EFFECTS OF WASTE PLASTIC AS PARTIAL REPLACEMENT OF BITUMEN AND USE OF COFFEE HUSK ASH AS MINERAL FILLER FOR THE PERFORMANCE OF HOT MIX ASPHALT

MSc. THESIS

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This is to certify that the thesis entitled **"Effects of Waste Plastic as Partial Replacement of Bitumen and Use of Coffee Husk Ash as Mineral Filler for The Performance of Hot Mix Asphalt"** submitted in partial fulfilling of the requirements for the degree of Master's with specialization in Road and Transport Engineering, the Graduate Program of the School of Civil Engineering and Built Environment and has been carried out by Nahusenay Muluken Tessera Id. No. PGRoTrR/020/11, under my supervision. Therefore I recommend that the student has fulfilled the requirements and hence hereby can submit the proposal to the department.

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We, the undersigned, members of the Board of Examiners the final Open Defense by Nahusenay Muluken Tessera, have read and evaluated his thesis entitled **"Effects of Waste Plastic as Partial Replacement of Bitumen and Use of Coffee Husk Ash as Mineral Filler for The Performance of Hot Mix Asphalt"** and examined the candidate. This is, therefore, to certify that the proposal has been accepted in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering (Specialization: - Road and Transport Engineering).

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DECLARATION

I hereby declare that this M.Sc. The thesis entitled **"Effects of Waste Plastic as Partial Replacement of Bitumen and Use of Coffee Husk Ash as Mineral Filler for The Performance of Hot Mix Asphalt"** is my original work and has not been presented for a degree in any other university and all sources of material used for this thesis have been appropriately acknowledged.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ACV	Aggregate Crushing Value Test
AIV	Aggregate Impact Value test
ASTM	American Society for Testing and Materials
AST	Amplitude Sweep Test
BBR	Bending Beam Rheometer
BS	British Standard
DSR	Dynamic Shear Rheometer
DTT	Direct Tension test
ERA	Ethiopian Road Authority
FST	Frequency Sweep Test
HDPE	High-density polyethene
HMA	Hot Mix Asphalt
LAA	Los Angeles Abrasion test
LDPE	Low-density polyethene
LVER	Linear Viscoelastic Range
MSCR	Multiple Stress Creep and Recovery
NAPA	National Asphalt Pavement Association
OBC	Optimum Bitumen Content
PAV	Pressure Aging Vessel
PE	Polyethylene
PET	Polyethylene terephthalate
PP	Polypropylene
PVC	Polyvinyl chloride
RTFO	Rolling Thin-film oven test
RV	Rotational Viscometer
SPI	Society of the Plastics Industry
TFV	Ten Percent Fines Value
VIM	Void in the Mixture
VMA	Void in mineral aggregates
VFB	Void fill with bitumen

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ABSTRACT

Bitumen properties and external factors such as climate and traffic volume conditions govern hot mix asphalt pavement performance. Bitumen is viscous-elastic naturally and very sensitive to temperature, which influences its rheological properties. Flexible pavements are linked to extreme high-temperature causing rutting and fatigue cracking. Pavement distress shortens service life and increases maintenance costs. This research focused on improving pavement resistance to distresses by modifying the rheological properties of bitumen and using alternative fillers such as coffee husk ash (CHA). In this study, two phases were utilized. The first phase was collecting samples, and the second contained three sub-phases. To design a material quality test, the first step was to develop a Marshal mix design and three types of mixtures which was a mixture of normal HMA with SD filler, with CHA filler, and a combination of CHA filler and 3, 6, 9, and 12% PET plastic. Rolling thin film oven test (RTFO) and Dynamic shear rheometer are used to analyze the rheological properties of asphalt binder (DSR). Finally, for evaluation purposes, results of 3D-Move analysis software for moving vehicles under various loads and speeds were obtained. AST result shows that at 6% PET containing bitumen has better LVER when compared with original bitumen that changes from 2.49, 23.0, and 64.7% to 7.5, 50.2, and 87.1% at a temperature of 21.1, 37.8, and 54.4^{0}C, respectively. On the other hand, PG determination of the original binder has no variation at 3% PET but improved from PG 58 to PG 64 and PG 70 when the bitumen is modified at 6 and 9% PET plastic. From MSCR criteria, test result at 9% PET modified binder can be used for heavy traffic and approaches to very heavy traffic compare with 0, 3, and 6% PET plastic and a significant improvement for rutting resistance at a test temperature of 70^{0}C. The penetration and ductility value decrease with softening point increase as the PET content increases from the conventional test result. Furthermore, from the marshal test, CHA filler is a better performance of HMA mixture compared with SD filler. At 6.8% optimum PET plastic higher stability (15.9KN) and density value (2.268g/cm^3) and lower flow (2.96mm), VMA (14.83%), and VFA value (73.01%) as compared with unmodified bitumen of 14.30KN, 3.5mm, 2.256g/cm^3, 15.62%, and 75.06% value respectively at 4% air void provided. The analysis 3D-Move software showed that asphalt binders with higher PET plastic content have the best performance as the stiffness of the asphalt mixes would increase with the addition of PET plastic. Therefore, the evaluation showed that partial replacement of PET plastic with the range of 6 to 9% improves the rheological properties by stiff the binder and rutting resisting performance of binders at high-temperature range and increasing the LVER has a better performance of HMA mixture.

Keywords: *Asphalt, Dynamic shear rheometer, 3D-Move Analysis Software, Polymer, Polyethylene terephthalate, Rheological properties, Superpave, and Viscoelastic.*

1. INTRODUCTION

1.1. Background

Road networks play an important role to connect a place to another, hence good road network is a key for the modern lifelines of societies with the integration of the world market, economic growth, and higher levels of income, transport has become a major economic sector in the world and most roads in the world are a flexible type of pavement (Obeta & Njoku, 2016). The Ethiopian economy depends on agricultural products and thus required an efficient road network to transport the product to the market for socio-economic development. A well-developed road network facilitates the transportation and marketing of farm products, while bad roads are delaying the movement of goods and services from producers to consumers and farm products from the rural areas to urban centres. Most roads in Ethiopia are also flexible pavement types (Road sector development program, 2015). Flexible pavement structure the total thickness consists of subgrade, subbase, base-coarse, and surface course. The surface course is the upper layer which is directly in contact with traffic load (Flamarz Al-Arkawazi, 2017). It is made up of asphalt concrete which consists of aggregate, asphalt binder, and filler material which is high quality and expensive materials compared to other materials in other layers. Asphalt binder is the most expensive material in Hot-Mix Asphalt (Babu, 2016).

In the construction of flexible pavements, the main role of asphalt is binding the aggregate together by coating over the aggregate surface and helps to improve the strength of the road (Prasad & Sowmya, 2015). However, it is well known that the structural defect of pavement did not depend only on traffic load but also several temperature weaknesses at low temperature thermal cracking damage, medium temperature fatigue cracking, and high-temperature rutting is the most predominant distress types in the hot mix asphalt pavements (A. F. Ahmad et al., 2017). This distress is reduced the service life of the pavement, develop higher surface roughness, which leads to vehicle damage and reduced comfort of passengers and increases the cost of maintenance. There are different solutions to reduce the pavement distresses the method to improve the quality of asphalt by modification like Polymers, crumb rubber, and fibre modification to improve the performance of asphalt pavements by increasing fatigue life, maximize the resistance to cracking and permanent deformation (Prasad & Sowmya, 2015). The Properties of asphalt can be improved by controlling the refines process are very difficult to achieve. Therefore, the most favoured method to improve the quality of asphalt by modification with polymer is one of the methods to improve the performance of hot mix asphalt pavement (Habib et al., 2010).

Nowadays, many types of research have been conducting on using Polymer in road construction such as Plastic roads because it has two benefits and more available. One is used as a modifier in the hot mix asphalt pavement and the other one is better to protect the environment from pollution (Chandu et al., 2016). Improvement of road performance is necessary because, there are numbers of traffic factor-like heavier loads, higher traffic volume, and higher tire pressure. Utilization of waste plastic such as Polyethylene terephthalate (PET), in asphalt mixture, can improve the properties such as softening point, resistance to rutting, durability, viscoelastic property, and fatigue life (A. F. Ahmad et al., 2017). Manju et al., (2017) indicated that the shredded PET plastic added to hot aggregates, form a fine coat of plastic over the aggregate and the coated aggregates mix with asphalt are found to have higher strength and resistance to rutting and better performance over the life of the pavement.

