm diameter and 115+/-10 mm height. To get the most accurate result Tex-207 Part 6 should be adopted if the apparatus will be available. The following calculations can be used to determine bulk specific gravity and percent of water absorbed by the specimen:

$$G_{mb} = \frac{A}{B-C}$$

Where, G_{mb} = bulk specific gravity

A = weight of dry specimen in air, g

B = weight of the SSD specimen in air, g

C = weight of the specimen in water, g.

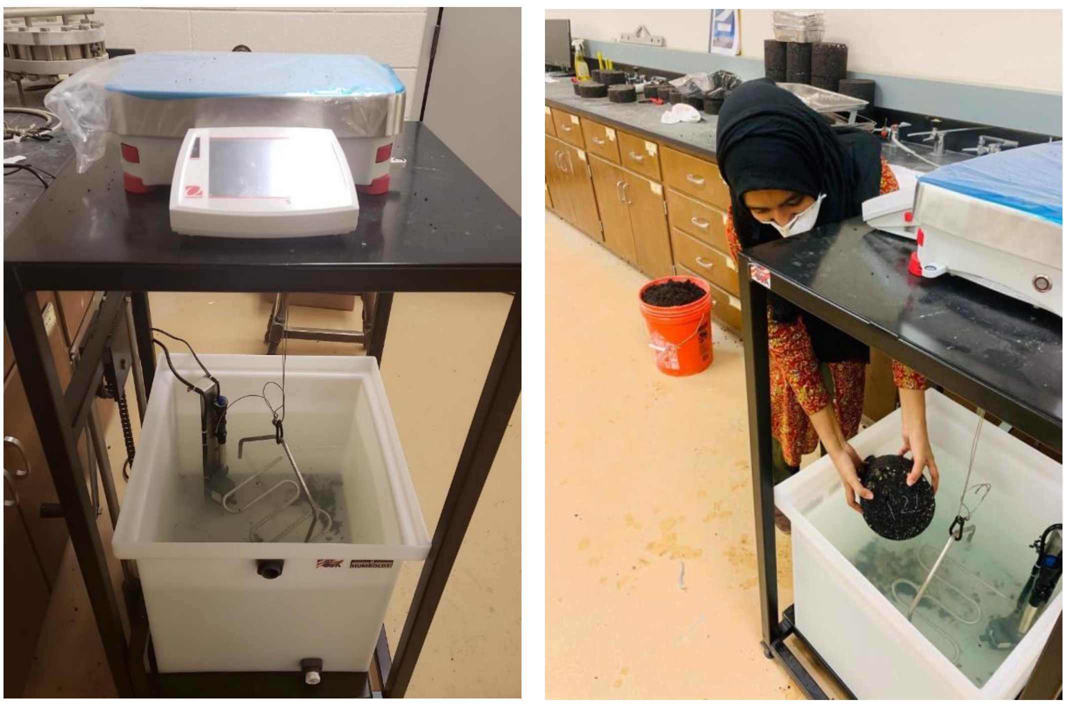


Figure 3.14. Determination of Bulk Specific gravity (SSD method)

3.8.2 Rice Gravity Test

Rice gravity test will be performed to determine the theoretical maximum specific gravity. HMA mixtures have a maximum specific gravity (Gmm) when air voids are excluded. The theoretical maximum specific gravity would be the aggregate and asphalt binder specific gravity added together if all the air voids were eliminated from the HMA sample. To obtain a theoretical maximum density, multiply the theoretical maximum specific gravity (62.4 lbs/ft3 or 1000 g/L) by the density of water (1000 g/L). Rice density (based on James Rice's procedure) is then the result. As part of the calculation of percent air voids in HMA, the theoretical maximum specific gravity is a critical HMA characteristic. Both Superpave mix design and void detection in-place are determined by this calculation.

The theoretical maximum specific gravity of HMA can be determined by weighing a sample of loose HMA (i.e., not compacted), then calculating the volume it displaces by calculating the weight of water it has (Figure 3.15). The sample weight divided by its volume can then be used to calculate the sample's theoretical maximum specific gravity. The standard theoretical maximum specific gravity test is AASHTO T 209, ASTM D 2041 and Tex-227-F: Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixture. The following equation can be used to calculate the theoretical maximum specific gravity:

$$G_{mm}= \frac{A}{A+D-E}$$

Where,

G_{mm}= Theoretical Maximum Specific Gravity

A = sample mass in air (g)

D = mass of flask filled with water (g)

E = mass of flask and sample filled with water (g)

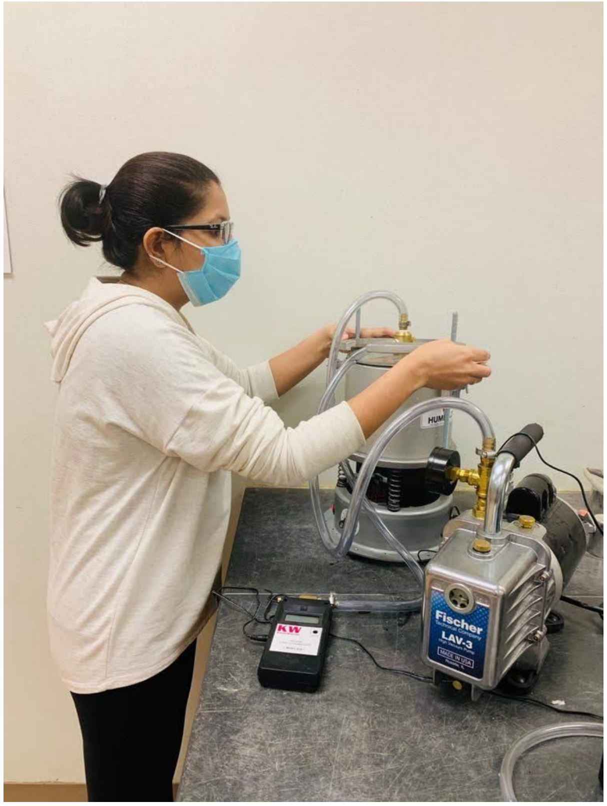


Figure 3.15. Determination of Theoretical Maximum Specific Gravity

3.9 Performance Test

Test that indicates how a mix will resist a particular form of distress is called performance test. Performance tests are used to relate laboratory mix design to actual field performance. Some of these kind of tests are-

1. Rutting test
2. Cracking test
3. Moisture susceptibility test

All the specimens of performance tests will be compacted as 150 mm diameter and 62 mm height maintaining 7% air voids.

3.9.1 Rutting Test

To measure rutting performance of asphalt mixtures different types of laboratory testing can be possible like Asphalt pavement analyzer test, Hamburg wheel track test, Flow number test. Among these three Hamburg wheel track test will be done for rutting test.

Hamburg Wheel Track Test (HWTT)

HWTT has been widely used by highway agencies, such as California, Colorado, Illinois, Iowa, Louisiana, Montana, Oklahoma, Texas, Utah, Washington, and Wisconsin (Mohammad et al. 2015). The HWTT has been found to have an excellent correlation with field performance (especially in moisture damage evaluation). Figure 3.16 shows a Hamburg Wheel Tracking Device. The HWTT is often conducted following AASHTO T324: Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA) or Tex-242-F. Both slab specimens and cylindrical specimens can be used. A loaded steel wheel is tracked on asphalt pavement samples back and forth with the Hamburg Wheel Tracking Device (HWTD), in order to determine rut resistance. As thousands of these cycles are repeated, it simulates the effects of traffic loads on the pavement over time (Rahman and Hossain, 2014). A continuous measurement of the depth of the ruts is made throughout the test. An HWTD test can also be conducted while a sample is submerged in water. Moisture resistance can also be evaluated using this method. The stability of the mix will, at first, determine how quickly rutting develops after the sample has been consolidated by the initial loading cycles. Following a certain number of load cycles (depending on the moisture susceptibility of the mix), damage from stripping accelerates rut development.

Figure 3.16. Hamburg Wheel Tracking Test

To conduct the test two samples of 62 mm height and 150 mm of diameter were compacted for each set of combination. After that the samples were cut 12 mm from their edge to fit them in the Hamburg molds. Then mounting trays with the samples in the molds were placed in an empty water bath. The computer control was activated via a software and

required information entered. Test specifications were as follows:

a) Testing temperature: 122±1.8°F (50±1°C).

b) Load: 158 lb. ± 5 lb. (705±22 N).

c) Number of passes per minute: 50±2.

d) Maximum number of passes setting: 20,000

e) Maximum speed of wheel: 1.1 ft./sec (approximately)

f) Maximum rut depth: 20 mm

g) Rut-depth measurements: every 100 passes.

Upon reaching the desired temperature, water was turned on and the specimen was soaked for an additional 30 minutes. As soon as the specimen was saturated, the arms with wheels were lowered until they rested on it. The device stopped automatically when the maximum rut depth or the maximum number of wheel passes were reached, whichever occurred first. Linear variable differential transducers (LVDTs) connected to the machine on either side measured vertical deformation (rut depth) at 11 different points along the wheel path of the specimen. The HWTD device was connected to a computer-based automated data acquisition system for measuring rut depth. By plotting the number of wheel passes against rut depth, we determined the post-compaction slope, the creep slope, the stripping inflection point, and the stripping slope.

3.9.2 Cracking Test

Two of most common cracking tests are Texas Overlay Tester (OT) and the Indirect Tension test. These test methods determine the susceptibility of bituminous mixtures to fatigue or reflective cracking. So to determine the tensile strength of compacted bituminous mixtures these tests will be done for this research.

Indirect Tensile Strength Test

The tensile strength of HMA is important since it can be used as an indicator of cracking. The high tensile strength at failure of a particular HMA means that it can tolerate higher strains before failing, implying it is likely to resist cracking better than one with a low tensile strength at failure.

Zhou et al. (2017) have developed an indirect tension test for asphalt cracking that requires no cutting, no drilling, no gluing, no notching, no instrumentation, minimal temperature conditioning, and minimal testing time. ASTM D6931 – 12 or Tex-226-F can be adopted to conduct this test. The loading head is a strip conforming to that required for a simple indirect tension (IDT) test (Figure 3.17). The only instrumentation required is a load cell capable of applying a compressive load at a controlled deformation rate of 2 in. per minute and loading strips, consisting of 0.5 × 0.5 in. square steel bars for 4 in. diameter specimens, and 0.75 × 0.75 in. square steel bars for 6 in. diameter specimens. The tensile strength can be calculated as follows:

$$\text{Tensile strength, } S_t = \frac{2P}{\pi D t}$$

Where, S_t = tensile strength (psi)

P = maximum load (lbs)

t = sample thickness (inches)

D = sample diameter (inches)

Figure 3.17. Test Set-Up for Indirect Tensile Strength Test

Overlay test

Rehabilitation of old asphalt and concrete pavements using asphalt overlays is common. A major distress of HMA overlays is the reflective cracking which occurs right above the underlying crack or the concrete slab joint, which is a significant problem with HMA overlays. As a result of daily temperature and moisture cycles, or traffic load, reflective cracking is caused by stresses concentrated in the overlay, which are caused by bending of joints and/or shearing movements at cracks (Hu et al. 2010).

Germann and Lytton (1979) developed the overlay tester in the 1970s to predict the cracking resistance of asphalt overlays. Research at the Texas Transportation Institute (TTI) has refined the model (Zhou and Scullion, 2003, Zhou and Scullion, 2005a). The overlay test has also been evaluated to determine the asphalt mixture resistance to fatigue cracking and low temperature cracking (Zhou et al. 2007b, Walubita et al. 2011, Zhou and Scullion 2003). While the test procedure is continuously being improved, the overlay test has been used to evaluate the cracking resistance of asphalt mixtures in the mixture design process and in the proposed asphalt overlay thickness design and analysis tool in Texas (Hu et al. 2011, Hu et al. 2010, Walubita et al. 2012, Hu et al. 2008). The overlay tester is illustrated in Figure 3.18 (Zhou et al. 2007a).

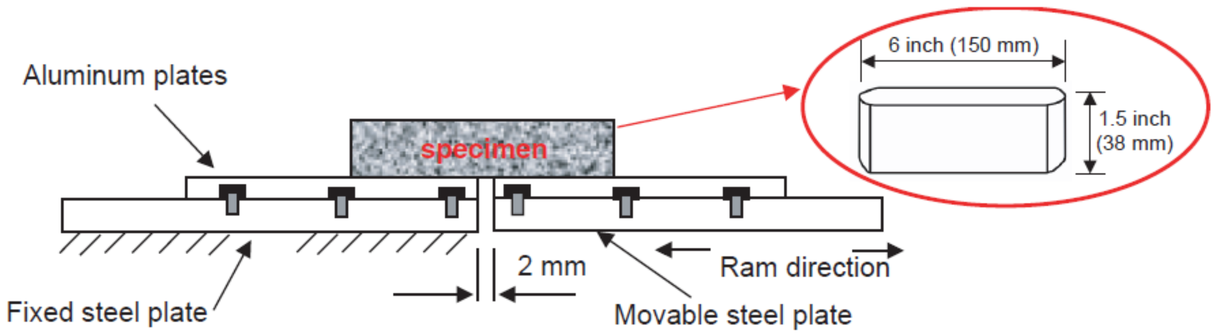


Figure 3.18. Concept of Texas Overlay Tester (Zhou et al. 2007)

The test specimen (150 mm long by 76 mm wide by 38 mm high) can be trimmed from a laboratory-molded specimen or a field core according to Tex-248-F-09 procedure (Figure 2.7). The laboratory-molded specimen should be compacted to 150 mm (6 in.) in diameter and 115 ± 5 mm (4.5 ± 0.2 in.) in height. The size requirement for a field core is 150 ± 2 mm (6 ± 0.1 in.) in diameter and at least 38 mm (1.5 in.) in height. An trimmed laboratory-molded specimen must have air voids of 7 ± 1 %. Following trimming of the specimen, it is glued to the plate with 4.5kg (10lbs) weight on top, after which the test can begin after the glue has cured (Figure 3.19).

Figure 3.19. Gluing and curing the OT samples

Two steel base plates are glued to the test specimen. It is necessary to fix one plate, then slide the other horizontally until the specimen fails. Every 0.1 second, the tensile load and displacement of the moving plate are measured (Tex-248-F-09). An electronic load cell is built into the Texas Overlay Tester. This load cell can measure up to 25 KN (5000 pounds) of load. Tests are conducted with controlled displacement. The moving plate is subjected to a continuous saw-tooth load to maintain a constant maximum opening displacement of 0.635 mm (0.25 in.). At a rate of 10 seconds per cycle, the load is continuously applied until the peak load is reduced by at least 93%

relative to the peak load at the first cycle (Figure 3.20). Similarly, if it is conducted for 1000 cycles without reaching 93 percent reduction, the test will be terminated (Zhou and Scullion 2003, Zhou and Scullion 2005a).

Figure 3.20. Overlay Test setup

3.9.3 Moisture Susceptibility Test

Water-induced damage of asphalt mixtures has produced serious distress, reduced performance, and increased maintenance for pavements in Texas, as well as in other areas of the United States (FHWA report, 1984). Two laboratory tests have received acceptance in United States to evaluate the moisture sensitivity of HMA: the Lottman procedure (AASHTO T 283) and the HWTT (AASHTO T 324) (Solaimanian et al. 2003). There exists also a TxDOT designation (Tex-531-C) of doing this test. The procedure will subject some molded specimens to moisture conditioning and will compare them by indirect tensile strength to unconditioned specimens (Figure 3.21). This is called the tensile strength ratio (TSR) of a mix. The TSR is, therefore, an indication of loss of strength caused by the moisture conditioning. The TSR value must be greater than 0.7 to be moisture resistant. The TSR value can be calculated as follows:

$$\text{TSR}= \frac{S1}{S2}$$

Where,

TSR = tensile strength ratio

S_1 = average tensile strength of unconditioned samples

S_2 = average tensile strength of conditioned samples

Figure 3.21. Conditioning (freezing thawing) the samples to determine TSR

As stated earlier, the stripping potential can be measured by conducting the rutting test under water. To this end, the concept of stripping inflection point (SIP) is based on rutting vs. wheel pass curves with a sudden rut depth increase when the number of passes increases. At this point, asphalt binder is believed to separate from aggregates.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

The results obtained from bulk density and maximum density, Hamburg rutting test, Indirect tensile strength, Overlay test and moisture susceptibility test are presented and analyzed here in this chapter. Test results are analyzed in terms of changing plastic and RAP content.

4.2 Grain Size Distribution

Based on Tex-110E specifications, the distribution of aggregate particles was quantified using the sieve analysis method. Sieve sizes were in accordance with the standard specifications. If less than 1% of the aggregate passed the No. 200 sieve, then hydrometer analysis was not required.

Particle size distribution for all types of materials was done by sieve analysis as per standard test method specified in TxDOT guidelines (Tex- 110E) for particle size analysis of aggregates. Recycled base materials were subjected to gradation to check its conformity with TxDOT standard. According to the Tex-110E guidelines, gradation of the base materials was determined, and the sample gradation results are shown in Figure4.1. Sieve analysis shows that about 99 percent of the materials retains on No. 200 276, no Hydrometer analysis is required if percent passing on No. 200 sieve is less than 1% and hence no Hydrometer analysis was performed. Atterberg limits were not also done because of the same reason. Through the sieve apparatus, a quantified amount of material was poured to transfer it from one sieve to other. The weight of materials retained on each sieve was measured before calculating the percentage of materials passing through the sieve. By dividing the weight of material retained on each sieve by the total weight of the sample, the percentage of the materials retained on each sieve was obtained. The amount of material that passed through each sieve was calculated by deducting the percentage retained on each sieve from 100%. The particle/grain size distribution curve was obtained by plotting the percent of materials that passed through each sieve against the size of sieve on a semi-log graph.