In addition to the modification of asphalt binder, a mineral filler material used in the production of asphalt mixture is a major role in the mechanical behaviour of hot mix asphalt. It has two major roles one is to fill the void between the aggregate skeleton in the mixture and become denser, raise the stability, and improve the rutting resistance of pavement by increasing aggregate to aggregate contact surface area. Secondly, during mixed preparation filler materials mixed with asphalt binder and form asphalt-mastic. This property is modified to the adhesive properties and overcoating binding nature and increasing the durability, and improve the properties of asphalt binder by reducing the binder's inherent temperature susceptibility (Bi & Jakarni, 2019). Therefore, it is important role to control the mechanical properties of the asphalt mixture.

Therefore, using waste or by-product material for HMA mixture production is a vital role to minimize the cost of the project by reducing the import of bitumen, improve the performance of asphalt pavement, reducing the amount of waste material. The purpose of this research is to use waste or by-product material to improve the performance of road and also create a low-cost, less polluting, and use locally available paving material for road construction.

1.2. Statement of the Problem

Asphalt roads are the most popular in the world because of their low construction costs and good for driving conditions compared to the rigid pavement but asphalt roads are higher maintenance costs than rigid pavement. However, several structural defects occurred on the asphalt road due to the quality of material, climatic condition, traffic load, and other factors. That is why the major problem in the world. Most of the Ethiopian roads are a flexible type of pavement that is compared to the rigid pavement. The increase in road construction cost has common in Ethiopia and many projects are being delayed and limited budget because of the higher cost of asphalt binder. Ethiopia is located in the tropical climatic zone. Due to this reason, the dominant problems that have been seen in hot mix asphalt (HMA) pavements are cracking, ravelling, depression, potholes, and rutting.

Asphalt is the raw material for HMA pavement and it is a visco-elastic material at a moderate temperature that stiffens when the temperature is low and softens when the temperature becomes high. At high temperatures, it fails by rutting because it is soft and at low temperatures, it fails by thermal cracking (Fazaeli et al., 2012). Repair and rehabilitation of cracks, rutting, and other distress in pavements cost millions of birr every year in Ethiopia. Road Sector Development Program (2015) has been reported that the annual expenditure of around 619.2, 459.6, and 234.1 million Ethiopian Birrs for maintenance purposes for federal, regional, and woreda road respectively and also forecast into 2020 about the cost of 998.3, 558.6, and 605.0 million Ethiopian Birr budget for maintenance. Which is too big and required special attention. In addition, damaged pavements also develop higher surface roughness, which leads to vehicle damage and affects the socio-economic activity of the society, which leads to affect the movement of goods and services from producers to consumers and farm products from the rural areas to urban centres. So, to reduce these effects by providing different alternatives to improve such kinds of problems based on the cost and availability of the material in the country. One of the most alternative methods to improve these problems is a modification of the asphalt binder. It is one of the approaches to improve the pavement performance of hot mix asphalt by modification of asphalt material in different ways like using polymer (plastic), fibre, lime, sulfur, Tyre rubber, and other additive materials. In addition to this, replacing conventional filler material with alternative other filler material like coffee husk ash also essential to reduce the above common distresses on asphalt concrete pavement and get better performance. But in Ethiopia, there are no detailed studies with polymer modification and used coffee husk ash as a filler material in the production of hot mix asphalt pavement.

Moreover, different scholars pointed out that using asphalt modifier and filler material is to increase the resistance of the asphalt to permanent deformation at high temperatures, thermal cracking, decreased fatigue damage, and stripping. Therefore, using by-product material like Plastic modifiers (PET) and coffee husk filler is an alternative modification material for the preparation of asphalt mixtures. It is highly abundant locally, cheap, protects environmental pollution by reducing waste plastic and utilize by-product material, easy to collect and handle. And also, it can help to improve the economy of the country as well as creates job opportunities for the poor community in the future as the technology develops and minimizes the cost of rehabilitation and maintenance costs of the road.

1.3. The objective of the study

1.3.1. General Objective

The general objective of the study was to evaluate the Effects of waste Plastic as Partial Replacement of Bitumen and Use of Coffee Husk-Ash as Mineral Filler for the Performance of Hot Mix Asphalt.

1.3.2. Specific Objectives

The specific objectives of the study are:-

- To evaluate the rheological and conventional behaviour of the asphalt binder containing PET plastic.
- To evaluate the volumetric and marshal properties of HMA containing PET plastic and CHA as a filler material.
- To determine the optimum PET plastic content.
- To evaluate the pavement response with and without PET plastic utilizing 3D-Move Analysis Software.

1.4. Research Questions

1. What are the rheological and conventional characteristics of the asphalt binder containing PET plastic?
2. What are the volumetric and marshal properties of the asphalt binder containing PET plastic?

3. What is the optimum PET plastic content to provide a good performance of HMA?
4. How the AC pavement layers respond with and without PET plastic content under moving the vehicle load using 3D-Move Analysis software?

1.5. Significance of the Study

Almost all road contractors to construct in Ethiopia have been using hot mix asphalt. This practice affects the overall construction cost of a project because of the high cost of imported bitumen, quality, it prevents us from seeing more alternatives like using waste plastic and other alternative filler material in hot mix asphalt. This study attempts to overview the alternative road construction technology to improve the performance of hot mix asphalt pavement and indirectly minimize the cost of the project, reducing the amount of waste plastic and by-product material and the area of land used for landfill, to protect against environmental pollution, to create more jobs for the poor community. There are only a limited number of studies on road construction technology using waste plastic and coffee husk ash as filler material in Ethiopia. Therefore, this study is contributing to some extent and give baseline information for users because the community pays attention to their daily waste as it has the value again, investors or contractors who want to invest in this technology it benefits the country, standard and code providers like Ethiopian Road Authority, manufacturers and researchers for further investigation. The government also implements and promotes this technology by helping the investor or contractor to work on this, it also reduces the cost of the road construction project for the country by reducing the high cost of imported bitumen and help it to expand other infrastructure. So, it shows the possibility of the use of waste plastic and coffee husk ash as filler material as the raw material of road construction.

1.6. Scope and Limitation of the Study

Due to the limitation of work of hot mix asphalt pavement using waste plastic and coffee husk ash as filler material in the country, the scope of the study restricted to overview the performance evaluation of hot mix asphalt using waste plastic material as a partial replacement of bitumen and using coffee husk ash as a filler material in the production of hot mix asphalt mixture and unable to compare the experimental result with the actual work in the country. In addition to this currently, the spread of the Covid-19 pandemic in our country has made me unable to do my laboratory work properly and as a result, this study been prevented from completing my work on the due date. Furthermore, these limitations include:

- The research was focused on the rheological characteristics of asphalt using a Rolling thin-film oven and dynamic shear rheometer test.
- The absence of Pressure Aging Vessel (PAV), Bending Beam Rheometer (BBR) and Direct Tension Test (DTT), low-temperature binder characterization was not conducted for performance grade (PG) determination.
- The binding material used for the test was 60/70 penetration grade because the grade is used in the project site above 27ºC.
- One type of waste plastic (PET) used for the modification of asphalt binder which easily available from the environment and cheapest cost from the market.
- Percentages of waste plastic were utilized in asphalt mix within the range of 3 -12% with 3% incremental by OBC weight.
- The gradation of waste plastic was limited between 2.36-4.5mm sieve sizes.
- The evaluation of pavement response utilizing 3D-Move Analysis software is considered only AC Rutting, Top-Down, and Bottom-Up cracking.
- The coffee husk used for this study burned in the furnace at 550 °C for 4 hours to turn it into ash.

2. LITERATURE REVIEW

2.1. Introduction

Road networks are important for modern societies and economic development and it is directly related to mobility and accessibility of communities to quality road networks (Obeta & Njoku, 2016). Ethiopia as a whole has about 110,414 km of roads spread all over the country and they are made up of 14,055 km roads that are flexible pavement road networks of the country (Road sector development program, 2015). Flexible pavements are one of the types of pavement that are composed of subgrade, base course, subbase, and surface course layers. The over layers of materials over the natural subgrade which its major function is to transfer and distribute the vehicle loads to the subgrade layers by grain-to-grain contact through the pavement structure. Flexible pavement failures are caused by the quality and proportion of the material (Asphalt binder, aggregate, and modifier) and it affects the performance of hot mix asphalt pavement structure. The major types of flexible pavement failures are include rutting, cracking, depression, potholes, and ravelling. These failures may result from traffic volume or load associated and environmental influences or non-load associated (Flamarz Al-Arkawazi, 2017).

The performance of hot mix asphalt pavement can be improved by the utilization of various types of modifier material, these include polymers, fibres, and other chemical additives (A. R. Prasad & Sowmya, 2015). The addition of polymers improved the durability, rutting, thermal cracking, and also to protect the environment from pollution (Chukka & Carr, 2016).

2.2. Rheological Properties of Asphalt Binder

Rheology of asphalt is defined as the study of flow and deformation of asphalt under the applied load. It is a controlling tool for describing and measuring material properties and their effect on pavement performance. The response of the asphalt binder under the influence of applied load is dependent on the loading time and temperature. Therefore, the rheology of bitumen is defined by the stress-strain-time-temperature response (Hafeez et al., 2013). In addition, the rheological properties of asphalt are changing during the production of hot mix asphalt and the service life of the pavement that expose to air, water, and other factors. Consequently, short and long term ageing has happened. So, it is essential to study the phenomenon of ageing of asphalt binder. The primary hardening effect of asphalt binder started from the batching plant, transported to the paving site, placed and compacted and the rheological properties also change such as an increase in viscosity and decrease the penetration of asphalt binder during the short mixing period by the oxidation process and loss of volatile components from the binding material. The hot mix asphalt pavement has cooled and opened to traffic the long term ageing of asphalt becomes happened. Due to this reason, the rheological properties of asphalt binders are changed within a long term slower ageing rate. This affects the performance of hot mix asphalt pavement (Joshi et al., 2013).

The properties of the asphalt binder exhibit both elastic and viscous properties and can be classified as visco-elastic materials. Elastic materials show recoverable deformation properties subjected to a constant load and immediately deform and back to the original position when the applied load is removed. However, the viscoelastic material subjected to load shows non-recoverable deformation properties that immediate deformation followed by continued time-dependent at a constant rate until the applied load is removed. Thus, a viscoelastic material experiences simply a partial recovery of the deformation resulting from the applied loading (Riccardi et al., 2016). The viscoelastic behaviour of asphalt can be characterized by its deformation resistance and relative distribution of that resistance between the elastic component and the viscous component within the linear range. The relative distribution of the resistance between the elastic component and the viscous component is dependent on the asphalt cement characteristics, temperature, and loading rate, which is described by the deformation being directly proportional to the applied load at any time and temperature. Nonlinear loading responses are difficult to model for viscoelastic materials such as asphalt. Linear response models are tolerable for the engineering analysis of asphalt binder response to the loading conditions and environmental stresses encountered in the field (Zhao et al., 2013).

2.3. Rheological Properties of Asphalt Binder Related to Pavement Performance

The rheological properties of asphalt binder such as stiffness, viscosity, temperature susceptibility, ductility, penetration, and age hardening. Which has a significant effect on the performance of asphalt pavement predominantly, these pavement distresses in asphalt pavement is high-temperature permanent deformation, low-temperature thermal cracking, load-associated fatigue cracking, and stripping. This distress is highly associated with the rheological properties of asphalt binders and affects the performance of asphalt pavement.

2.3.1. High-temperature permanent deformation

Rutting is the most common type of distress related to high-temperature deformation. It is a surface depression that happens in the pavement layers or the sub-grade. It is caused by the movement of material under repeated traffic load. This can occur either through consolidation or plastic flow. Consolidation is occurring due to further compaction of hot mix asphalt pavement by traffic after construction. It occurs because of insufficient compaction of the pavement. Several factors can contribute to lack of compaction these are too few rollers passes, asphalt material cooling earlier to achieving target density, and high asphalt content. However, the major consideration of Rutting also results from the lateral plastic flow of the hot mix asphalt under the wheel track. It is a significant factor in rutting because asphalt cement has viscoelastic properties, at a higher temperature the asphalt binder becomes less viscous or more soften and this produced lower stiffer pavement that can be susceptible to lateral movement subjected to traffic load and this behaviour shows a non-recoverable deformation strain resulting from the applied wheel loads to the pavement (Abdulmajeed & Muniandy, 2017). Plastic Flow can also result from high pavement temperature and asphalt production. To increase the resistance to rutting can be obtained by stiffer asphalt cement. However, stiffer asphalt cement is more prone to cracking at low temperatures. Therefore, to find other solutions for this kind of problem by modification of asphalt binder using polymer materials to improve the viscoelastic properties of asphalt binder and reduced temperature susceptibility to provide more elastic. In addition, using mineral filler material used in the asphalt mixture is essential to increase the viscosity of asphalt cement and thus make the mixture is more resistant to rutting (Shaffie et al., 2015).

2.3.2. Low-temperature thermal crack

Low temperature cracking is the major type of distress in non-load associated cracking. It is a transverse shrinkage cracking in the asphalt pavement layer and the crack starts at the surface and propagates down with time because first the road surface is exposed to low ambient temperature. Then the asphalt layer becomes highly contrast and develops tensile stress. This stress exceeds the tensile strength of the hot mix asphalt pavement layer. Due to this stress transverse cracking occurs. Asphalt binder at a lower temperature has high stiffness modules and very prone to cracking. From this, the stiffness of hot mix asphalt pavement depends on the stiffness of the asphalt binder. Which is mainly influenced by the asphalt binder properties. Consequently, higher binder stiffness at low temperatures is the major cause of thermal cracking or non-load associated cracking. Therefore, the use of modifiers can improve the low-temperature elasticity and strength of the asphalt mixture (Zhang et al., 2020).

2.3.3. Load-associated fatigue cracking

Load associated cracking has been described as fatigue cracking, the phenomenon of fracture under repeated stress having a maximum value less than the tensile strength of the material due to the continuous application of loads over a long period. Easily Asphalt binder becomes hard through ageing to demonstrate poor fatigue life and develop fatigue cracking in hot mix asphalt pavement. However, the property of asphalt binder is the major factor that the cause of the crack is related to the rheological properties of asphalt. To be concluded, load associated fatigue cracking is influenced by the asphalt binder properties and the mixture properties (Rahbar-Rastegar et al., 2018).

2.3.4. Stripping

Stripping is the loss of bond between the aggregates and asphalt cement in the presence of moisture that produced excess pore pressure due to traffic and thermal expansion. Which typically begins at the bottom of the HMA layer and the moisture is high then progresses upward. The strength of the mixture is derived from the cohesion resistance of binder and grain interlock and frictional resistance of the aggregate (Merusi et al., 2010). The stripping of asphalt cement from an aggregate surface occurs with two mechanisms. These are loss of adhesion and cohesion. The first mechanism, the adhesive bond between asphalt binder and aggregate are deteriorated due to the action pore pressure and coated asphalt binder physically separated from the aggregate, its surface is easily wetted by water than asphalt because the aggregate has higher affinity properties for water than asphalt binder and displaced from the aggregate surface. Therefore, the adhesive strength of a material depends on affinity and attraction between aggregate and asphalt binder. Second, the asphalt binder interacts with water and decrease the cohesion properties of the asphalt binder by spontaneous emulsification. This leads to assist reduction in the asphalt mixture strength (Sarsam & Al-Azawi, 2013). Moreover, stripping is depending on aggregate and asphalt characteristics, environment, traffic, and use of anti-strip additives. Asphalt-aggregate interaction plays a vital role in the stripping phenomenon. However, the physical properties of the asphalt binder also a major factor in the phenomenon. The relation between the properties of the asphalt binder in a paving mixture and the tendency of the mix to strip relates stripping resistance to the viscosity of the asphalt binder in service is essential because high viscosity of asphalt binder has a better resistance to displace by water than those of low viscosity. Low viscosity fluid has more wetting power than one of higher viscosity. Additionally, high viscosity asphalt cannot be used in several cases because of other situations like low-temperature cracking in a cold area and a potential reduction in fatigue life of the surface course. So that the problem can be minimized by other means like improving the viscoelastic properties of asphalt binder that is better to use a modification of asphalt binder using polymer (Kakar et al., 2015).

2.4. Performance measuring method for asphalt binder

In the construction of road projects especially flexible types of pavement are used asphalt binders are a raw material because it is a major role in the production of hot mix asphalt pavement. The main importance is to bind the aggregate together by coating over the aggregate surface and helps to improve the performance of asphalt pavement as well as reduce pavement distresses such as rutting, cracking, and moisture damage. So, before construction of the pavement first to evaluate the quality of asphalt binder by taking different types of asphalt binder tests are essential because to determine what materials provide acceptable performance and serve for a given service life under traffic with minimum maintenance and also to determine what materials provide acceptable performance against those modes of failure. Finally, using manuals by following the test procedures and cross-checking the result that satisfies the provided specification. The evaluation of the quality of asphalt binder follows two methods either conventional performance measuring techniques or the current advanced performance measuring method called the Superpave system. For this research, the two Asphalt performance measuring methods was conducted. The Superpave system and conventional performance measuring techniques.