As per TxDOT recommendation, a Superpave SP-C aggregate gradation is chosen to conduct this research. In this research, two mix designs where mix design 1 has 25% Type C rock, 30% Type D rock, 30% Man sand, 15% recycled asphalt pavement (RAP) and mix design 2 has 25% Type C rock, 28.5% Type D rock, 21.5% Man sand, 25% recycled asphalt pavement (RAP) are chosen to proceed the research. Figure 4.1 and 4.2 are showing the aggregate gradation according to Superpave mix design where the particle size distribution for all types of materials was done by sieve analysis as per standard test method specified in TxDOT guidelines (Tex- 110E). The nominal maximum aggregate size (NMAS) for both mix design is found 12.5 mm. The gradation of the chosen aggregates meets the requirement as specification of TxDOT (Item 344).

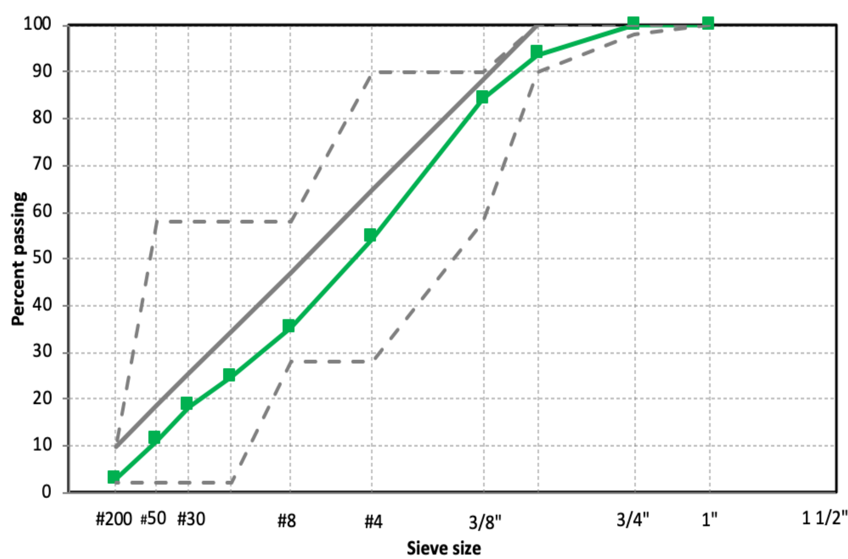


Figure 4.1. Superpave SP-C Aggregate Gradation for Mix Design 1

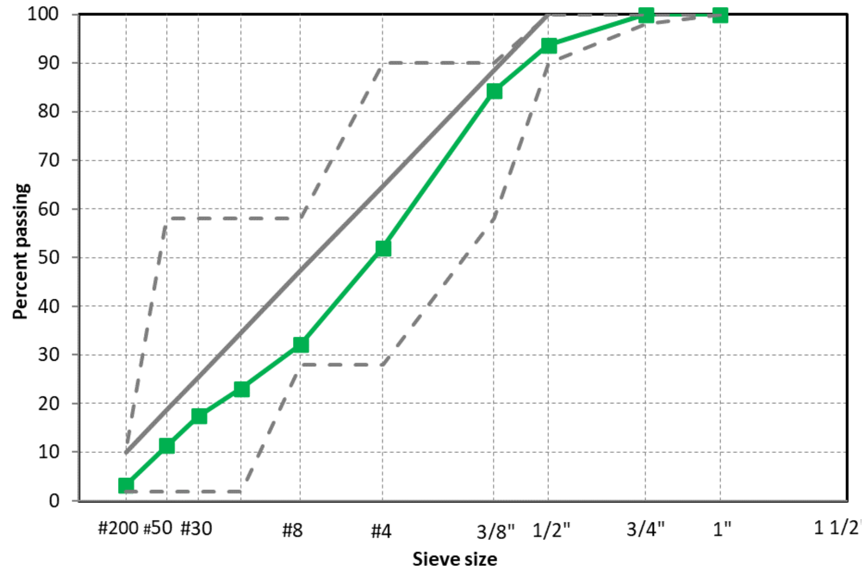


Figure 4.2. Superpave SP-C Aggregate Gradation for Mix Design 2

4.3 Optimum Bitumen Content (OBC)

At the beginning of the research, it is necessary to determine the optimum bitumen content of the control mix. Once this optimum bitumen content is fixed, further testing will be run by taking out some percentage of the optimum bitumen content and add the same amount of plastic into it to replace the bitumen. Superpave mix design method was used for determining the OBC of the mixtures. The Superpave Gyratory Compactor (SGC) with consolidation pressure of 600 kPa, angle of gyration 1.16° and speed of gyration 30 rpm was used to prepare the 150 mm diameter of cylindrical specimens (Priyanka et al., 2018). For the given traffic density and climatic conditions, the compactive effort is a function of design number of gyrations (N_{des}). N_{des} of 50 gyrations corresponding to high traffic (design ESALs ≥ 30 millions) was considered according to the TxDOT manual.

Loose Superpave mixtures were prepared to determine the maximum theoretical density (G_{mm}) and the test was conducted as per ASTM D 2041 (2011). Volumetric properties of Superpave mixtures were determined for all asphalt contents using cylindrical specimens. For each mixture, a minimum of two specimens were prepared and the average was considered. The properties including bulk density (G_{mb}), air voids (V_a), voids in mineral aggregates (VMA) and voids filled with asphalt (VFA) were calculated, and the asphalt content corresponding to 4% air void was selected as the OBC. According to Superpave mix design it is recommended to ensure 4% air void in laboratory. If the air void is more the pavement is compromised in terms of pavement strength, fatigue life, durability, raveling, rutting and susceptibility to moisture damage. And if air voids fall below 3%, there will be inadequate room for expansion of the asphalt binder in hot weather. When the void content drops to 2% or less, the mix becomes plastic and unstable. Table 4.1 and 4.2 are showing all the specific gravity and corresponding air voids to determine optimum bitumen content. For control mix, conducting the volumetric tests the optimum bitumen content is found 4.8% for mix design 1 and 5.3% for mix design 2 (Figure 4.3 and Figure 4.4) respectively.

The volumetric properties of mix design 1 and mix design 2 at corresponding OBC are presented in Table 4.1 and Table 4.2, and these values are within the specified Superpave mix design requirements. Volumetric properties of Superpave mixtures at OBC also calculated and shown in Table 4.3.

Table 4.1. Air void calculation to determine optimum bitumen content for mix design 1

Asphalt Content %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
4	2.464	2.649	6.98
4.5	2.482	2.613	5.01
5	2.477	2.576	3.84
5.5	2.497	2.544	1.85
6	2.513	2.521	0.32

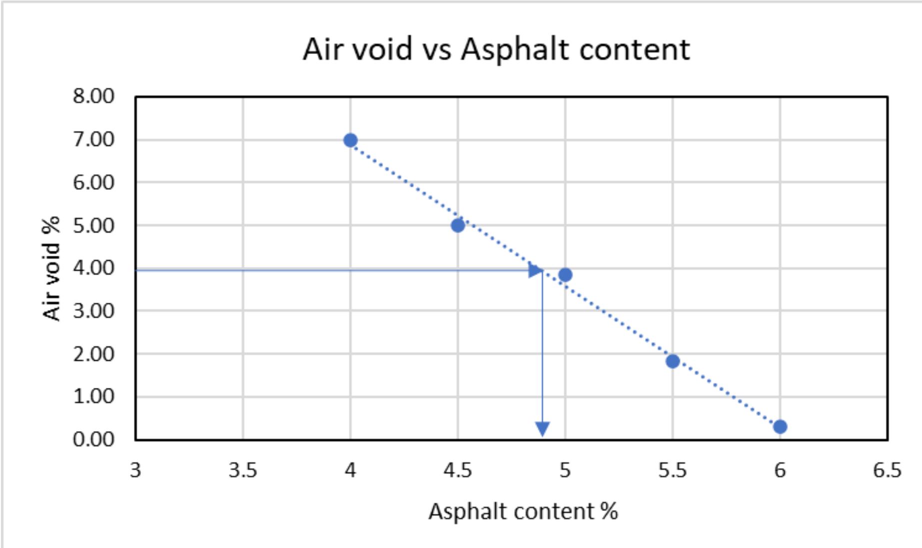


Figure 4.3. Determination of optimum bitumen content of mix design 1

Table 4.2. Air void calculation to determine optimum bitumen content for mix design 2

Asphalt Content %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
4	2.404	2.609	7.86
4.5	2.433	2.585	5.88
5	2.455	2.567	4.36
5.5	2.459	2.549	3.53
6	2.472	2.526	2.14

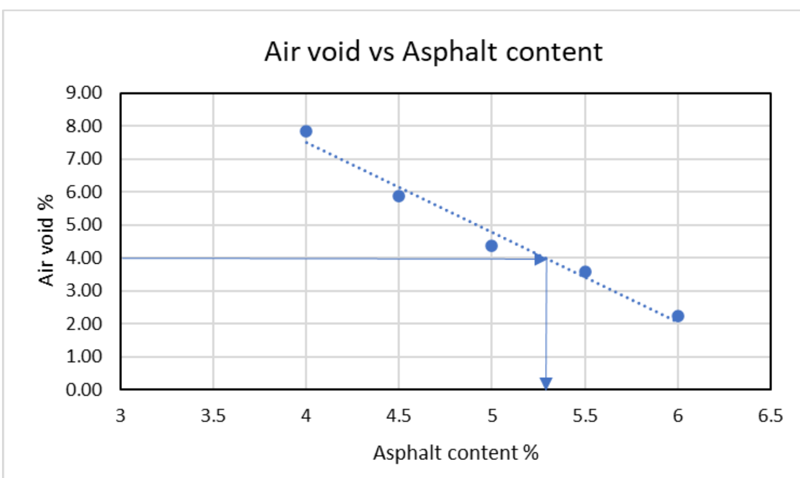


Figure 4.4. Determination of optimum bitumen content of Mix Design 2

Table 4.3. Volumetric properties of Superpave mixtures at OBC

Property	Mix Design 1	Mix Design 2	Requirement
OBC (%)	4.8	5.3	-
G_{mb} (g/cc)	2.479	2.457	2.2-2.5
G_{mm} (g/cc)	2.591	2.565	2.4-2.7
VMA (%)	15.6	15.8	>13
VFA (%)	74.3	74.6	65-75
Dust Proportion	0.6	0.6	0.6-1.2

4.4 Volumetric Analysis

After getting the optimum bitumen content different percentage of plastic are added into the mix to replace that amount of bitumen. Two volumetric tests: Bulk density and Rice gravity test were performed according to the Tex-207 Part 1 and Tex-227-F respectively. For the bulk density tests, minimum two samples of 115 mm height and 150 mm diameter are compacted with Superpave Gyratory Compactor. For the Rice gravity tests, two loose sample of minimum 1500 gm are used and tested in the laboratory. From the results of these two test air voids of the sample are calculated.

$$\text{Air void (\%)} = (1 - \frac{Gmb}{Gmm}) * 100$$

Where, G_{mb}= Bulk specific gravity (gm/cc)

G_{mm}= Maximum specific gravity (gm/cc)

At first, LDPE plastics are taken into account to start the test. After doing the bulk density and rice gravity test with mix design 1, a trend of air void at different plastic content is observed. The laboratory air void should be between 3-5%. However, from Figure 4.5. It is seen that the air void didn’t fall within the acceptable requirement while using LDPE.

Table 4.4. Volumetric analysis of plastic content (LDPE) (Mix Design 1)

Plastic %	Bitumen %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
4	96	2.459	2.613	5.89
8	92	2.448	2.582	5.19
12	88	2.435	2.561	4.92
16	84	2.428	2.555	4.97

To keep the air void within the range a new combination has been considered where 0.5% PP is mixed as aggregate replacement while different percentage of LDPE used as bitumen replacement. Here, the air void is decreasing with the increase of plastic content up to some percentage since plastic is filling up the internal voids. From Table 4.5, it is found that the air voids are in range up to 12% LDPE mix along with 0.5% PP mix as aggregate. It can be said that this mix contains less air voids than only LDPE mix.

Table 4.5. Volumetric analysis of plastic content (LDPE+0.5%PP) (Mix Design 1)

Plastic %	Bitumen %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
4	96	2.459	2.573	4.43
8	92	2.476	2.582	4.11
12	88	2.447	2.574	4.93
16	84	2.428	2.575	5.71

Similarly, PP and HDPE plastics are used to determine the air void where air void of PP mixed samples is within the range up to 8% Plastic usage (Table 4.6) and air void of HDPE plastic is within the limit up to 12% of replacement (Table 4.7). The air void is increasing with the increase of plastic content (beyond 12%) as the addition of more plastics can be dispersed which causes more air voids in the mix.

Table 4.6. Volumetric analysis of plastic content (PP) (Mix Design 1)

Plastic %	Bitumen %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
4	96	2.498	2.593	3.66
8	92	2.467	2.585	4.56
12	88	2.449	2.583	5.19
16	84	2.428	2.577	5.78

Table 4.7. Volumetric analysis of plastic content (HDPE) (Mix Design 1)

Plastic %	Bitumen %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
4	96	2.49	2.613	4.71
8	92	2.49	2.594	4.01
12	88	2.46	2.582	4.73
16	84	2.42	2.568	5.76

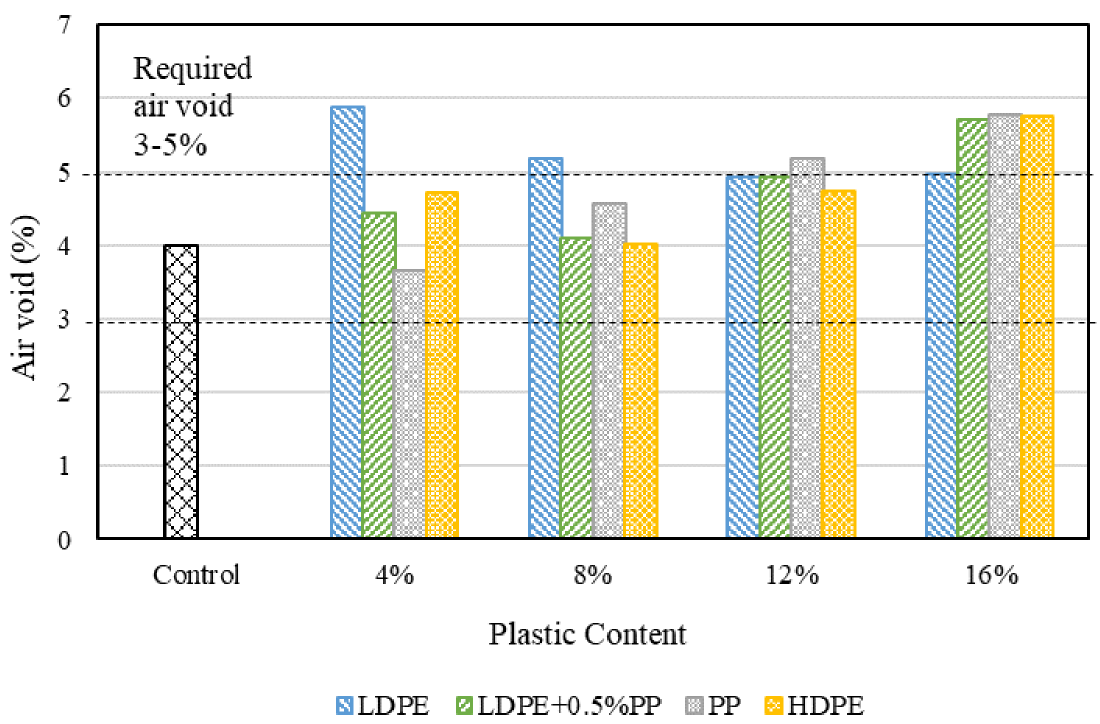


Figure 4.5. Air void of the mix in different plastic combinations for mix design 1

In this study, several variables are expected to affect the performance of Superpave mixes. Besides the effect of plastic types and plastic percentage, change in RAP content may also has effect on the performance of the asphalt mix. To further study this behavior, mix design 2 is considered in this study where the RAP content is taken 25%. Since the LDPE only mix didn't show good result in mix design 1, this combination is discarded for mix design 2. Bulk density and maximum density are calculated to determine the air void of the mix containing LDPE along with 0.5% PP as aggregate replacement, PP and HDPE at different percentage (Table 4.8). From Figure 4. It is observed that asphalt mix containing PP and HDPE has the air void within the limit up to 8% of bitumen replacement (Figure 4.6). On the other hand, LDPE mix with 0.5% PP has only passed the requirement up to 4% usage.

Table 4.8. Volumetric analysis of plastic content (Mix Design 2)

Plastic Type	Plastic %	Bitumen %	Bulk specific gravity, G_{mb}	Maximum specific gravity, G_{mm}	Air void %
LDPE+0.5%PP	4	96	2.466	2.571	4.08
	8	92	2.432	2.565	5.19
	12	88	2.428	2.572	5.6
	16	84	2.419	2.591	6.64
PP	4	96	2.431	2.545	4.48
	8	92	2.403	2.522	4.72
	12	88	2.428	2.559	5.12
	16	84	2.429	2.589	6.18
HDPE	4	96	2.495	2.606	4.26
	8	92	2.476	2.582	4.11
	12	88	2.455	2.595	5.39
	16	84	2.418	2.571	5.95

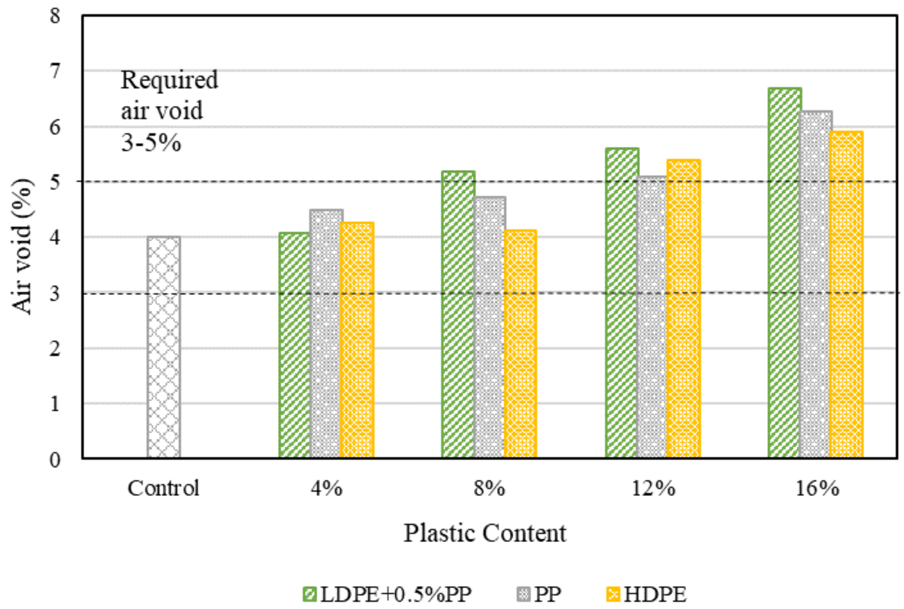


Figure 4.6. Air void of the mix in different plastic combinations for mix design 2

4.5 Rutting Analysis

The Hamburg wheel tracking test (HWTT) is a laboratory procedure that uses repetitive loading in the presence of water and measures the rut depth induced in an asphalt mixture with increasing load cycles. After the results have been processed, moisture susceptibility and rutting resistance of the asphalt mixture are determined. For this test, two samples of 150 mm diameter and 62 mm height are molded and they are cut by 12.5 mm from the edge. The samples are then placed in the Hamburg Wheel Tracker which runs up to 20000 cycles or rutting up to 12.5 mm whichever comes first.

The most significant advantage of adding plastics in asphalt mixture was observed in case of rutting reduction. To observe that, Hamburg wheel tracking test is done using different combination of plastic in bitumen mix. Results obtained from the test results for different types of plastic with mix design 1 are showed in Figure 4.7. It is observed that the rut depth is significantly decreasing with the increase of plastic content. Similar trend found in the study of Mashaan et al., (2021) where 4-8% plastic was used, and rut depth was reduced with the increase of plastic content.

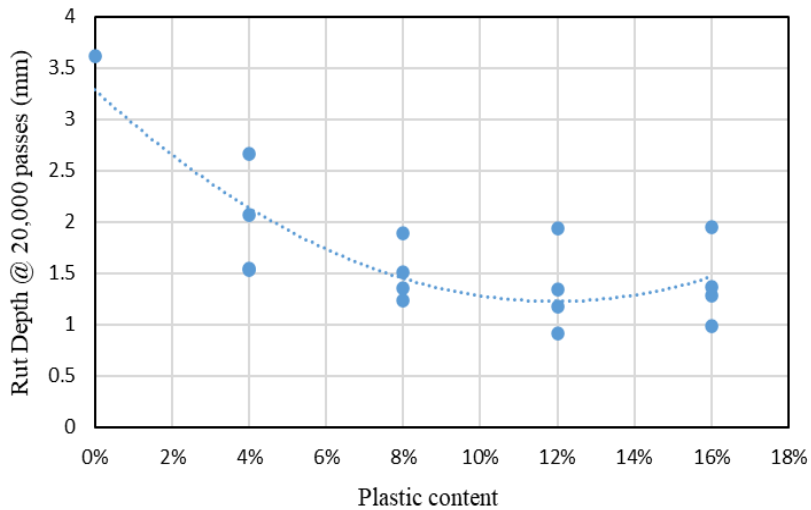


Figure 4.7. Determination of rut depth with different plastic content

The change of rut depth with the number of cycle is shown in Figure 4.8. Where it can be seen that rutting depth of control sample is found 3.62 mm whereas all the plastic sample of 4% to 16% show a rut depth less than the control sample. Based on these results, plastic might be able to improve the mixture's deformation resistance. Plastic and asphalt have a wide variety of physical and chemical properties which may explain the results. Melting and mixing of plastic bitumen mix affects the chemical properties, which will result in changes in particle dimensions. As a result, the engineering properties of Plastic modified asphalt would be enhanced and improved with regards to elasticity. Groupings of molecules and their bonds are responsible for the increase in elasticity (Ameri et al., 2017; Sojobi et al., 2016).

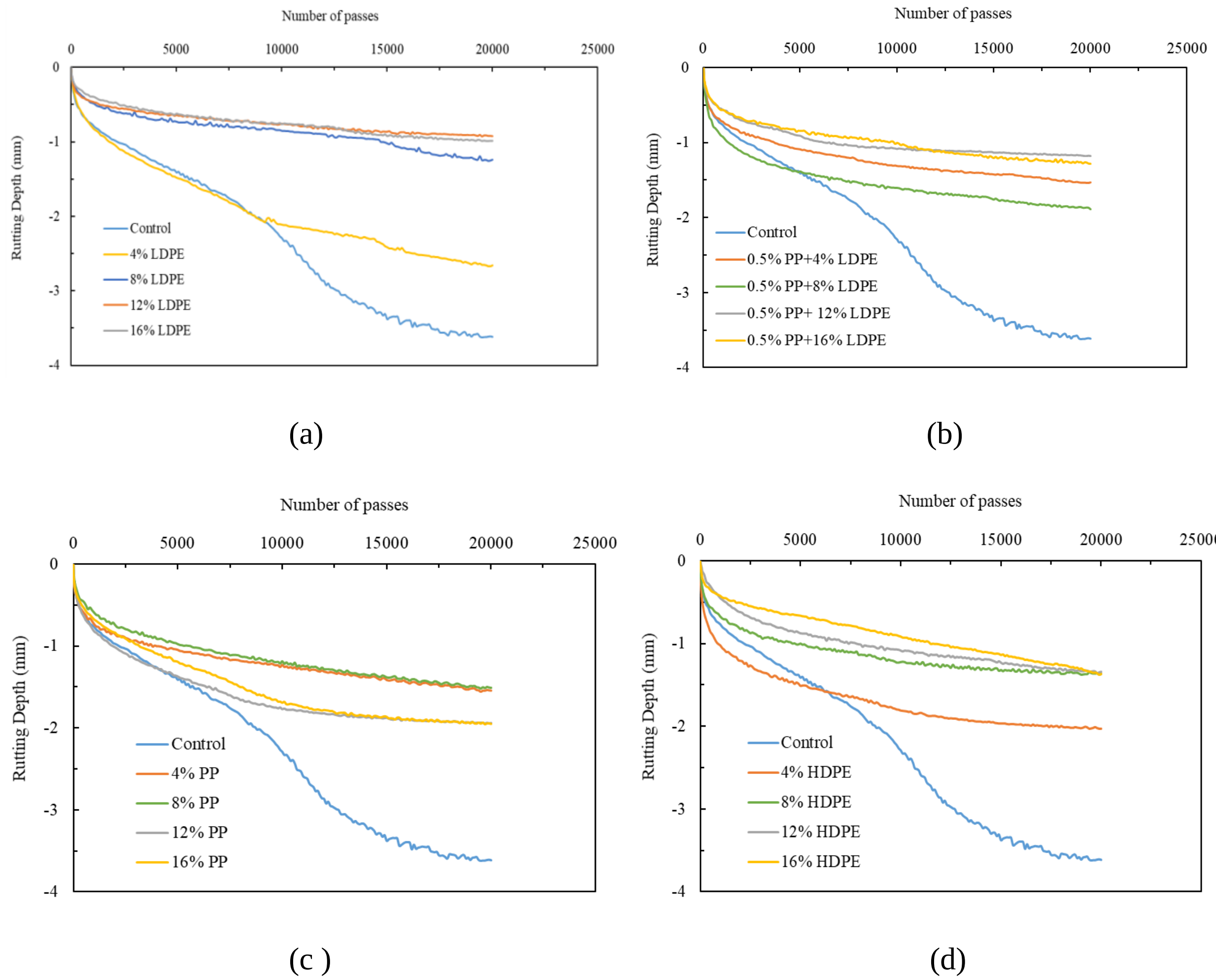
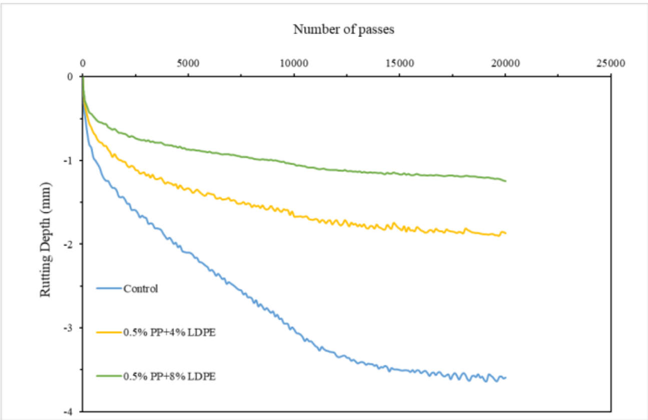


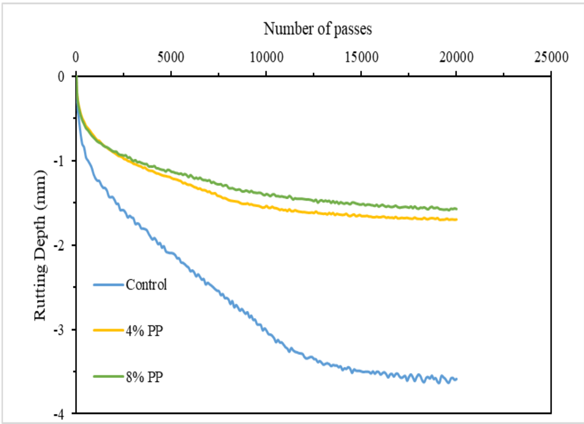
Figure 4.8. Change of rut depth with number of cycles (a) LDPE only (b) LDPE+0.5%PP (c) PP (d) HDPE for mix design 1

However, the mix using only LDPE becomes stiffer while replacing more than 4% bitumen (Figure 4.8 (a)). In order to make this mix less stiff, 0.5% PP was incorporated to the mix keeping in mind that PP would be flexible than aggregate, and this amount of aggregate replacement may make the mix flexible. As a result, while modifying the mix with 0.5% PP as aggregate replacement it is comparatively showing flexible behavior with larger rutting value (Figure 4.8 (b)). Similarly, bitumen mix PP and HDPE plastics also show flexible behavior compared to only LDPE used mix. By analyzing the behavior of the plastics in rutting result it can be concluded that only LDPE is not making good bonding or coating with the aggregate and bitumen which leads to poor air void and rutting resistance in the bitumen mix.

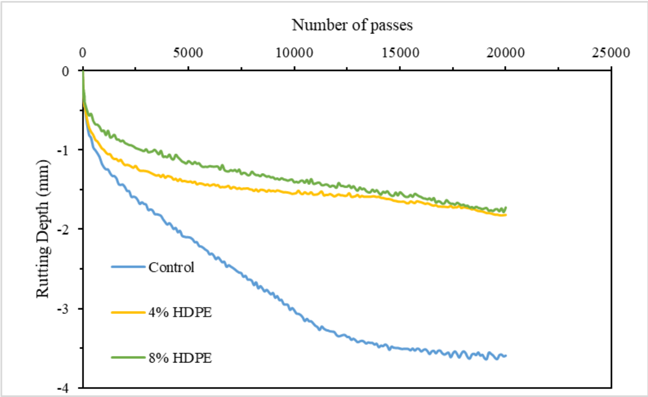
For this reason, in the testing with mix design 2, LDPE only combination was discarded. Moreover, for this mix design plastic are used up to 8% since air void requirement didn't fulfil while exceeding this amount. Like the mix design 1, this mix with higher RAP content also shows lower rutting while incorporating plastic in the mix (Figure 4.9). Also, rutting depth decreases with increase of plastic content. As a result, no significant change in rutting observed due to higher RAP content.



(a)



(b)



(c)

Figure 4.9. Change of rut depth with number of cycles (a) LDPE+0.5%PP (c) PP (d) HDPE for mix design 2

As observed, with the addition of plastics, the rutting depth of the pavement can be reduced up to 75% for mix design 1 and 65% for mix design 2 (Figure 4.10 and Figure 4.11). Therefore, indicating that utilization of plastic modified bitumen mix can significantly enhance the longevity of flexible pavements.

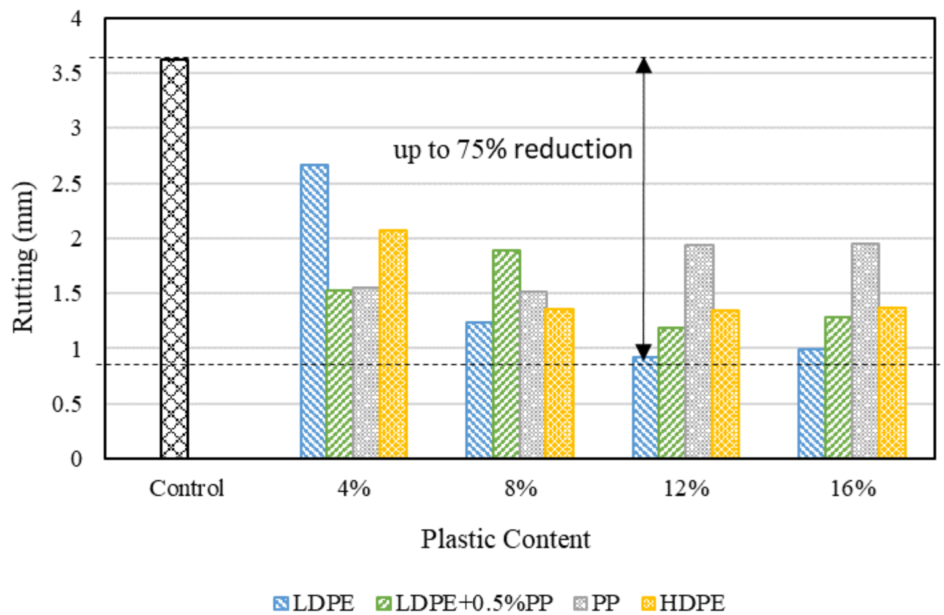


Figure 4.10. Reduction of rut depth for mix design 1

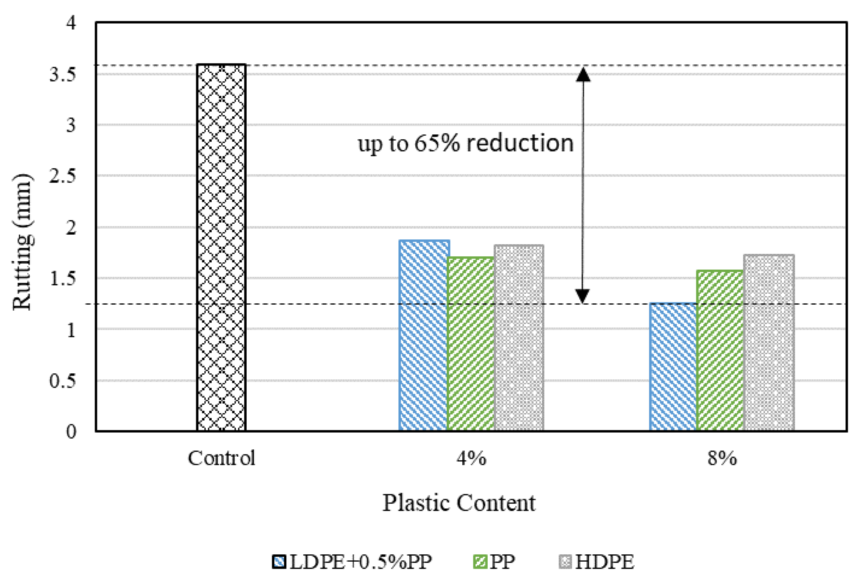


Figure 4.11. Reduction of rut depth for mix design 2

Currently, HWTT parameters such as stripping inflection point (SIP) and rut depth at a certain number of load cycles are widely used to evaluate asphalt mixture moisture susceptibility and rutting resistance, respectively. HWTT tests suggest that asphalt mixtures with a higher SIP value and lower rut depth will perform well. Figure 4.12 Illustrated the determination of SIP value from the rut vs load cycle curve. In this study, the slope of control mix rutting line shows an abrupt change in slope which leads to lower SIP value for the control mix. The stripping point of control mix is found 7000 cycle for mix design 1 (Figure 4.13) and 8500 cycle for mix design 2 (Figure 4.14) whereas no stripping point found in plastic mix. Thus, no moisture susceptibility observed when the control mix is modified with different plastic combination

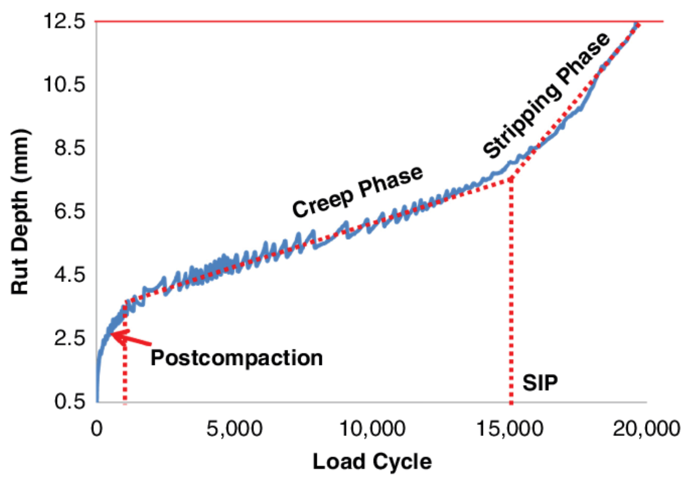


Figure 4.12. HWTT output of rut depth versus load cycle and SIP calculation (Yin et al., 2014)