2.4.1. Superior Performing Asphalt Pavements measuring techniques

The term "Superpave" is a shortening term for Superior Performing Asphalt Pavements. The Superpave asphalt binder test was firstly established by the Strategic Highway Research Program (SHRP) in 1987 and it continues to improve. This method was primarily established to improve the previous limitation of asphalt binder physical tests (Zumrawi & Edrees, 2016). Some of these are:-

- The selecting asphalt binder grade is based on the climatic conditions.
- The physical properties measured directly related to field performance.
- The entire range of pavement temperature at the project site is considered.

- The specification required the asphalt binder to test after simulating its three stages: the first, represented by the original asphalt binder. The second, represented by short term ageing, and the last, represented by long term ageing.
- The test and specification are designed to control three specific types of distresses such as rutting (high temperature), Fatigue cracking (intermediate temperature), and thermal cracking (low temperature)

Different tests were performed to characterize the properties of asphalt binders using Superpave physical tests. These tests are - Rolling Thin Film Oven test (RTFO), Pressure Aging Vessel (PAV), Rotational Viscometer (RV), Dynamic Shear Rheometer (DSR), Bending Beam Rheometer (BBR), and Direct Tension Tester (DTT).

2.4.1.1. Rolling Thin-film oven test (RTFO)

The RTFO simulates the asphalt binder ageing (hardening) during the production and construction of HMA pavement. It exposes the fresh binder to heat and airflow during rolling. It takes only 75min to perform the test, then provides an aged asphalt binder for further testing by the dynamic shear rheometer and allows the determination of the mass of volatile lost from the binder during the test. The amount of volatile loss indicates the amount of ageing that may occur during HMA production and construction (Yan et al., 2017).

2.4.1.2. Pressure Aging Vessel test (PAV)

The PAV simulates the asphalt binder ageing that happens during the service life of HMA pavement. It is a cylinder of clean, dry compressed air with a pressure regulator, release valve, and a slow-release bleed valve that is used to supply and regulate air pressure. The PAV consists of stainless steel and able to operate under the temperature and pressure situations of the test (Mohammadafzali et al., 2017).

2.4.1.3. Rotational Viscometer (RV)

RV has been adopted in Superpave for determining the viscosity of asphalt binder at a high temperature above 100^0C to ensure that the binder is sufficiently fluid for pumping and mixing. Therefore, a viscosity measurement is sufficient to represent the workability of the binder. RV is more suited for testing many modified asphalt binder compared to capillary viscometer because the latter can get clogged up partially inhibiting smooth flow. It is determined by measuring the torque required to maintain a constant rotational speed (20 rpm) of a cylindrical spindle while submerged in an asphalt binder at a constant temperature (135^0C). The measured value of torque is directly related to the viscosity of the asphalt binder sample (M. Abdullah et al., 2011).

2.4.1.4. Dynamic shear Rheometer (DSR)

The DSR is used to characterize the elastic solid and viscous liquid behaviour of asphalt binder at high and intermediate service temperatures. It is also known as an oscillatory shear rheometer and measures the complex shear modulus G* and phase angle δ asphalt binder at the desired temperature and frequency of loading. Complex modulus can be considered as the total resistance of the binder to deformation when repeatedly sheared. It consists of two-component these are storage modulus or elastic (recoverable) part, G' and loss modulus or viscous (non-recoverable), G". It is the ratio of maximum shear stress (τ_{max}) to maximum shear strain (γ_{max}) (Cho & Im, 2014), and also phase angle is the time lag between the applied stress and the resulting strain. For a perfectly elastic material is an instant response and the time lag between the applied stress and the resulting strain or phase angle is zero. However, viscous liquid materials like asphalt binder are tested, the time lag is large and the phase angle approaches 90 degrees. Some asphalt binders have the same complex modulus but with different phase angles. So, the complex modulus is not sufficient to characterize the asphalt binder, the phase angle is also needed. Both the temperature and frequency of loading significantly affect the value of G* and δ for asphalt binders (Nam et al., 2010).

Two types of oscillatory shear rheometers are available. These are constant stress and constant strain. In constant stress mode, the specified magnitude of shear stress is applied to the bitumen by applying torque to the spindle and the resultant spindle rotation is measured, a form in which the magnitude of shear strain is calculated. In the control strain mode, the magnitude of spindle rotation is specified and the required torque to achieve this is measured, from which the magnitude of shear stress is calculated. Then it evaluates the specimen's response and calculates parameters of the asphalt, such as complex shear modulus, phase angle, and accumulated strain or stress. These parameters help to measure viscoelastic properties, fatigue crack, and rutting resistance at intermediate and high temperatures in the hot mix asphalt pavements.

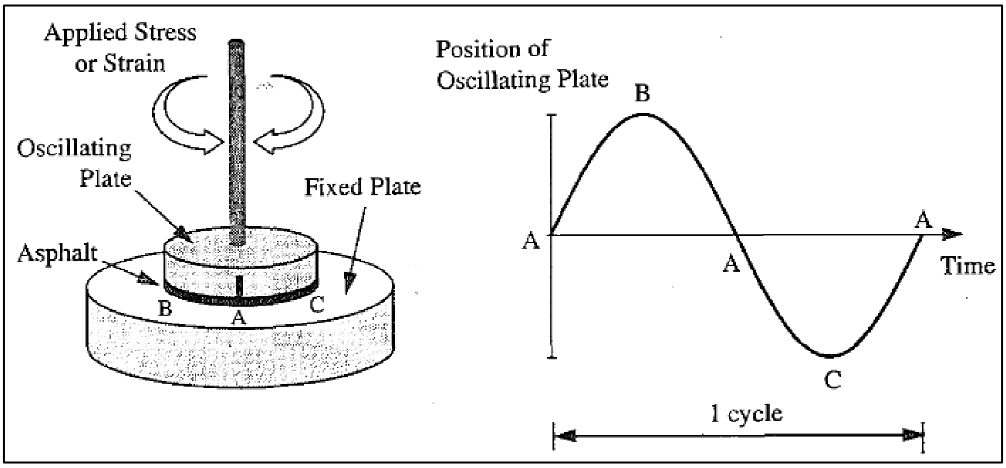


Figure 2.1 Basic of Dynamic shear rheometer equipment (Cho & Im, 2014)

For rutting resistance, the higher complex modulus value and low phase angle are both desirable because the higher the G* value, the stiffer the asphalt binder and thus more resistance to rutting and the lower δ value, the more elastic the asphalt binder. So, it is more resistant to permanent deformation. To minimize permanent deformation (rutting), the amount of work dissipated during each loading cycle must be minimized (Arabani & Shabani, 2019).

$$W_c = \pi * \sigma_0^2 * \left[\frac{1}{G^*/_{\text{Sin}\delta}}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2.1)}$$

Where δ_0 = stress applied during the load cycle and

W_c = work dissipated per load cycle

This equation indicates that W_c is inversely proportional to G*/Sinδ, which is W_c decrease by increasing the value of G* and decreasing the value of δ. Therefore, G*/Sinδ is a parameter for rutting resistance.

For fatigue cracking resistance, lower complex modulus value, and low phase angle are both desirable because the lower the G* value, the lower stiffer the asphalt binder and the lower the δ value, the more elastic the asphalt binder, it's more resistant to fatigue cracking. To minimize permanent deformation (rutting), the amount of work dissipated during each loading cycle must be minimized.

$$W_c = \pi * \epsilon_0^2 * [G^* * \sin\delta] \text{ ………………. (2.2)}$$

Where ϵ_0 = strain by applied during the load cycle and

W_c = work dissipated per load cycle

This equation indicates that W_c is directly proportional to G*Sinδ, which is W_c decrease by decrease both value of G* and the value of δ. Therefore, G*Sinδ is a parameter for fatigue crack resistance.

Accordingly, three main types of tests are conducted using DSR. These are:-

A. Amplitude Sweep test
B. Frequency Sweep Test
C. Multiple Stress Creep and Recovery

A. Amplitude Sweep Test (AST):- The amplitude of the deformation or the amplitude of the shear stress is varied while the frequency is kept constant until the structure breaks down. Amplitude sweeps are mainly used to determine the linear-viscoelastic range of material. The linear viscoelastic part is the region where the applied oscillation is nondestructive. The complex shear modulus G* (y-axis) versus strain (x-axis) plot use to determine the linear viscoelastic region. The amplitude sweep test carries out following the test standard AASHTO T 315 at a constant frequency of 10rad/sec or 1.59Hz at specific test temperatures 21.1^{0}C, 37.8^{0}C, and 54.4^{0}C (Saboo & Kumar, 2016).