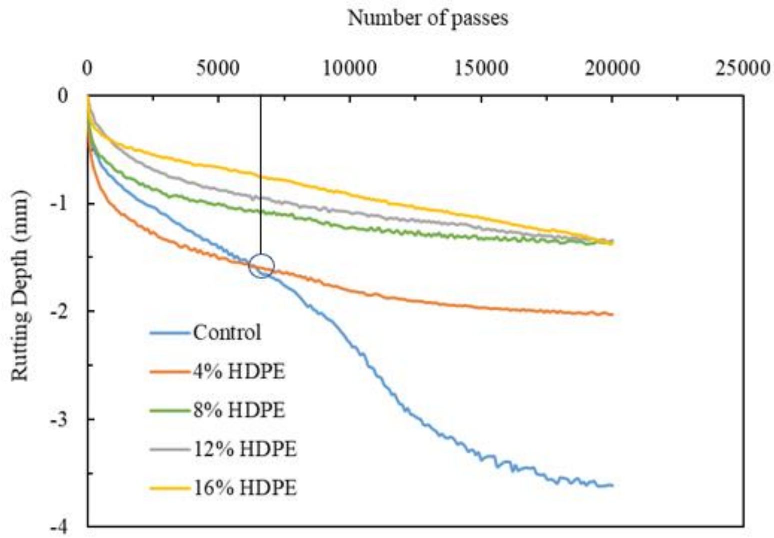


Figure 4.13. Stripping point determination from rut data for mix design 1

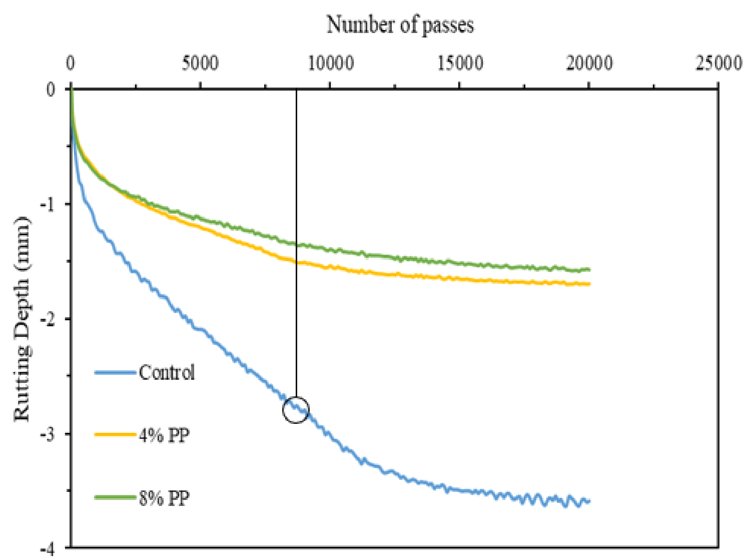


Figure 4.14. Stripping point determination from rut data for mix design 2

The addition of plastic is making a coating over aggregate that helps the mix to be tougher. As a result, when the deformation happens due to traffic wheel load, it sustains to larger load and longer time simultaneously, maintaining very little rut depth. Figure 4.15 is showing samples after completing the rutting test where it can be clearly seen the sample with plastic mix has minimal amount of rutting whereas rutting of control samples is clearly visible.

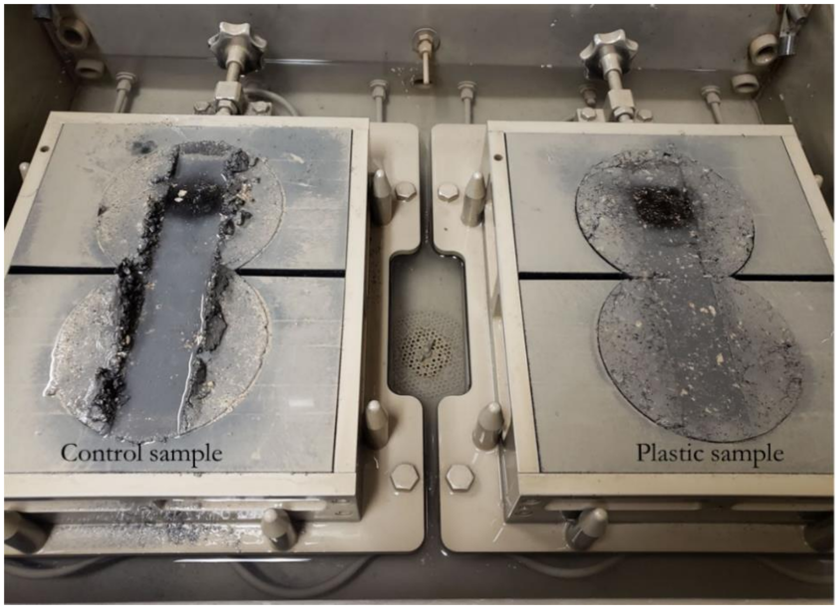


Figure 4.15. Hamburg sample after testing

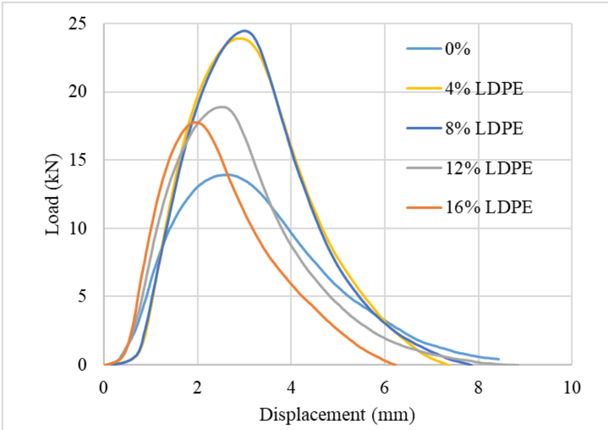
4.6 Indirect Tensile Strength Analysis

The tensile strength of Plastic bitumen mix is investigated by utilizing the load-displacement relationship. Load-displacement curve can be obtained from the indirect tensile strength tests. The value of tensile strength is determined maximum load to compare the values of different combinations of recycled plastic at various RAP contents. To ensure the repeatability of the test results, three replicate specimens of 150 mm diameter and 62 mm height were tested for each material combination. The average value of the three specimens was reported to investigate the variation in IDT with change in plastic types and content for mix design 1 (Table 4.9). From the result, it is seen that at first indirect tensile strength increases with the increase of plastic content, after that it starts decreasing for the higher plastic content. Similar trend is found in the study of Anurag and Rao, 2018; Attaelman et al., 2011; Punith and Veeraragavan, 2007 where they used LDPE and HDPE plastic.

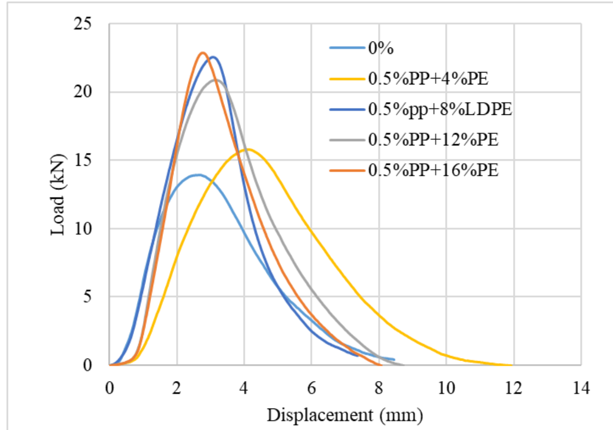
Table 4.9. Determination of Indirect tensile strength (IDT) for Mix design 1

Plastic type	Plastic content	IDT (psi)	Standard deviation
No plastic	Control (0%)	140	4.26
LDPE	4%	241	4.78
	8%	247.2	8
	12%	190	1.41
	16%	179	6.81
LDPE+0.5% PP	4%	159.6	1.03
	8%	227.5	1.44
	12%	211.2	4.93
	16%	231.3	2.47
PP	4%	215.5	8.7
	8%	222	2.6
	12%	194.3	7.7
	16%	181.2	3.1
HDPE	4%	200.3	7.8
	8%	191	8.9
	12%	167.4	4.93
	16%	140.3	2.47

Indirect tensile strength test is done on bitumen mix design 1 using LDPE, LDPE+0.5% PP, PP and HDPE in different percentage. From the Figure 4.16, it is observed that all plastic mix have a tensile strength greater than the control mix. Moreover, it gives the lower rutting and higher strength which is needed for better performance of pavement. Previous literature suggested that the greater ITS value is the indication of the better crack resistance ability (Punith and Veeraragavan, 2007; Hadidy and Yi-qui, 2009; Attaelman et al., 2011). TxDOT also specifies that the ITS value should be greater than 85 psi for having a good performance in cracking. It is seen that all the plastic mix satisfy the TxDOT requirement.



(a)



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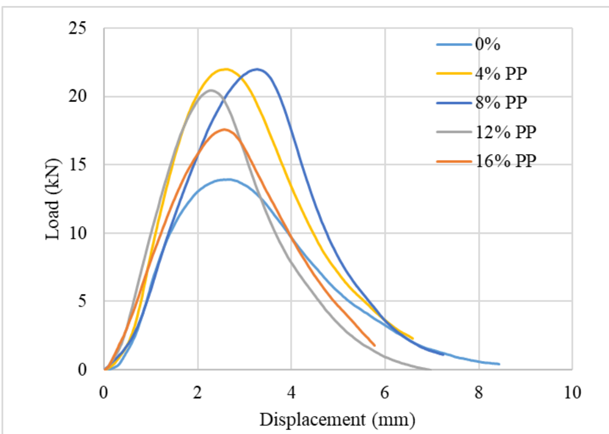

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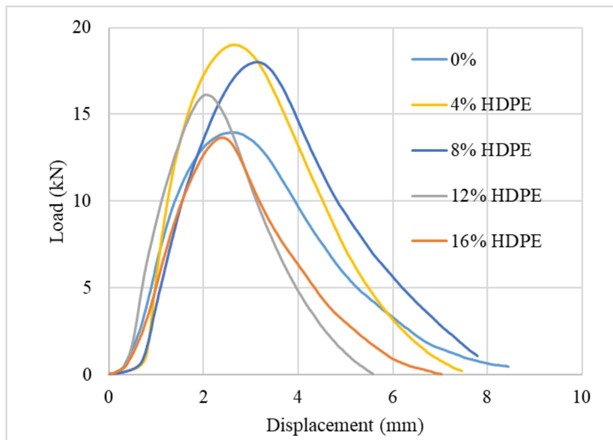

(d)

Figure 4.16. Load vs displacement curve for Indirect tensile test (a) LDPE only (b) LDPE+0.5%PP (c) PP (d) HDPE for mix design 1

Nonetheless, all types of tested plastics have a positive effect on the tensile strength of asphalt mixture. Specifically, addition of up to 8% of LDPE and PP is found to increase the tensile strength of the mixture up to 77% and 58% respectively (Figure 4.17). On the other hand, while using the LDPE+0.5%PP the tensile strength is increasing up to 16% addition of LDPE. With this combination of mix tensile strength can be improved by 66%. While incorporating HDPE in the mix, it can increase the tensile strength as much as 30%. With increasing plastic content, this increase in strength is observed to decrease, indicating that a lower dosage (up to 8%) is more beneficial.

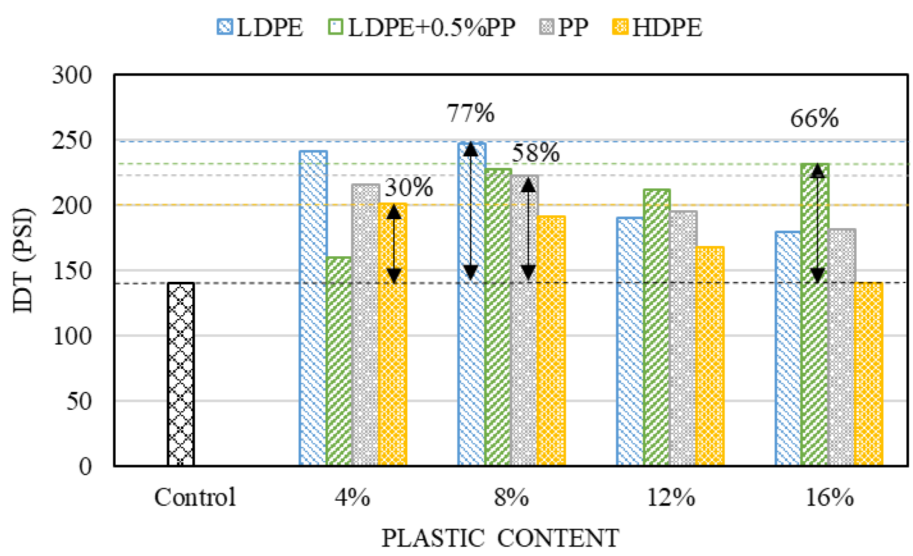

Figure 4.17. Indirect tensile strength with different combination of plastic for mix design 1

Moreover, the energy needed to break the sample is higher in plastic mix up to 8% compared to the control one that makes the plastic mix tougher. The area under the curve represents the energy (work) required to deform the materials which is the definition of toughness. Though all the plastic types show higher work energy up to 8% usage, use of LDPE and PP exhibits higher slope of the curve and less work energy which makes the mix stiffer. On the other hand, using HDPE make the mix tougher compared to LDPE and PP mix because of its flatter slope and higher work energy (Figure 4.18). This behavior will make the mix more sustainable while enduring higher tensile stress. This may happen due to the melting and coating behavior of different plastics. While mixing LDPE and PP they become completely melted and coated over the aggregates. On the contrary, HDPE doesn’t melt completely rather it gets soften and stick together with the aggregates which generally fills the void between the aggregates. Thus, HDPE not only acts as a bitumen replacement but also a filler material which makes the mix tougher than other mix.

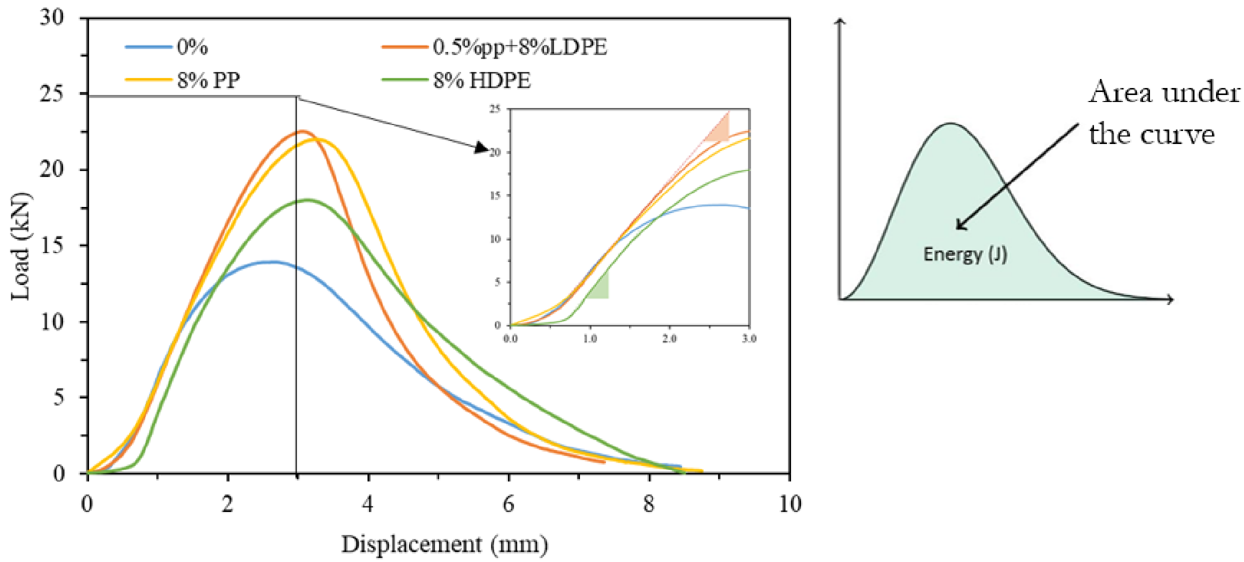

Figure 4.18. Work-energy determination for different plastic types for mix design 1

From the results of mix design 2, it is seen that mix with LDPE+0.5% PP and HDPE are not showing higher tensile strength or higher work energy to break the sample for any amount of usage than the control mix (Table 4.10). Though use of PP exhibits higher tensile strength, the work energy is way less compared to the control mix (Figure 4.19). Thus, increase of RAP content decreases the tensile strength of the bitumen mix and reduce the energy needed to break the sample. It can be concluded that higher RAP percentage won't be beneficial while pairing them with plastic content.

Table 4.10. Determination of Indirect tensile strength (IDT) for Mix design 2

Plastic type	Plastic content	IDT (psi)	Standard deviation
No plastic	Control (0%)	203.9	8.7
LDPE+0.5% PP	4%	201.2	7.4
	8%	181.9	1.7
PP	4%	217.1	6.67
	8%	222.4	11.9
HDPE	4%	189.9	4.3
	8%	181.3	10.18

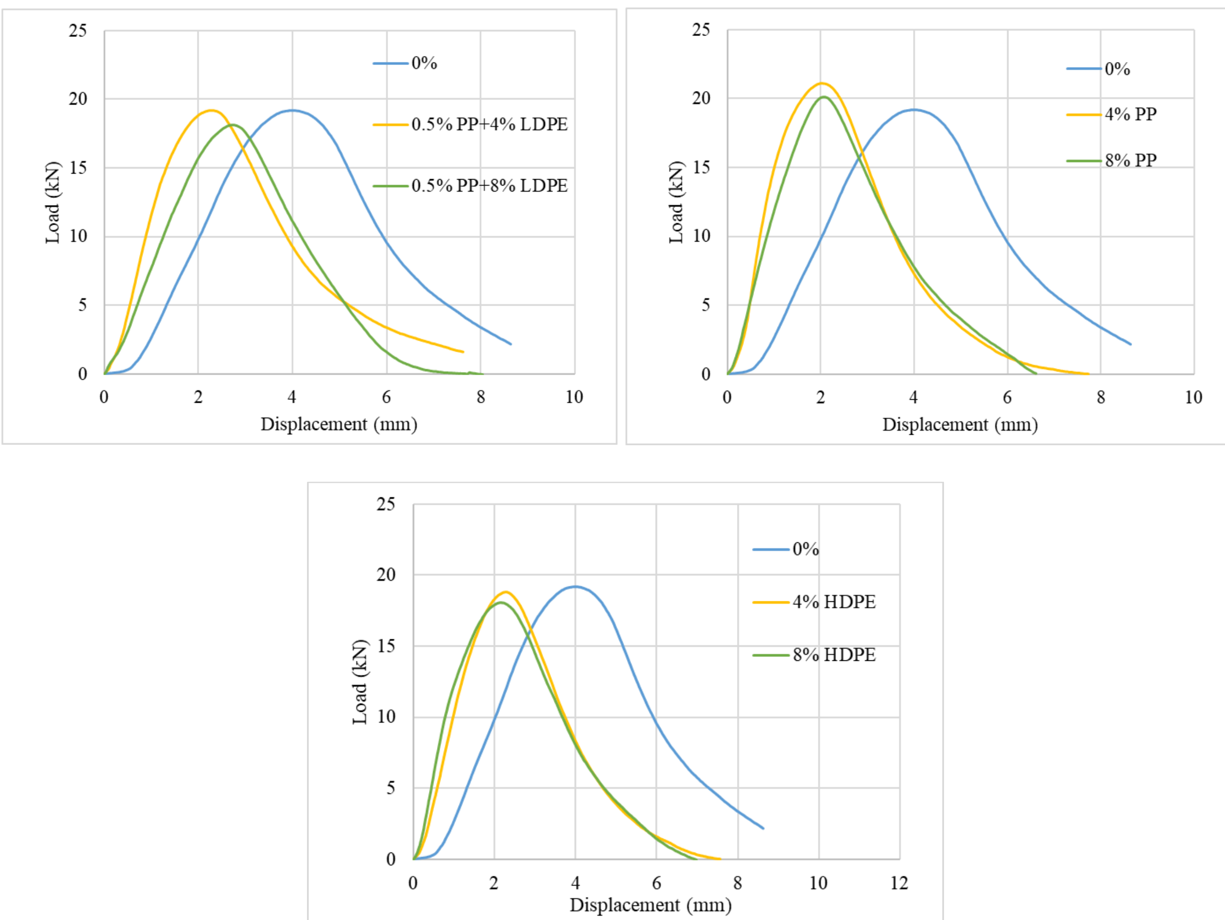

Figure 4.19. Load vs displacement curve for Indirect tensile test (a) LDPE+0.5%PP (c) PP (d) HDPE for mix design 2

4.7 Overlay Analysis

The failure point (number of cycles to failure) in Tex-248-F-09 is determined based on a 93-percent peak load reduction. According to Walubita et al. (2012), it is the point at which a crack has passed through the entire thickness of the specimen. Based on this method, the asphalt mixture design can use the failure point as a pass-fail criterion. The mixture should be redesigned if the failure point (number of cycles to failure) is below the criterion. The overlay test was carried out by Zhou and Scullion (2005a) on selected road sections in Texas with known performance in the field. For Superpave SP-C mixtures, it was suggested that at least 300 cycles be used to distinguish between reflective crack resistant mixtures. Zhou et al. (2007a) reported that the preliminary criterion (300 cycles) for distinguishing the reflective cracking resistance was also reasonable for distinguishing the fatigue cracking resistance of asphalt mixtures. Tests are conducted until peak loads are reduced by 93 percent from their original values or until the termination point is reached (the 1000th cycle) according to the Tex-248-F-09 procedure.

Figure 4.20 depicts the curve associated with the 93% load drop criteria for Mix design 1. For this mix type, the curves associated with the individual mixes were equally distributed around the 93% load drop curve meaning that some of these mixes would have performed satisfactorily while some would have exhibited poor cracking performance.

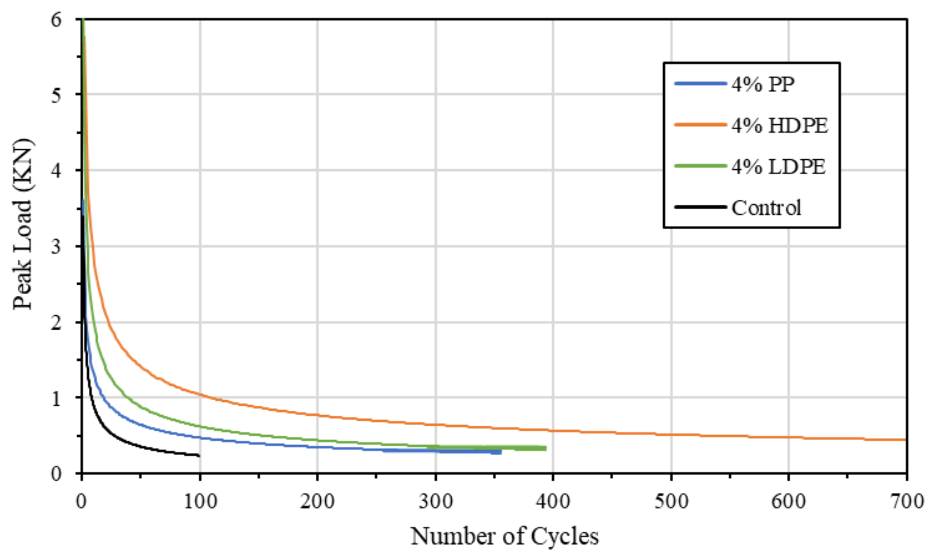

Figure 4.20. Peak vs number of cycles curve generated from OT test results

Three representative samples were tested for each combination to ensure the repeatability of the test where the coefficient of variance (COV) was below 30% for all the combinations. From Figure 4.21 it is observed that control mix only passes the OT up to 98 cycles where Plastic modified mix fulfills the criteria significantly. Though LDPE and PP modified mix pass the OT test while using up to 4%, HDPE mix can sustain in the OT test up to 8% of usage. It can be possible since the HDPE mix becomes more tougher than the other two plastic mix. As a result, it can withstand with the fatigue and reflective cracking significantly.

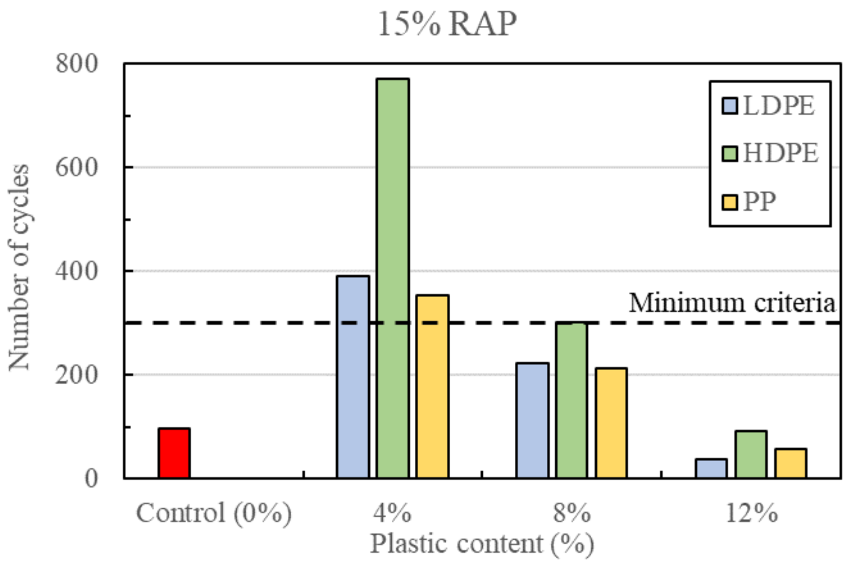

Figure 4.21. Number of cycle passes in OT for mix design 1

Figure 4.22 represents the number of cycle mix design 2 passes in OT. From the plot, it can be seen that due to the excessive amount of RAP content in mix, it cannot pass the criteria of the OT test rather it fails below 300 cycles. However, it is evident that plastic mix can sustain little more than the control sample since the control sample only sustain in the test up to 49 cycles.

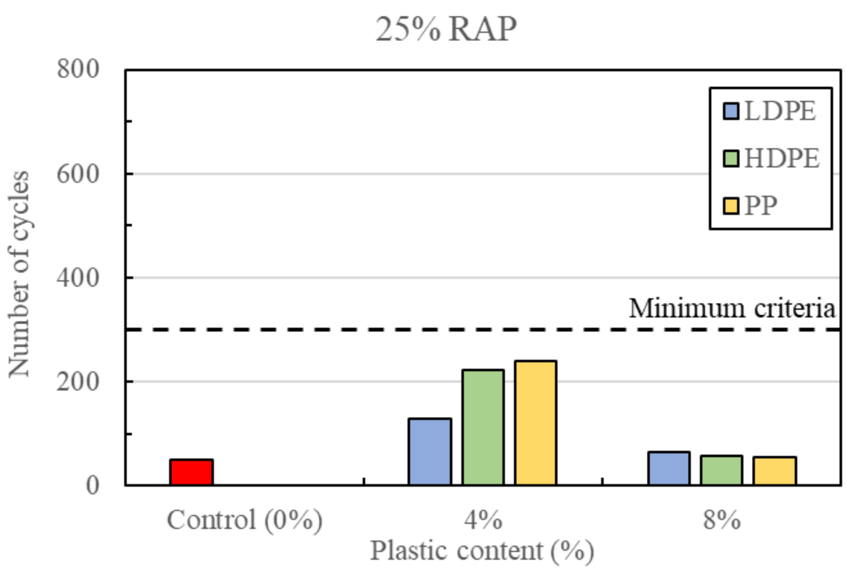

Figure 4.22. Number of cycle passes in OT for mix design 2

Zhou et al., (2007) conducted a study where the OT characterizes crack propagation that can be used for fatigue cracking. This finding has been further validated by field FHWA-ALF fatigue test results. The correlation between the OT results and the FHWA-ALF results from Zhou et al, 2007 is shown in Figure 4.23.

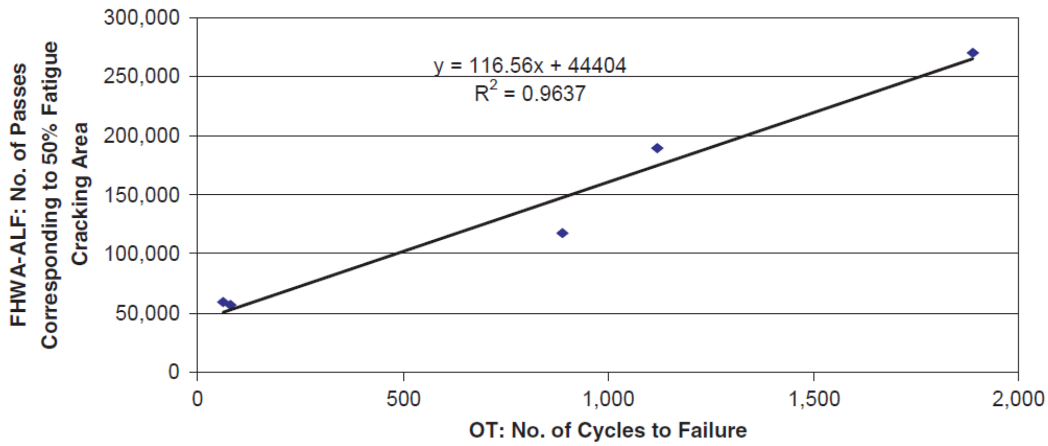

Figure 4.23. Correlation between OT and FHWA-ALF fatigue test (Zhou et al, 2007)

Using the above correlation, the value of this study further analyzed to find out the number of passes a specific mix can withstand in the field. From Table 4.11 it is seen that plastic modified mix can double the service life of a pavement structure.

Table 4.11. Determination of service life of the mix using a correlation by Zhou et a;., 2007

RAP (%)	Plastic Type/Plastic content	Control (0%)	4%	8%	12%
		FHWA-ALF: No. of Passes corresponding to 50% fatigue cracking area			
15%	LDPE	55827	90096	70513	48833
	HDPE		134388	79489	55244
	PP		85783	69231	51281
25%	LDPE	50115	59557	52097	-
	HDPE		70397	51164	-
	PP		72495	50815	-

4.8 Moisture Susceptibility Analysis

To ensure the repeatability of the test results, three replicate specimens of 150 mm diameter and 62 mm height were tested for each material combination. The average value of the three specimens was reported to investigate the variation in conditioned IDT with change in plastic types and content. For this test, the samples are being conditioned under freeze and thaw cycle according to AASHTO T 283 and after that the indirect tensile strength test is performed on the samples. The allowable value for TSR is reported as minimum 0.7.

Table 4.12. Determination of Tensile strength ratio (TSR) for mix design 1

Plastic type	**Plastic content**	**Unconditioned IDT (psi)**	**Conditioned IDT (psi)**	**TSR**
No Plastic	Control (0%)	140	92.2	0.7
LDPE	4%	241	104.7	0.4
	8%	247.2	111.6	0.5
	12%	190	88.1	0.5
	16%	179	110.9	0.6
LDPE+0.5% PP	4%	159.6	150	0.9
	8%	227.5	172	0.8
	12%	211.2	176.6	0.8
	16%	231.3	162.7	0.7
PP	4%	215.5	173.9	0.8
	8%	222	157.9	0.7
	12%	194.3	118.6	0.6
	16%	181.2	98.3	0.5
HDPE	4%	200.3	165.1	0.8
	8%	191	131.3	0.7
	12%	167.4	100.44	0.6
	16%	140.3	84.18	0.6

It is observed that incorporating plastic in bitumen mix makes it less susceptible to moisture than the control mix. Tensile strength ratio (TSR) of plastic modified bitumen mix has increased by 0.9 in this study (Table 4.12). Tiwari, 2018; Sangita 2011; Hadidy and Yi-qui, 2008; Pamungkas et al 2019 also found similar outcome where LDPE and HDPE were used and plastic modified mix had the TSR value increased by 0.9. However, while using only LDPE, the asphalt mixtures do not satisfy the requirement of minimum 0.7 tensile strength ratio (TSR) as per the AASHTO specification. This may happen due to the softness of LDPE material. Since LDPE is softer and thinner than other types of plastic it melted quickly leaving less coating on the surface of the aggregate and less bonding between the mix. As a result, a significant amount of voids may remain in the mix which allows water to percolate and make the sample susceptible to the moisture. To improve the moisture susceptibility, 0.5% by weight of aggregate was replaced by PP. Such combination of plastics may fill the void of the mix and make better bonding between them. Thus, it is observed to improve the moisture resistance of the asphalt mixture and showed TSR values in between 0.7 to 0.9. Similarly, PP and HDPE modified mix has improved the moisture susceptibility while using up to 8% plastic. Plastic usage greater than 8% may not be beneficial according to the observed results.

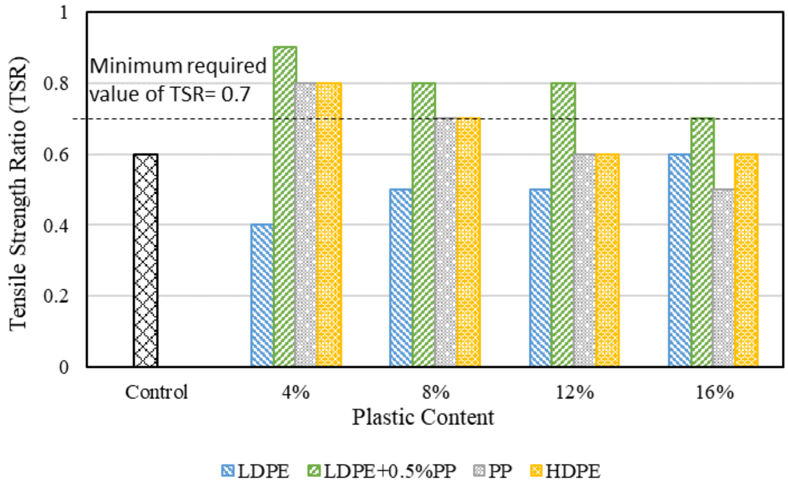

Figure 4.24. Tensile strength ratio of asphalt mix with different combination of plastic for mix design 1

In the mix design 2, control mix having 25% RAP is already susceptible to moisture due to high RAP content. While modifying this mix with different plastics it significantly improves the susceptibility to moisture. However, for higher RAP content plastic incorporation can be beneficial up to 4% usage (Figure 4.25). The results of the TSR value for mix design 2 is tabulated in Table 4.13.