B. Frequency Sweep Test (FST):- The frequency is varied while the amplitude of the deformation or the amplitude of the shear stress is kept constant. The tests are used to construct master curves of binder complex shear modulus (G*) and phase angle (δ) (Saboo & Kumar, 2016). The master curves characterize binder rheological properties over a wide range of temperature or frequency. It can be used to estimate binder G* and δ values at any interested temperature and frequency. It is conducted in a strain-controlled mode varying from 0.1Hz to 25 Hz (Hafeez et al., 2013).

- **Master Curves**

Master curves are constructed using the principle of time-temperature superposition because of the relationship between temperatures and frequencies (times of loading). From the data collected over a range of temperatures and frequencies, several rheological graphs are prepared (Hafeez et al., 2013). To represent those graphs with one master curve a standard reference temperature must be selected. Then, the data at all other temperatures are shifted relative to this reference temperature and at a reduced frequency until a smooth curve is generated. The master curves of the complex modulus, storage modulus, loss modulus and phase angle with the change in frequency can be constructed. Master curves allow the rheological data to be presented over a wide range of frequencies and temperatures in one plot. Therefore, to avoid presenting a large number of graphs, the results are mainly presented and analyzed as master curves (Ghuzlan, 2015).

C. **Multiple Stress Creep and Recovery (MSCR):-** The test to evaluate the binder's potential for permanent deformation. AASHTO T350 stated that a1sec. Creep load is applied to the asphalt binder sample and after 1sec. the load is removed, the sample is allowed to recover for 9 seconds. Normally the test is started with the application of low-stress 100Pa for 10 creep/recovery cycles then the stress is increased to 3200Pa and repeated for an additional 10 cycles. In the MSCR test, higher levels of stress and strain are applied to the binder, better representing what occurs in an actual pavement (Hossain et al., 2016).

2.4.1.5. Bending Beam Rheometer (BBR)

BBR is testing the asphalt binder at low service temperature and to measure the binder tendency to thermal cracking. Thermal cracking of HMA pavement caused by the temperature at cold, then the pavement become contracts and develop tensile stresses. This stress exceeds the tensile strength of the pavement, the pavement develops cracking. The BBR uses creep load applied by pneumatic pressure at the centre of the asphalt beam specimen held on a container at a constant low temperature. The temperature is controlled by a fluid bath filled with a mixture of methanol, ethylene glycol, and water. Although creep stiffness is desirable at the minimum HMA pavement design temperature after two hours of loading. Based on different past research determined that by raising the test temperature by 10^0C, an equal creep stiffness is obtained after a 60-sec loading time which is more accurate for testing (Aflaki & Hajikarimi, 2012).

The data acquisition system records the load and deflection test results and calculates two parameters (Sun et al., 2017). These are:

- **Creep stiffness, S(t)** which is a measure of how the asphalt binder resists the creep load. According to Superpave binder specification the maximum value of S(60 sec) = 300 MPa

To calculate the creep stiffness $S\ (t) = \frac{PL^3}{4bh^3\delta(t)}$ …………………….. (2.3)

Where, S(t) = creep stiffness at 60 sec., P = applied creep load, b = beam width

L = distance between beam support, h = beam thickness, δ (t) = deflection at 60 sec.

- **M-value** is a measure of the rate at which the creep stiffness changes with loading time. And also the slope of the creep stiffness versus time at any time, t. the specified value of a minimum m-value to be m ≥ 0.30 at 60 sec.

2.4.1.6. Direct Tension Tester (DTT)

DTT is a properly good relationship between the stiffness of asphalt binder and the amount of stretching they undergo before breaking at low temperatures. Typically, softer asphalt binders are more ductile, and stiffer asphalt binders are more brittle. The creep stiffness of asphalt binders are measured by the BBR is not adequate to completely characterize the low-temperature behaviour of the asphalt binders in terms of thermal cracking. Some modified asphalt binder may have higher creep stiffness S (t) ≥ 300 MPa but does not crack because they have they can stretch further before cracking. Therefore, DDT is only used for testing asphalt binders that have a BBR creep stiffness between 300 and 600 MPa at low pavement service temperature. However, the creep stiffness is less than 300MPa, the DDT not be used (Marasteanu et al., 2017).

2.4.2. Conventional performance measuring techniques

Different tests were performed according to established Standards to characterize the properties of asphalt binders. Some of the Major tests are Consistency tests and Durability tests.

2.4.2.1. Consistency tests

Consistency defines the degree of fluidity of asphalt binder at any certain temperature. Subsequently, asphalt binder is a thermoplastic material and its consistency varies with temperature. Therefore, it is necessary to measure the consistency of different asphalt binders at the same temperatures. *Table 2.1* includes the following tests performed on the asphalt binder.

Table 2.1 The consistency test and its functions

No.	Consistency Test	Functions
1	Penetration	• It is an empirical test used to measure the consistency of asphalt binder and usually measured at 25^{0}C which also the approximately average temperature of the HMA pavement.
2	Softening Point	• It measures at which temperature an asphalt binder cannot be supported by the weight of a steel ball and start flowing. • Its purpose is to determine the temperature at which a phase change occurs in the asphalt binder. • Using by Ring and Ball method.
3	Ductility test	• It measures the distance in centimeter that a standard briquette of asphalt binder stretches before breaking. • Usually measured at 25^{0}C

2.4.2.2. Durability Tests

Asphalt binders undergo substantial short-term ageing (hardening) when they are mixed with hot aggregate in the HMA mixing facility. Their long term ageing continues during the life of HMA pavements which are subjected to environmental and other factors. A ductility test is also used to evaluate the hardening of the asphalt binder by showing the brittles of a material but not showing the short and long term ageing of the asphalt binder.

2.5. Hot Mix Asphalt

Hot-Mix Asphalt (HMA) is the most widely used paving material around the world and it is a heterogeneous material that consists of asphalt cement, aggregate, mineral filler, additives, and air voids. Flexible pavement is known by many different names: Hot-mix Asphalt, asphaltic concrete, plant mix, bituminous mix, bituminous concrete, and others. It is a combination of two primary ingredients aggregates and asphalt binder. Aggregates include both coarse and fine materials, typically a combination of different size rock and sand, and the aggregates approximately 95% of the total mixture by weight are mixed with approximately 5% asphalt binder to produce hot mix asphalt (Nega et al., 2013). The performance of a hot mix asphalt mixture depends on external and internal conditions; the external conditions being traffic load and environment and the internal conditions being the quality and proportion of material (Aodah et al., 2012). These conditions fail the pavement structure. Therefore, modification of HMA pavements is an essential objective as it increases its performance and service life and decreases the maintenance cost of the road (Mosa, 2017)

2.6. Components of Hot Mix Asphalt

Hot Mix Asphalt mixture consists of aggregate and asphalt binder. Aggregates play an important role in determining the nature and characteristics of pavement in preparing the structure of mutual-lock while the binder acts as a glue between aggregate particles with the layer below the surface of the road (Nega et al., 2013).

2.6.1. Aggregate

Aggregates are one of the raw materials used in the construction industry and it covers the largest portion of an asphalt pavement. Aggregates in HMA can be divided into three types according to their size: Coarse aggregates that are retained on the 2.36mm sieve, Fine aggregates that are pass-through 2.36mm and retained on the 0.075mm sieve and Mineral filler are a portion of the aggregate passing the 0.075-mm sieve, which is added to the hot mix asphalt to improve the density and strength of the mixture (Abo-Qudais & Al-Shweily, 2007). The fundamental measurements of aggregate morphological properties (angularity, shape, surface texture, and toughness) are essential for good quality control on the behaviour of hot mix asphalt mixture. Aggregates are characterized by graduation test, flakiness index, durability, absorption, soundness test. The properties of aggregates are very important to the performance of hot mix asphalt pavements. Hence most of the performance problems are directly or indirectly related to the physical and chemical properties of aggregates (Golalipour et al., 2012). The problems such as rutting, stripping, surface disintegration, and lack of adequate surface frictional resistance can be attributed directly to improper aggregate selection and use. The aggregate type has a significant effect on the fatigue resistance and permanent deformation of asphalt mixtures. Thus, to obtain a mixture having good performance, evaluation of various mineral aggregate physical properties is essential (Ahmed & Attia, 2013).