Table 4.13. Determination of Tensile strength ratio (TSR) for mix design 2

Plastic type	Plastic content	Unconditioned IDT (psi)	Conditioned IDT (psi)	TSR
No Plastic	Control (0%)	203.9	101.9	0.5
LDPE+0.5% PP	4%	201.2	140.8	0.7
	8%	181.9	109.1	0.6
PP	4%	217.1	134.4	0.7
	8%	222.4	103.3	0.5
HDPE	4%	189.9	144.3	0.8
	8%	181.3	121.0	0.6

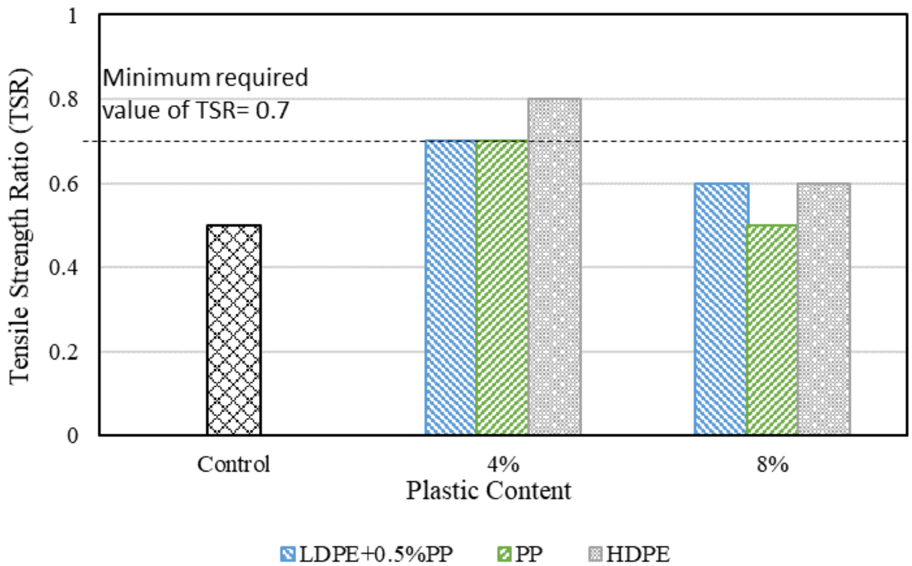

Figure 4.25. Tensile strength ratio of asphalt mix with different combination of plastic for mix design 2

Therefore, it is essential to carefully balance the use of different plastics or their combination to achieve the adequate performance criteria of flexible pavement materia

CHAPTER 5

REGRESSION ANALYSIS

5.1 Introduction

Rutting is one the most important parameters to observe the performance of flexible pavement structures. Hamburg wheel tracker test is advised to conduct to measure the rutting of specific asphalt mix. However, this test equipment and test procedure are expensive, time consuming, and labor intensive. Consequently, it is necessary to determine the value of rutting by using other strength properties, which can be obtained from tests that are easier to administer. In this study, four parameters, Indirect tensile strength (IDT), RAP content (RC), Plastic type (PT) and Plastic Content (PC), were used to develop a statistical model. A multiple linear regression (MLR) model was developed to correlate the Rutting (RT) with Indirect tensile strength (IDT), RAP content (RC), Plastic type (PT) and Plastic Content (PC). Commercially available software RStudio ver 1.4.1103 was used for performing the statistical analyses (RStudio, 2021). The flow of the analysis is presented in Figure 5.1.

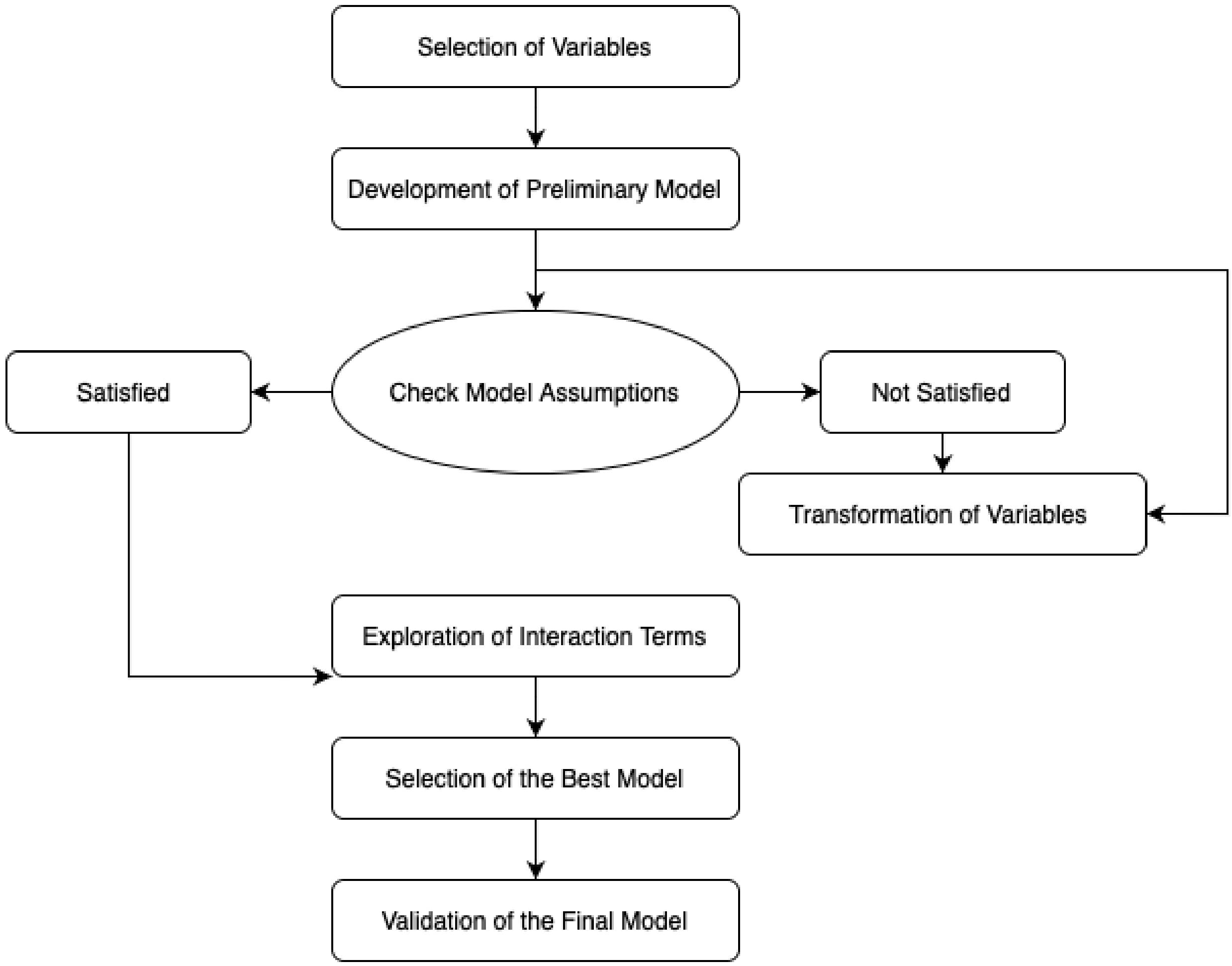

Figure 5.1. Statistical Analysis Flow for the Model Development

5.2 Parameters Selection for Model

The predictors for the model were selected in such a way that they were not highly correlated to each other. If the predictors have a high degree of collinearity among each other, the developed model might not be very reliable. This could lead to smaller coefficient of regression, higher variance, and difficulty in explaining the effect of unit change of predictor on the response (Pituch and Stevens, 2015). However, predictors cannot always be controlled in real life scenarios, and correlation among themselves does exist. The problem of interrelation among predictor variables is designated as multi-collinearity. If a strong correlation exists among predictors, the answer obtained depends highly on the predictors in the model. Change in expected results for unit change in a predictor variable may be inappropriate in this situation. Multi-collinearity may pose three setbacks in a MLR model, such as: a) reducing the coefficient of regression b) difficulty in determining the importance of the variables, and c) increasing the variance (Stevens, 2012).

The objective of this study is to develop a Multiple Linear Regression (MLR) model to correlate Rutting (RT) of plastic modified bitumen mix with IDT, RAP content, Plastic type and Plastic content so that rutting test data can be obtained from an indirect tensile strength test, rather than doing the rutting test, which is more time consuming to perform. In this study, Rutting (RT) was modeled to be the response, while Indirect tensile strength (IDT), RAP content (RAP), Plastic type (PT) and Plastic Content (PC) were the predictors.

Since all the independent predictors affect the response to some extent, it was decided to include all the parameters in the preliminary statistical model. The parameters were denoted as follows:

RT = Rutting Depth (in mm)

RAP = RAP percentage (%)

PC = Plastic Content (%)

PT = Plastic Type (HDPE, LDPE, PP)

IDT = Indirect Tensile Strength (psi)

5.3 Multiple Linear Regression Analysis

This section includes a detailed description of the multiple linear regression analysis. Based on the lab test results, a MLR equation was developed to predict the rutting of the plastic modified bitumen mix as a function of Indirect tensile strength (IDT), RAP content (RAP), Plastic type (PT) and Plastic Content (PC).

5.3.1 Correlation Analysis

Correlation analysis was performed between the response variable and each of the predictor variables to evaluate the relationship between them. It was also performed among to predictor variables to assess any multicollinearity, if present. There should be no multicollinearity among the predictor variables (Kutner et al., 2005). The existence of multicollinearity means that two or more predictors can explain the same variation of the response. If a strong correlation exists among the predictor variables, it will pose setbacks to the MLR model.

Response vs Predictor Plots

The response variable was plotted against each of the predictor variables, as shown in the following figures. The units used for Rutting are in mm, IDT are in psi while the RAP content and plastic contents are in percentage. The relationship between the response and predictors are not following any trend. Figure 5.2 to Figure 5.5 represents all of the response vs predictor plots.

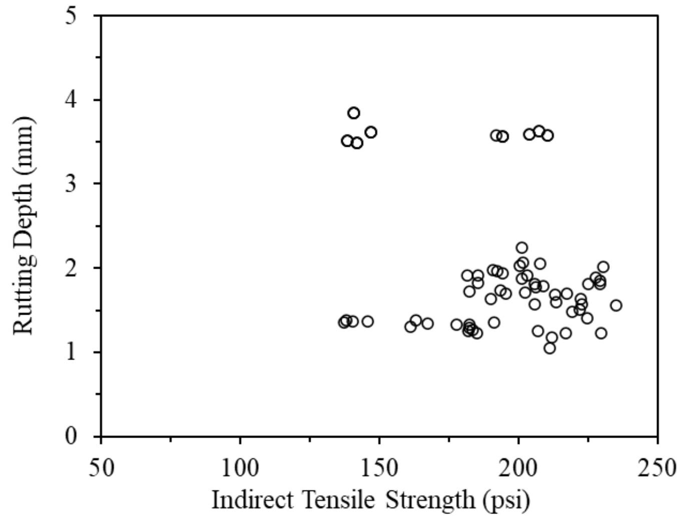

Figure 5.2. The correlation of rutting depth (RT) with indirect tensile strength (IDT)

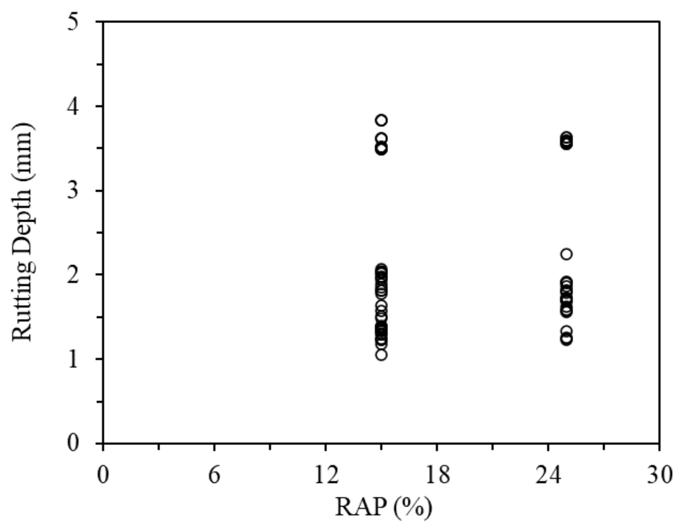

Figure 5.3. The correlation of rutting depth (RT) with RAP content (RAP)

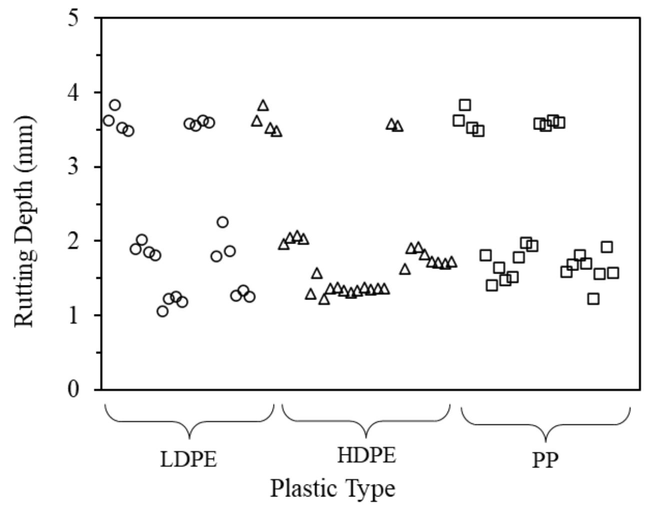

Figure 5.4. The correlation of rutting depth (RT) with plastic type (PC)

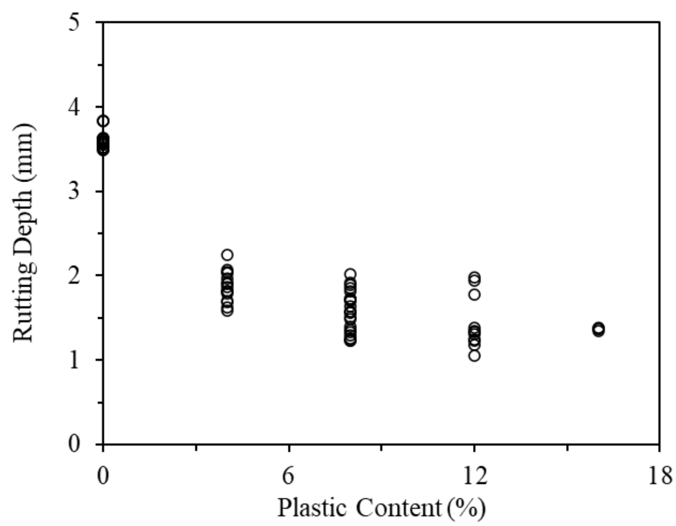

Figure 5.5. The correlation of rutting depth (RT) with plastic content (PC)

Predictor vs Predictor Plots

The predictor vs predictor plots help us determine the multicollinearity between predictor variables. According to the predicting plot (Figure 5.6), no predictor variables have any substantial correlation between each other.

The Pearson Correlation Coefficients between the predictors are shown in Table 5.1. The highest correlation was found to be between the RAP content and plastic content, i.e., -0.367. However, Kutner et al. (2005) states that any correlation less than 0.7 can be regarded as weak. Elastic modulus is also strongly correlated with unconfined compressive strength. The value of r > 0.7 for two of the predictor variables, which suggests that multicollinearity exists within the model. Thus, no significant collinearity was observed among the predictor variables.

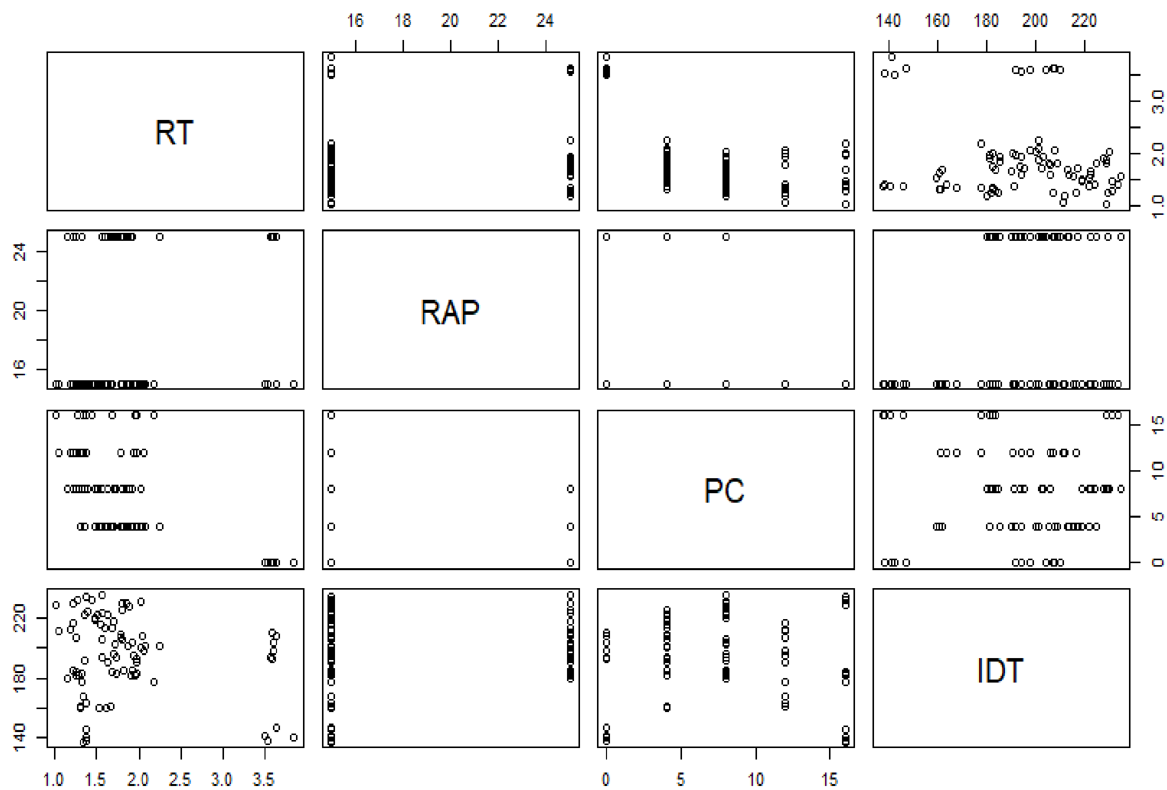

Figure 5.6. The correlation among the predictor variables

Table 5.1. Correlation among the Predictor Variables

Variables	RAP	PC	IDT
RAP	1.00	-0.36	0.27
PC	-0.36	1.00	0.17
IDT	0.27	0.17	1.00

The linear strength between the response and the predictor variables were also measured using the correlation coefficient. Based on the statistical analysis (Table 5.2) RAP content has positive correlation with rutting depth. This means that an increase of RAP content will increase the rutting depth. Likewise, plastic content and IDT have negative correlation coefficients, such that an increase in this factor will reduce the rutting depth as well.

Table 5.2. Correlation between the Rutting depth and Predictor Variables

RT	RAP	PC	IDT
1.00	0.168	-0.740	-0.351

5.3.2 Development of Preliminary Model

A preliminary multiple linear regression model was developed, correlating rutting depth (RT) with Indirect tensile strength (IDT), RAP content (RAP), Plastic type (PT) and Plastic Content (PC). The Preliminary MLR Model was found as follows:

$$RT = \beta_0 + \beta_1 \ RAP + \beta_2 \ PC + \beta_3 \ PT + \beta_4 \ IDT + \varepsilon_i \qquad (5.1)$$

Where, RT= rutting depth (mm), IDT = indirect tensile strength (psi), PT= plastic type, RAP = RAP content (%) and PP = plastic content (%) are regression parameters. β0, β1, β2, β3 and β4 are correlation coefficients which are determined through regression analysis by minimizing the sum of squared errors for the model data. εi is the random error.

The physical meaning of the correlation coefficients is that they explain the variation in mean response per unit change of a predictor variable when all other predictor variables are kept constant. The regression parameters were estimated by minimizing the sum of squared errors for the sample. The predictor variables are quantitative in nature. Multiple linear regression was performed on the model data.

The parameter estimates and summary of the analysis of variance (ANOVA) are presented in Table 5.3 and Table 5.4, respectively. The sign conventions of the correlation coefficients are as expected and follow the results obtained from laboratory test data. RAP content, plastic content and IDT had negative coefficient, i.e., an increase in those coefficient decreased the rutting depth. The ANOVA summary showed that the adjusted R^2 was satisfactory and is acceptable. The p-value of the residuals was also very less. The preliminary fitted MLR equation can thus be presented as follows:

$$RT = 4.417 - 0.0038\ RAP - 0.1206\ PC + 0.0.0687\ LDPE + 0.2608\ PP - 0.0082\ IDT \tag{5.2}$$

The next step is to check if the MLR model assumptions are verified. The model should satisfy the constant error variance, normality of residuals, outliers, and multicollinearity among the predictor variables checks (Stevens, 1996; Kutner et al., 2005, Faysal, 2017).

Table 5.3. Parameter Estimates of the Preliminary Model

	Coefficient	**Std. Error**	**t value**	**Pr(> |t|)**
(Intercept)	4.417434	0.428362	10.312	$< 2.2e-16$
RAP	-0.003801	0.014289	-0.266	0.79084
PC	-0.120640	0.012795	-9.429	$4.4e-15$
Type LDPE	0.068730	0.151650	0.453	0.65149
Type PP	0.260821	0.155658	1.676	0.09729
IDT	-0.008219	0.002451	-3.353	0.00117

Table 5.4. ANOVA Summary of the Preliminary Model

Residual Standard Error	R^2	**Adjusted** R^2	**F-statistic**	**p-value**
0.5805	0.6136	0.5921	28.58	$< 2.2e-16$

5.3.3 Verification of Preliminary Model

Multiple linear regression (MLR) models must satisfy some assumptions. Graphical plots and different statistical tests will be used to verify the following model assumptions:

- There should be a linear relationship between the response and predictor variables.
- The residuals should have constant variance.
- The residuals should be normally distributed.
- The residuals should not be auto correlated.

5.4 MLR Model Form

Residuals vs predictor variables and residuals vs fitted values plots are generally used to identify the applicability of linear regression for a data set. The appropriate situation for the applicability of a linear regression model is when the residuals are located within

a horizontal band centered on a horizontal axis. The points in the residuals vs predictors have to be scattered, and there is no systematic trend of the points. If any curvature is found in the plots, then the linear regression model is not appropriate, and a quadratic term is needed in the model.

Constant Error Variance

Plots showing residuals vs. predictor variables and residuals vs. fitted values help to determine constant error variance or homoscedasticity. The residuals should be randomly scattered without any trend when plotted against predictor variables. Similarly, there should be no specific trend of residuals when plotted against fitted values. This ensures that the constant error variance of an MLR model has been fulfilled. The presence of funnel shape or any curvilinear trend indicates presence of non-constant variance. The regression in such a case might not be valid. This condition can be mitigated by transformation of variables.

From the Figure 5.7 residuals vs fitted values shows scattered plot. However, a clear curvilinear trend (marked by red) can be seen in the plot. This indicates absence of constant error variance and thus, points towards a need for transformation of the response variable. Further analysis was done by conducting the studentized Breusch-Pagan test in RStudio. The p-value from the test was 0.02353, which is greater than $\alpha = 0.01$. So, the null hypothesis was not rejected indicating that the residuals are homoscedastic at $\alpha = 0.01$.

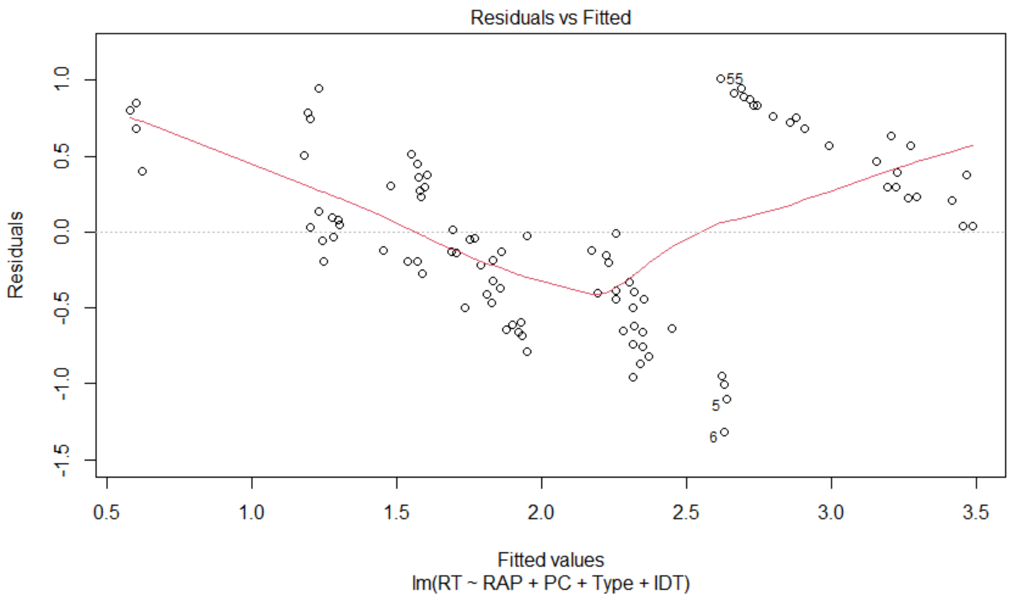

Figure 5.7. Residuals vs. Fitted Values Plot for the Preliminary Model

Normality

The error or the residuals of an MLR should be normally distributed. The normality of the residuals can be determined from a normal probability plot. A moderately linear plot signifies that the residuals are normally distributed. Figure 5.8 shows the normal probability plot for the preliminary MLR model.

A long tail at the right side and a short tail at the left side can be seen from the plot. This indicates that the distribution of the residuals might not be normal. To further verify the normality assumption, Shapiro-Wilk normality test was carried out in RStudio. The test estimated a p-value of 0.07505 which is greater than $\alpha = 0.01$. So, the null hypothesis was failed to be rejected indicating that the residuals are normally distributed at $\alpha = 0.01$.

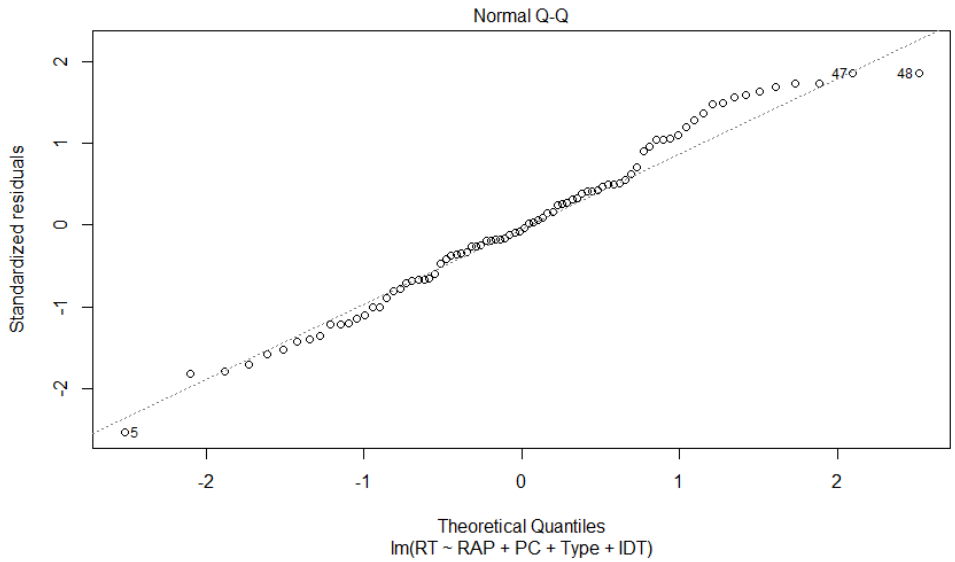

Figure 5.8. Residuals vs. Fitted Values Plot for the Preliminary Model

Outlier Test

Outliers are some extreme observations in a data set. They can mislead the regression by pulling the fitted line disproportionally towards the extreme observation (Kutner et al., 2005). The outliers, if any, were checked using several standard tests in RStudio. Bonferroni outlier test was used to detect outliers. DFFITS (Figure 5.9), DFBETAS (Figure 5.11), and Cook's Distance (Figure 5.10) were used to determine the influence of the outliers in the preliminary model. DFFITS (Difference in fits) estimates the influence of an observation in the predicted value. It is suggested that an absolute DFFITS value greater than 1 (for small to medium data set) for an observation is to be flagged for further check. An absolute DFBETAS value greater than 1 (for medium to large data sets) also suggests flagging the corresponding observation. Similarly, the observation with Cooks Distance (Di) > F (p, n − p) should also be flagged.

Based on the Bonferroni outlier test, one of the observations resulted in a p-value of 0.018721, which is greater than α = 0.01, thus the corresponding observation was identified as an outlier. The observation was flagged as per DFFITS, DFBETAS, and Cooks Distance tests as well.

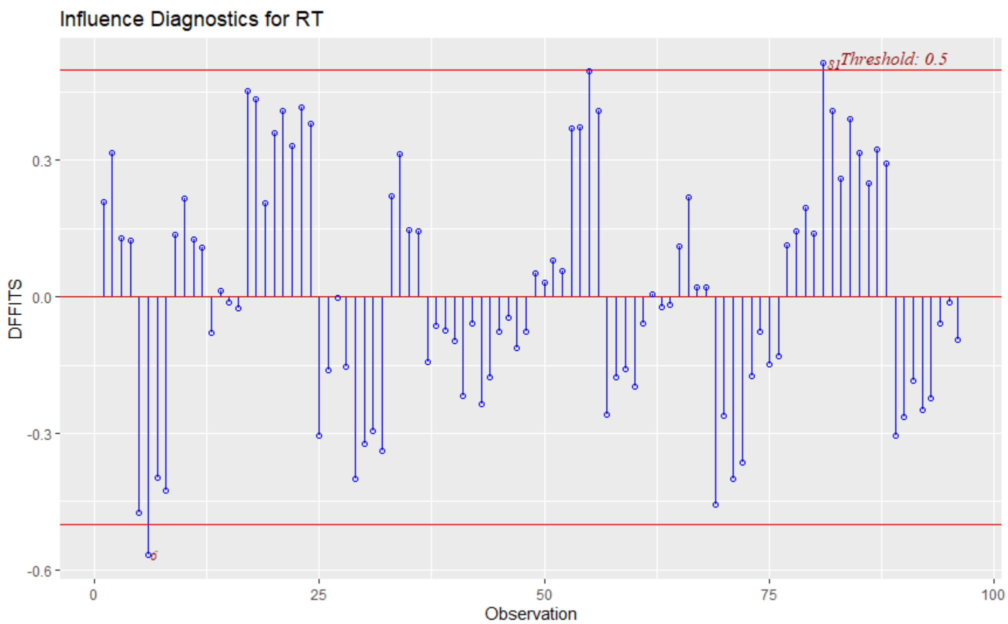

Figure 5.9. Outlier test by using DFFITS

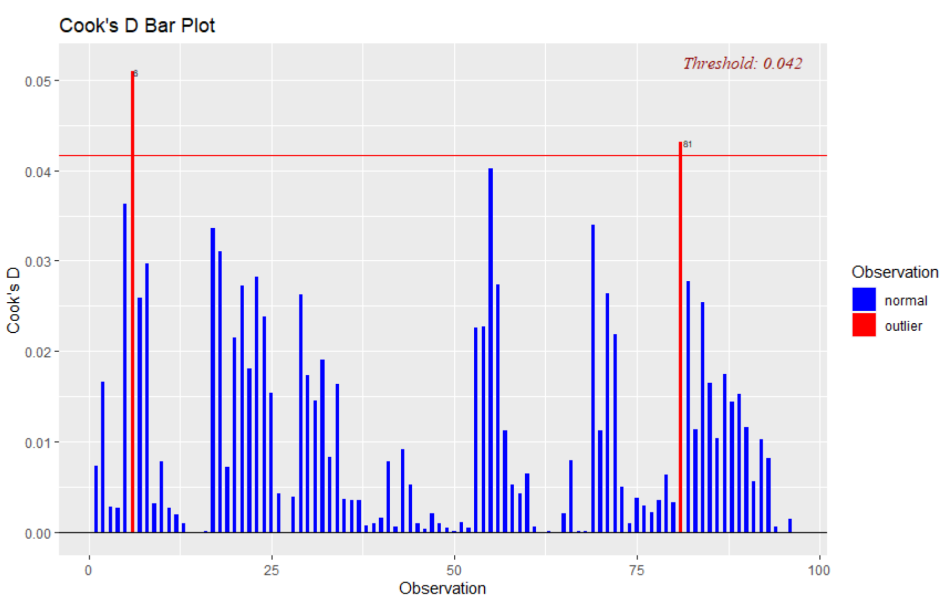

Figure 5.10. Outlier test by using Cook's Distance

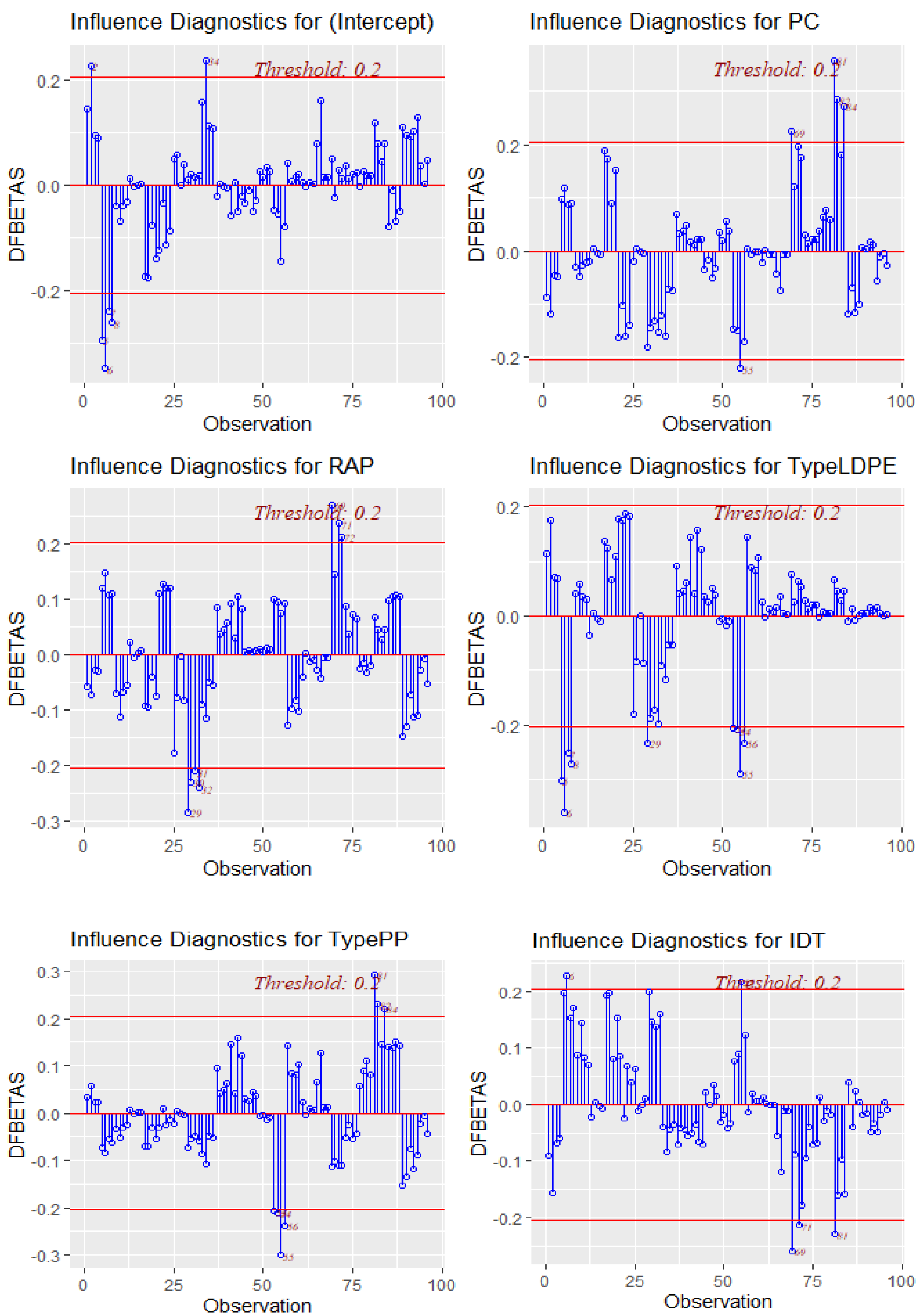

Figure 5.11. Outlier test by using DFBETAS

Multicollinearity

An important assumption of an MLR model is that the predictors should not be highly correlated among each other. Variation Inflation Factor (VIF), which quantifies how much the variation is inflated, can be used to detect multicollinearity in a model. If VIF > 1, multicollinearity occurs among the predictors. However, only predictors with a VIF > 5 maybe problematic. A VIF > 10 suggests high multicollinearity and indicates a poor estimate of the response. Thus, the VIF is preferable to be less than 5. Based on the VIF in Table 5.5, all the VIFs are within the suggested range. Thus, no serious multicollinearity exists among the predictor variables.