2.6.2. Coffee Husk Ash as Filler Material

Ethiopia has produced coffee and is listed in the top five coffee-producing countries in the world (International Coffee Organization, 2016). The country produces a massive quantity of coffee however, the utilization of coffee husk produced during coffee production is poor and its husk requires a large area for storage and it is environmentally unfriendly. Proper usage of this waste material can reduce storage areas and environmental concerns arising from the disposal. The coffee husk used for this study by collected from the coffee industry and then burned in the furnace at 550 °C for 4 hours to turn it into ash and take the remaining ash used as coffee husk-ash mineral filler.

The basic use of filler material in the preparation of asphalt mixture is to fill the void between the aggregate skeleton in the mixture and during mixed preparation filler materials mixed with asphalt binder and form asphalt-mastic and its effect on the mastic behaviour asphalt binder. This property is essential to increase stability, rutting resistance, denser, durability, and increasing aggregate to the aggregate contact surface area and improve the resistance properties of asphalt binder against the action of traffic load and temperature to improving the properties of asphalt binder by reducing the binder's inherent temperature susceptibility (Miró et al., 2016). Sarir et al., (2019) stated that the marshal stability of bagasse ash filler increased by 20% and reduced the flow by half as compared to the conventional stone dust filler. Modupe et al., (2019) used Cow bone ash to replaced quarry dust filler with 50% cow bone ash material in the hot mix asphalt mixture and the result showed that increase stability and reduce flow and improve the volumetric properties of filler compared with quarry dust. This research was carried out to determine the suitability of coffee husk-ash as a filler material and its proportion in HMA mixes.

2.6.3. Asphalt Binder

Asphalt is a sticky, black, and highly viscous liquid or semi-solid form of petroleum and it is found in natural deposits or by refined products. Asphalt materials are commonly grouped in various classes or grades based on consistency. Most asphalts contain carbon 82-88 %, hydrogen 8-11 %, sulfur 0-6 %, oxygen 0-1.5 %, nitrogen 0-1 % and it also contains trace quantities of metals such as vanadium, nickel, iron, magnesium, and calcium, which occur in the form of inorganic salts and oxides. Approximately 85% of all asphalt produced is used as the binder in asphalt for roads (Holý & Remišová, 2019). Asphalt as a binder serves two major functions in road pavement, first, to hold the aggregates firmly and second to act as a sealant against water. However, due to some distresses like fatigue failure, the performance and durability of asphalt are highly affected by changes with time in terms of its characteristics which can lead to the cracking of pavement (Mashaan et al., 2014).

The performance of bitumen difference at different temperatures related to its penetration grade, high penetration grade of asphalt means it is soft, higher cracking resistance at low temperatures area, and higher permanent deformation under loading at high temperatures area. On the other hand, asphalt with low penetration grade or hard bitumen has less low temperature cracking but more rutting resistance. This condition can be related to the complex behaviour of bitumen at different loading times and temperatures. To improve the behaviour of the asphalt mixtures, a well-known solution is a modification of bitumen using synthetic polymers for improvement of its properties of bitumen material (Fazaeli et al., 2012).

2.7. Mechanical Behavior of Hot Mix Asphalt Mixtures

The mechanical behaviour of hot mixed asphalt mixture depends on the properties of asphalt binder and the volumetric properties of the mixture components these are aggregate, asphalt, filler, and provided air void. The asphalt content, amount of filler added and provided air void in the mixture are the basic parameter to characterize the volumetric composition and performance evaluation of hot mix asphalt pavement. However, for a complete evaluation of the performance of hot mix asphalt mixture, the following mechanical properties such as elastic stiffness, resistance to fatigue cracking, and permanent deformation (rutting) are vitally important (Choudhary et al., 2019). Asphalt is a thermoplastic material that behaves stiffer or brittle when the temperature is a low and viscous liquid at high temperature and visco-elastic at moderate temperature. These properties of asphalt are highly affected by temperature and rate of loading, it affects the elastic stiffness of asphalt material. This means that for short loading time the stress-strain characteristic of asphalt mixture is elastic, but for long loading time and high-temperature condition leads to viscous response and is characterized by lower stiffness and permanent deformation (rutting) problem respectively. So, these are the fundamental or major rheological properties of asphalt binder and a versatile binder for making hot mix asphalt mixture. In addition to these properties, the asphalt mixture must be durable for a given service life of the pavement without excessive deterioration of the above mechanical properties (Silva et al., 2017).

The utilization of proper material and suitable mix design is the major way to reduce those failures listed above and improve the performance of hot mix asphalt. During the preparation of hot mix asphalt using three materials. These materials are asphalt, aggregate, and filler material. The quality of aggregate used in the mixture highly depends on the availability of local material. Nowadays, it is difficult to control the quality of asphalt binder during the refining process but using another alternative way to utilize modifier materials like polymer, fibres, sulfur, and other additive materials to modify the quality of asphalt binder (Habib et al., 2010). The filler material also a major role in the mechanical behaviour of hot mix asphalt. It has two major roles one is to fill the void between the aggregate skeleton in the mixture become denser, raise the stability, and improve the rutting resistance of pavement by increasing aggregate to aggregate contact surface area. Secondly, during mixed preparation filler materials mixed with asphalt binder and form asphalt-mastic. This property is modified the adhesive properties and overcoating binding nature and increasing the durability, and improve the properties of asphalt binder by reducing the binder's inherent temperature susceptibility. Therefore, an important role to control the mechanical properties of the asphalt mixture (Bi & Jakarni, 2019).

2.8. Mineral Filler in the Hot Mix Asphalt Pavement

Mineral fillers are the finest particle that passes through 0.075mm sieve size. It is used in the production of asphalt mixture. Essentially, it has two main roles one is to fill the void between the aggregate skeleton in the mixture and become denser, raise the stability, and improve the rutting resistance of pavement by increasing aggregate to aggregate contact surface area. Secondly, during mixed preparation filler materials mixed with asphalt binder and form asphalt-mastic. This property is modified to the adhesive properties and overcoating binding nature and increasing the durability, and improve the properties of asphalt binder by reducing the binder's inherent temperature susceptibility (Bi & Jakarni, 2019). In the constructions of the road in Ethiopia, the most common types of filler materials used in the production of asphalt mixture are Limestone, Portland cement, hydrated lime, fly ash, and stone dust. Due to this factor, the production of filler material is an insufficient amount (Ararsa et al., 2019). However, in the construction of asphalt pavement, the main issue is how to get enough amount, better quality, and locally available materials. Based on this issue it should find alternative materials that used replaced the conventional filler with other filler material that gives a better performance in the production of hot mixed asphalt mixture (Varma & Lakshmayya, 2018). Nowadays, Some of these materials using as alternative materials and different researchers have been carried out the possibility of conventional materials are replaced by waste and by-products material from recycling processes used in road construction those materials such as Bagasse ash, Rice husk ash, Fly ash, waste glass powder, sub-base course dust, Sawdust ash, Ceramic dust, Brick dust, Marble dust, Coal waste, and wood ash (Sangiorgi et al., 2017). Therefore, in this research used coffee husk ash is used as a filler material in the production of asphalt mixture because it is easily available and low cost in our country.

Coffee husk is the main solid residue or by-product during the processing of coffee. According to the International Coffee Organization, (2016) Ethiopia is the fifth country that produced coffee in the world about 384,000 metric tons (384 million Kg) per year. From this production, around 192,000 metric tons (192 million Kg) of coffee husk residual or by-product is produced during the processing of coffee. The solid waste (coffee husk) is used as a supplement for animal feed but most of this waste around 134,400 metric tons (134.4 million Kg) are discarded, fill, and burned in the ground (Sime et al., 2017). By doing this, to polluting the environment. It is greatly important that converting to value-added products. So it has to do better as a professional, utilizing these solid waste material (coffee husk) by using furnace convert to ash and used as an alternative filler material in the production of hot mix asphalt mixture and improve some mechanical properties of hot mix asphalt mixture.