Table 5.5. Variation Inflation Factors for the Preliminary Model

Variables	**RAP**	**PC**	**PT**	**IDT**
VIF	1.36	1.29	1.16	1.38

5.4.1 Transformation of Variables and Check for MLR Assumptions

Since the preliminary model satisfied the constant error variance and normality assumptions, no transformation of the response variable was performed. Hence a outlier data was discovered, the preliminary model did not pass the outlier test.

Multiple linear regression was performed again without the outliered data. The parameter estimates and summary of the analysis of variance (ANOVA) for the model presented in Table 5.6 and Table 5.7, respectively. The sign conventions of the correlation coefficients are as expected and follow the results obtained from laboratory test study data. The ANOVA summary showed that the adjusted R2 was satisfactory and is acceptable. The p-value of the residuals was also very less. The final fitted MLR equation can thus be presented as follows:

$$RT = 4.8421 - 0.0114\,RAP - 0.1382\,PC + 0.2616\,LDPE + 0.3018\,PP - 0.0101\,IDT \qquad (5.3)$$

Where, RT = Rutting Depth (mm)

RAP = Rap percentage (%)

PC = Plastic Content (%)

IDT = Indirect Tensile Strength (psi)

For LDPE plastic type, LDPE = 1, PP = 0

For PP plastic type, LDPE = 0, PP = 1

For HDPE plastic type, LDPE = 0, PP = 0

Table 5.6. Parameter Estimates of the Final Model

	Coefficient	**Std. Error**	**t value**	**Pr**$(>\|t\|)$
(Intercept)	4.842054	0.359533	13.131	$2e-16$
RAP	-0.01137	0.012400	-0.917	0.3618
PC	-0.138170	0.010880	-11.507	$2e-16$
Type LDPE	0.261561	0.137826	1.898	0.0614
Type PP	0.301817	0.140392	2.150	0.0347
IDT	0.010094	0.002221	-4.546	$1.97e-0.5$

Table 5.7. ANOVA Summary of the Final Model

Residual Standard Error	R^2	**Adjusted** R^2	**F-statistic**	**p-value**
0.4811	0.743	0.7265	45.11	$< 2.2e-16$

The next step is to check if the MLR model assumptions are verified.

5.4.2 Verification of Final Model

Constant Error Variance

Figure 5.12 shows the residuals vs. fitted values plot for the final MLR model.

No curvilinear trend or funnel shape was detected from the plot. The residuals seem to be randomly scattered. Further analysis was done by conducting the studentized Breusch-Pagan test in RStudio. The p-value from the test was 0.0859, which is greater than α = 0.01. So, the null hypothesis was failed to be rejected indicating that the residuals are homoscedastic at α = 0.01. The constant error variance assumption was fulfilled for the final model.

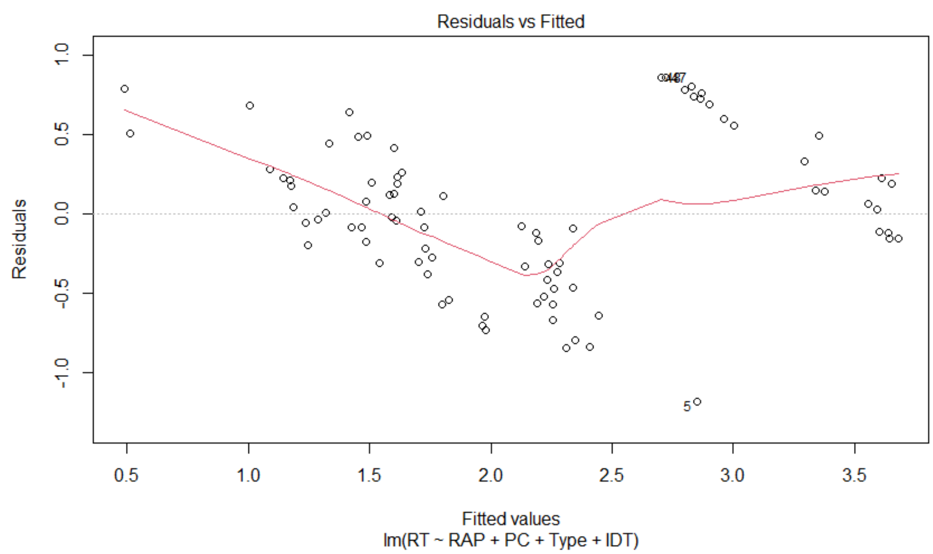

Figure 5.12. Residuals vs. Fitted Values Plot for the Final Model

Normality

Figure 5.13 shows the normal probability plot for the final MLR model.

Short tails on both sides can be seen from the plot. To further verify the normality assumption, Shapiro-Wilk normality test was carried out in RStudio. The test estimated a p-value of 0.312, which is greater than α = 0.01. So, the null hypothesis was failed to be rejected indicating that the residuals are normally distributed at $\alpha = 0.01$. The histogram plot in Figure 5.13 represents normal distribution too.

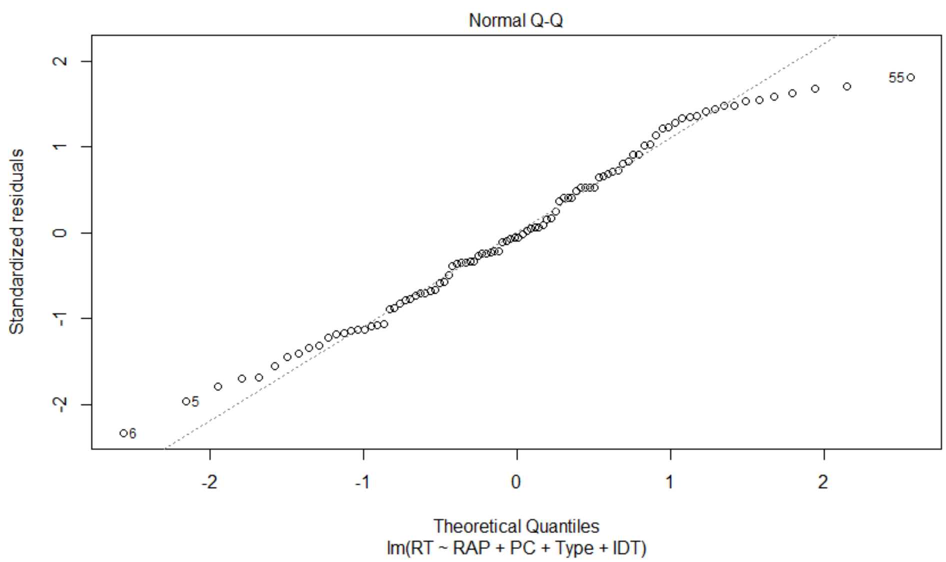

Figure 5.13. Normal Probability Plot for the Final Model

Outlier Test

The outliers, if any, were checked using several standard tests in RStudio. Bonferroni outlier test was used to detect outliers. DFFITS, DFBETAS, and Cook's Distance were used to determine the influence of the outliers in the final model. The F-statistic to compare the Cook's Distance for this set was 1.151 for $\alpha = 0.05$. It is also suggested that Di greater than 0.5 should be investigated, as it may be influential (Faysal, 2017).

Based on the Bonferroni outlier test, none of the observations were flagged as potential outliers. All the observations satisfied the assumptions as per DFFITS, DFBETAS, and Cook's Distance tests as well.

Multicollinearity

All the VIFs, except unconfined compressive strength and elastic modulus, are within the suggested range. The high VIF of unconfined compressive strength and elastic mod- ulus are expected since they are extracted from the same laboratory test, so a relation is inevitable. Thus, no serious multicollinearity exists among the predictor variables.

5.4.3 Final Model Selection

Best subset method, stepwise regression, and backward elimination were performed in RStudio to finalize the best prediction model.

Best Subset Selection

The parameters under consideration for the best subset selection method are R2, adj. R2, Mallows Cp, and Bayesian Information Criteria (BIC). The method selects the best model with the highest R2 and adj. R2, and the lowest Mallows Cp and BIC. The summary of the results is represented in Table 5.8. Based on this method, the combination with three predictor variables without UCS was the best model.

Table 5.8. Summary of Best Subset Selection Method

Predictor Variables				R^2	Adj. R^2	C_p	BIC
RAP	PC	Type	IDT				
✓	-	-	-	0.647	0.642	27.23	-78.5
✓	-	-	✓	0.721	0.714	6.83	-93.8
✓	-	✓	✓	0.727	0.717	6.79	-91.4
✓	✓	✓	✓	0.74	0.727	4.84	-91.1

Backward Elimination

The backward elimination method starts with all the predictor variables in the model. Then, it incrementally removes statistically insignificant variables. The analysis is completed when there is no insignificant variable remaining in the model. Based on this method, all the predictor variables were significant at $\alpha = 0.01$ significance level except unconfined compressive strength. So, UCS was removed from the model.

Stepwise Regression

Stepwise regression method utilizes both the backward selection and forward selec- tion algorithms. The model starts with the most significant predictor variable. The regression is carried out and the parameters under consideration are calculated. Then, other variables are incrementally added as per their significance. The procedure is re- peated until the model with the best criteria parameters is obtained. The F-statistic test is used to conduct the statistical significance tests (Kutner et al., 2005). Based on this method, the three predictor variables except UCS formed the best model.

However, the model with all four predictor variables obtained best R2 value. Cp and BIC values were very close to those of the three variables model. Unconfined compressive strength tests were straight forward and did not require much calculations. On the contrary, determining elastic modulus required the same unconfined compressive test, which is why all predictor variables were kept in the model.

5.4.4 Validation of the Final Prediction Model

The experimental test results were used to evaluate the predictive capacity of the developed multiple linear regression model for rutting depth (RT) value of plastic modified bitumen mix. The rutting values were used for different combinations of RAP materials, plastic type and plastic contents. According to Figure 5.14, the developed model can predict 81% of the variation in resilient modulus at different combinations.

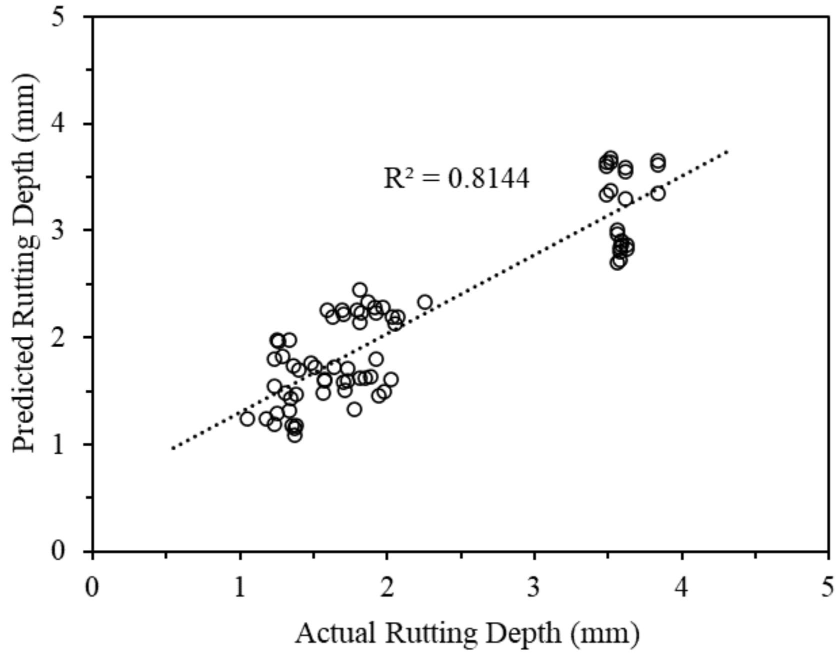

Figure 5.14. Validation of the Final Prediction Model

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Introduction

Disposal of waste plastic is a worldwide problem as the usage of plastics and plastic products has increased since 20th century. But plastic recycling is still a time consuming and costly process. Incorporating waste plastics into the construction materials in terms of improving certain characteristics have been a trend and have shown to be advantageous over the past two decades. However, limited study has been conducted to determine the overall guidelines for use of different recycled waste plastic along with recycles aggregates for flexible pavement. Recycled asphalt pavement (RAP) is widely available recycled materials and might be used as an alternative to natural virgin aggregates for new pavement construction. Therefore, it was necessary to conduct a study on the performance evaluation of plastic modified bitumen mix where different grades of plastics with different proportion were used and the effect of variation of RAP content with the plastic mix also observed. An experimental program was developed to conduct the bulk density test, Rice gravity test, Hamburg rutting depth, indirect tensile strength, overlay test, moisture susceptibility test. The findings from the laboratory test results and analysis of the data are summarized in this chapter. Finally, recommendations for further study have also been provided.

6.2 Summary and Conclusions

Based on the current study the main activities done so far are summarized as follows:

- Three types of recycled plastics which were High density polyethylene (HDPE), polypropylene (PP) and Low density polyethylene (LDPE) plastic were collected from the Republic Services Material Recovery Facility(MRF). The MRF usually collects the waste from curbside trash of nearby cities and conduct a thorough sorting of the collected waste plastics according to the seven categories.
- Collected plastic were cleaned, dried and shredded into 3-6 mm sizes.
- Five different types of aggregate were collected from Austin paving Co., Dallas for the surface course testing. These were Type C rock, Type D rock, Man sand, recycled asphalt pavement (RAP) and recycled asphalt shingles (RAS). PG 64-22 bitumen was also collected from there.
- Grain size analysis for aggregates of surface course was also conducted.
- Two mix designs were used where mix design 1 had 15% RAP content and mix design 2 had 25% RAP content.
- For surface course, four combination of plastic such as LDPE (replaced as bitumen), LDPE (replaced as bitumen) + 0.5% PP (replaced as aggregate), PP (replaced as bitumen), and HDPE (replaced as bitumen), are used along with recycled asphalt pavement (RAP). All the plastics are used in different amounts to replace up to 16% by weight of bitumen.
- Volumetric tests and performance tests (Rutting, Indirect tensile strength (IDT), Overlay, Moisture susceptibility) are conducted to evaluate the usage of plastic in asphalt mix.
- Optimum bitumen content for two mix designs obtained as 4.8% and 5.3%.
- Air void of different plastic combinations was calculated through bulk specific gravity and maximum specific gravity. For mix design 1, all types of plastic mix pass the criteria of 3-

5% air void up to 12% usage except when only LDPE was used. For mix design 2, the plastic mix pass the air void criteria up to 8% usage.

- Rutting depth is decreasing significantly with the increase of plastic content. Moreover, plastic modified asphalt mix can improve the rutting susceptibility by 75%.
- Addition of up to 8% of all types of plastics was found to increase the tensile strength of the mixture up to 77%. After that with the increasing plastic content, this increase in strength was observed to decrease, indicating that a lower dosage (up to 8%) is more beneficial. However, the work energy needed to break the sample is higher in HDPE mix sample compared to other types of plastics. As a result, HDPE mix is tougher than LDPE and PP which leads to sustain more tensile stress. The slope of load vs displacement curve of LDPE and PP is higher indicating a stiffer mix.
- For the lower RAP content (up to 15%) overlay test can pass while using up to 4% any type of plastics. However, since HDPE makes the mix tougher it can withstand till minimum 300 cycles up to 8% usage. This HDPE mix can double the service life of the pavement. While using higher RAP content, plastic mix fails to satisfy the minimum criteria.
- Using Plastic in bitumen mix improves the moisture susceptibility to a great extent. However, only LDPE as bitumen replacement is not meeting the minimum requirement of moisture susceptibility due to its higher void that penetrates the water more easily to the mix. Nonetheless, using PP as aggregate replacement along with LDPE as bitumen might fills the void of the mix which leads to a better moisture susceptibility. The Tensile strength ratio (TSR) of this mix meets the minimum requirement up to 16% plastic replacement. Whereas, for HDPE and PP mix it can improve the moisture susceptibility up to 8%.

- To determine the value of the rutting depth (RT) for different combinations of recycled materials, MLR models were developed using indirect tensile strength (IDT), RAP content (RAP), plastic type and plastic content (PC). As a final model, we have the following:

  $$RT = 4.8421 - 0.0114\,RAP - 0.1382\,PC + 0.2616\,LDPE + 0.3018\,PP - 0.0101\,IDT$$

  This model had a regression coefficient of 81%. Thus, 81% of the variation in rutting depth (mm) is explained by the model in relation to indirect tensile strength, RAP content, Plastic type and plastic content.

- All three types of plastics can be used up to 8% by weight of bitumen in asphalt mix to improve the performance of flexible pavement. This is valid only an usage of RAP content up to 15%. For higher RAP content the plastic usage will be reduced to 4% where this mix can only use for new construction. Rehabilitation work or overlay design won't be beneficial in higher RAP content plastic mix.

6.3 Recommendations for Future Studies

1. Current study was conducted according to dry mixing of plastic. Hence, the effect of adapting wet mixing can be investigated.
2. In the present study, no asphalt binder test was done. It can be investigated to observe the effect of using plastics on the grade of bitumen.
3. LDPE, HDPE and PP were mixed separately for this experimental program. As sorting of different plastics were time consuming, combination of different plastics can be utilized for a future study.

4. Clean plastic was used in this study. Plastic generally found dusted and impurity remained over it. Cleaning procedure was time consuming. A study can be conducted to observe if there is any variation in strength properties if plastic would not clean before use.

5. The compaction temperature of the plastic bitumen mix was assumed based on the compaction temperature of polymer (rubber) bitumen mix specified in TxDOT specification. In the future studies, a required compaction temperature should be determined prior to the compaction of the each types of plastic mix.

6. Current study was conducted for the wearing course layer. Further study can be done for intermediate or crack attenuating (CAM) layer.

7. The effect of microplastic in the environment after using in pavement construction needs to be investigated.

8. A detailed life cycle analysis and cost analysis can be done in future studies to know about the sustainability and cost effectiveness of plastic use in pavement.

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