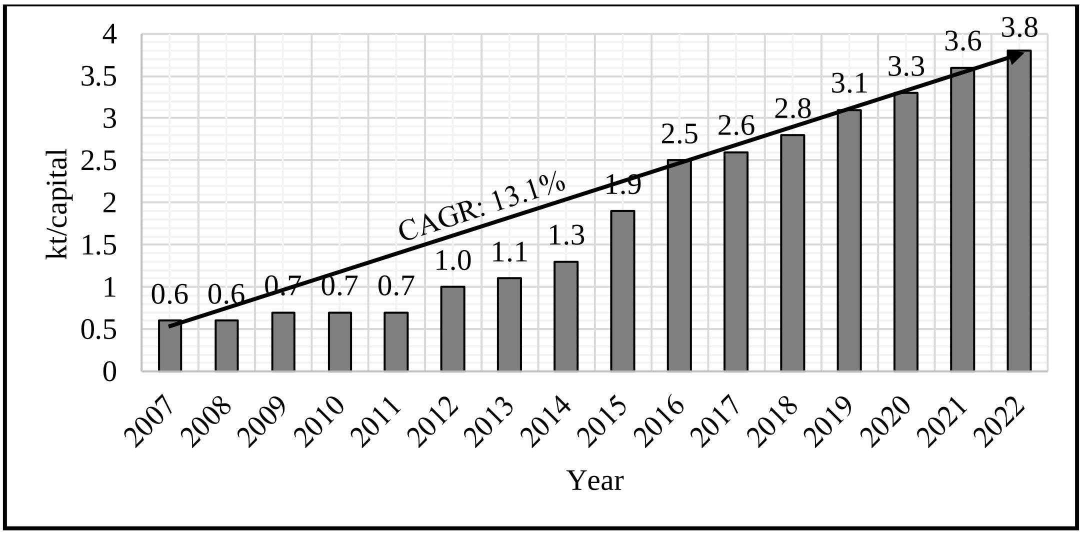
Different scholars used different types of filler materials in the hot mix asphalt mixture some of these are here. Baby et al., (2017) used waste glass powder as a filler material in the asphalt mixture and the result showed that higher stability and lower flow value of about 44% and 3.79% respectively of normal mixes with quarry dust. Ararsa et al., (2019) used sub-base course dust as a filler in the production of asphalt mixture and the result showed that increasing the marshal stability, density and decrease flow, void in the mineral aggregate and void fill with asphalt compared with hydrated lime, marble dust, and ordinary Portland cement. Zainudin et al., (2016) used Bagasse ash as a filler material in the hot mix asphalt mixture and the result showed that the filler is effective in increasing the Resilient Modulus, marshal stability, and flow by 17.4%, 0.6%, and 4.9% respectively of ordinary asphalt mixture. Rashwan, (2016) used limestone powder as a filler in the production of asphalt mixture and the result showed that increasing the marshal stability, density, Indirect tensile strength, and decrease flow, void in the mineral aggregate, and strain failure compared with conventional filler. Bi & Jakarni, (2019) used wood ash as a filler material in the asphalt mixture and the result showed that better resistance to permanent deformation, increase the resilient modulus, decrease rutting, and improve fatigue performance compared with conventional filler. However, a few researchers are used coffee husk ash as a stabilizer in subgrade treatment and partial replacement of cement in the production of concrete but no one can be used coffee husk ash as filler material in the asphalt mixture but Our country produced a massive quality of coffee husk produced during coffee production and poorly utilizing this waste material. Therefore, the above results showed that partial or full replacement of conventional filler with other waste material significantly improves the performance of the asphalt mixture.

2.9. Waste plastic in road construction

2.9.1. Polymer (Plastic)

From its early beginnings during and after World War II, the commercial industry for polymers long-chain synthetic molecules of which “plastics” are commonly produced and grown rapidly. Plastics is a versatile packing material that comes from the ability to mould, laminate, or shape and it is non-biodegradable material after it uses, gets mixed with domestic waste, and makes the disposal of municipal solid waste and become a problem to the environment. Due to population growth, industrialization, consumers, and technological development, there has been a tremendous increase in the rate of production of waste. In 2016, the world's cities generated 2.01 billion tons of plastic, amounting to a footprint of 0.74 kilograms per person per day (Kaza et al., 2018).

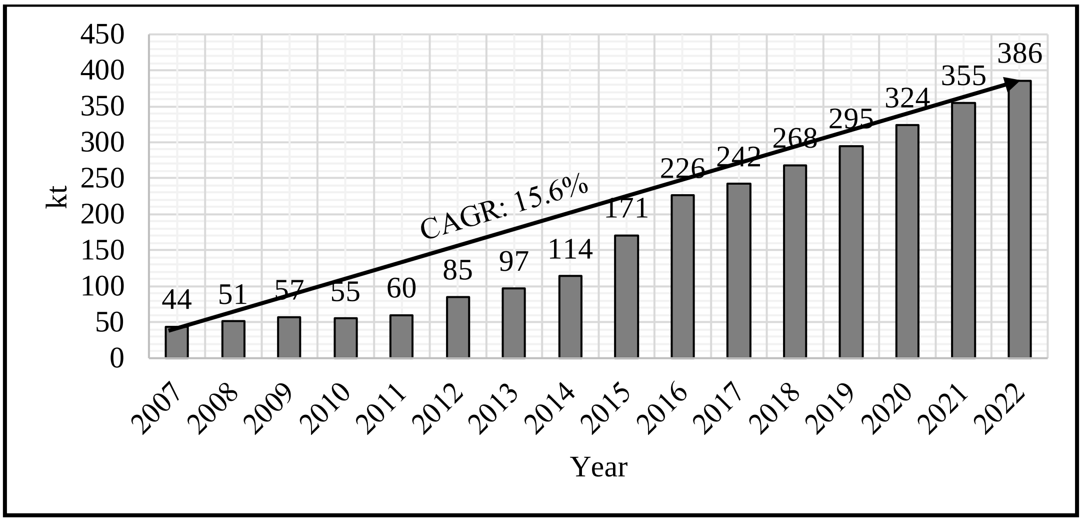
In Ethiopia, the per-capita plastic consumption has grown by about 13.1% annually over the past years, the *Figure 2.2* show that from 0.6 Kg in 2007 to 2.8 Kg in 2018 and is estimated to be 3.8Kg in 2022.



Source: (https://addisstandard.com/analysis-the-plastic-pile-weighing-on-ethiopia/, Access date: July 07, 2021 G.C).

Figure 2.2 Per capita plastic consumption over a year in Ethiopia

Figure 2.3 shown that the country's plastics consumption has grown by 15.6% annually over the past year, from 44kt in 2007 to 268 kt and is estimated to be 386 kt in 2022.



Source: (https://addisstandard.com/analysis-the-plastic-pile-weighing-on-ethiopia/, Access date: July 07, 2021 G.C).

Figure 2.3 Plastic consumption over a year in Ethiopia

Different literature uses waste plastic as an Asphalt modifier in the construction of paved roads to improve the performance of HMA and reduce environmental pollution. Dhriyan & Bhardwaj, (2017) pointed out that plastics have non-biodegradable characteristics and harmful to human health, therefore, disposal of waste plastic is of great concern. The municipal solid waste stream is recycled, as well as considering the very serious global. Countries such as Denmark, Netherlands, the USA, and India have made the recycling of wastes in construction a national priority. Recycled wastes have been found useful in the construction of roads especially in rural areas (Sojobi et al., 2016).

2.9.2. Types of Waste plastic

Polymers are generally categorized as Elastomers or Plastomers. Elastomers refer to the ability of a material to return to its original shape when a load is removed. It includes copolymers of styrene and butadiene. Plastomers attain high strength and resistance to deformation at a rapid rate but are brittle. It includes ethylene-vinyl acetate, polyethene, and various compounds based on polypropylene. Elastomers and Plastomers polymers are more classified as either thermoset or thermoplastic. Thermoplastic is a plastic material that becomes moldable at a specific temperature and solidifies upon cooling. Therefore, thermoplastics can be reshaped by heating. It uses polyethene a type of thermoplastic in the construction of roads whereas thermoset plastics are synthetic materials that strengthen during heating and but cannot be remoulded after the initial process was completed (Chukka & Carr, 2016).

The Society of the Plastics Industry (SPI) established a special numbered coding system in 1988 to allow consumers and recyclers to properly identify the type of plastic. Manufacturers follow a coding system and place an SPI code, or number, on each plastic product. *Table 2.2* illustrates the common types of plastics used, applications, and SPI code.

Table 2.2 Types of plastics, their applications, and the SPI code (Archna et al., 2015)

No.	**Plastic Type**	**Abbreviation**	**Example of Application**	**SPI**
1	Polyethylene Terephthalate	PET	Soft drink and water bottles.	01 PET
2	High-Density Polyethylene	HDPE	Cleaners and shampoo bottles, moulded plastic cases	02 PE-HD
3	Polyvinyl Chloride	PVC	Pipes, fittings, credit cards, toys, electrical fittings, pens; medical disposable; etc.	03 PVC
4	Low-Density Polyethylene	LDPE	Grocery bags and packaging films	04 PE-LD
5	Polypropylene	PP	Bottle caps, diapers, and syrup bottles, also produced as fibres and filaments for carpets	05 PP
6	Polystyrene	PS	Take-away food containers, egg cartons, plastic forks, and spoons.	06 PS
7	Other types of plastics		Polycarbonate is Compact discs, eyeglasses, riot shields, security windows.	07 O

Generally, the shredded PET plastic modifier type was chosen for this study because of different factors including availability of the material, cost, and expected performance of the paved structure which is better than other types of waste plastic. The expected performance means utilization of PET would improve the performance of hot mix asphalt pavement by increase stability, stiffness, and viscosity, improve stripping, thermal cracking, temperature susceptibility, fatigue damage, and rutting resistance (A. F. Ahmad et al., 2017).

2.10. Experimental Investigation of Waste Plastic in Hot-Mix Asphalt

Scholars show different experimental investigations using waste plastic material in the hot mix asphalt pavement used as bitumen modifier and aggregate coted. Prasad & Sowmya, (2015) stated that Polyethylene terephthalate has been used as an asphalt modifier and the optimum content was obtained 6% by weight of bitumen to improve pavement stability. (Abd-Allah et al., 2014) stated that low-density polyethene (LDPE) has been used as an asphalt modifier and content was typically 4-6% by weight of the modified binder. The addition of LDPE to asphalt cement reduces its penetration and increases its kinematic viscosity, absolute viscosity, and softening point. (Sarkar, 2019) stated that by adding waste plastic the property of bitumen improved, increase the marshal stability, and saving the usage of bitumen when using waste plastic as an asphalt modifier. Azizi & Rashid, (2018) pointed out that modified bitumen by plastic shows good properties as compared to original asphalt. But if to add more percent of the plastic in bitumen the blend gets separates on cooling and finally affects the properties of asphalt. In the dry process, the aggregate is coated with plastic. The aggregate coated with plastic shows that improved binding properties due to the increased contact area between asphalt and polymer. Neha et al., (2015) explain that the addition of plastic is the innovative technology that strengthens road construction, improves the property of bitumen, lower rutting depth, and more durable compared to the conventional flexible pavement.

2.10.1. Plastics Utilization in Asphalt Mixes

Several investigations have been carried out of using plastic to improve the performance of asphalt mixtures and can serve as a binder modifier furthermore it can be used as an aggregates coating material.

2.10.1.1. Using plastics for binder modification

Prasad et al., (2013) investigated the possibility of using waste PET as a binder modifier and he mixed in proportions 2-10% (by the weight of OBC) at an interval of 2%. Finally resulted in higher resistance to rutting due to their higher softening point when compared to conventional asphalt. Sahu & Singh, (2016) pointed out that penetration and ductility values of modified asphalt were decreasing with the increase in the proportion of the plastic additive, up to 12 % by weight and polymer bitumen blend is a better binder compared to plain asphalt. Abdo & Khater, (2018) stated that asphalt mixes with 4% waste plastic yielded the highest Marshall Stability and smallest flow and better resistance to rutting compare to a conventional mix.

2.10.1.2. Using Plastics as an Aggregate Coat

Sahu & Singh, (2016) stated that aggregate coating by plastics reduces the porosity, absorption of moisture, and improves the soundness of aggregate. Waste Plastic can be used in the asphalt wearing course as an aggregate coat and the optimum waste plastic content value is 12% of HMA by the weight of OBC. Manju et al., (2017) indicated that the waste plastic when added to hot aggregates, form a fine coat of plastic over the aggregate, and such aggregates, when mixed with a binder, is found to have higher strength, higher resistance, and better performance over a time.

2.11. 3D-Move Analysis Software (ver. 2.1)

2.11.1. Overview of 3D-Move Analysis Tool

3D-Move analysis software is a continuum-based finite-layer approach to analyze asphalt pavement responses under a variety of moving traffic load, traffic velocity, axle configurations, and tire contact area to a pavement structure. This approach treats each pavement layer as a continuum and uses the Fourier transform technique. Therefore, it can handle complex surface loadings such as multiple loads and non-uniform tire pavement contact stress distribution. The software was developed by the Asphalt Research Consortium. Which is a group of five organizations; Western Research Institute, Texas A&M University, University of Wisconsin-Madison, University of Nevada-Reno (UNR), and Advanced Asphalt Technologies and available at http://www.arc.unr.edu/Index.html.
In addition, 3D Move Analysis Software allows inputting the frequency sweep test data (E* and G* test data) of asphalt mixes in the analysis. It was taken into account the properties of the viscoelastic material in the analysis. Furthermore, 3D Move Analysis software utilized stresses and strains due to loading to predict rutting depths, top-down, and bottom-up cracking due to loading of asphalt pavement (Abdo & Khater, 2018).

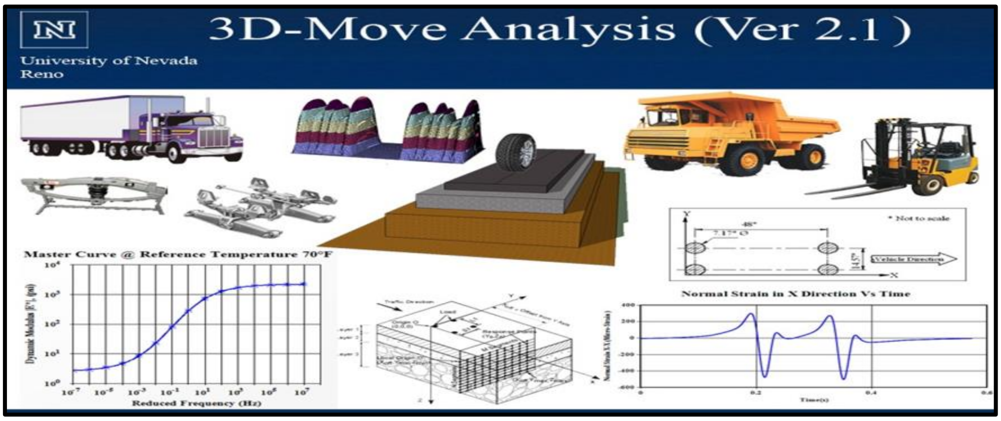


Figure 2.4 Screen Shoot of Main Screen of 3D Move Analysis software

Mechanistic procedures for estimating pavement responses due to traffic load were introduced in the early 1960s and have been evolving since then. For different performance models, the Strategic Highway Research Program (SHRP) has used critical inputs such as the applied stresses due to traffic load and compute pavement responses. It accounts for the changes in characteristics of vehicle loading, pavement materials, and the method of pavement construction (Grellet et al., 2012).

2.11.2. 3D-Move Analysis of software selection criteria

Most of the current mechanistic procedures used to compute pavement responses are much simpler; the stress distributions at the tire-pavement interface are modelled as static, uniform, and stationary circular loads. For example, ELSYM5, WESLEA 3.0, BISAR 3.0, CIRCLY 7.0, KENLAYER, ILLI-PAVE, MICH-PAVE (Ghadimi, 2015). Therefore, compare 3D-Move Analysis software with others based on different conditions. These are:-

2.11.2.1. Model Approach

The 3D-Move model has been recognized as an efficient tool to simulate moving loads as compared to the other software because the 3D-Move program uses a finite layer approach and Fourier Transform Technique for estimating pavement responses. This Fourier series expansion decompose the loads into harmonic components in space (x and y directions) and total response at a given location is then calculated by adding the individual responses from each harmonic component. It can handle any number of layers with complex loading at the surface and any number of response evaluation points. The 3D-Move model is ideally suited for pavement response evaluations since only a few critical responses are needed for pavement performance evaluations. For example, maximum horizontal tensile strain at the bottom of the AC layer and maximum vertical compressive strains within the layers are typically used to investigate fatigue and rutting failures of flexible pavements. Therefore, 3D-Move performs much more computationally efficient than the moving load models based on the finite element method such as KENLAYER, ILLI-PAVE, and MICH-PAVE (Nasimifar, 2015).

2.11.2.2. Tire-Pavement Interaction

3D-Move uses tire-pavement interaction-induced loading to model pavement responses. This is a critical factor considering that the noncircular loaded area and non-uniform contact stress induced from the tires can significantly affect pavement response computation. In addition, the tire-induced load varies with the speed of the vehicle as it travels through the pavement. To ensure the success of mechanistic modelling, tire-pavement interaction, loading characteristics, and material behaviour need to be incorporated realistically. However, Conventional multi-layer programs such as BISAR, ELSYM, and WESLEA software are simple to use, but they do not accurately consider the mechanisms associated with moving tire-induced loading on the pavement. Most of them are limited to defining static uniform circular loads. Moreover, the software developed based on Burimister's layered theory solves for an elastic multilayered system under stationary single or multiple circular loaded areas with uniform normal contact pressure (Elseifi et al., 2019).

2.11.2.3. Defining Loading Characteristics

The load applied by a moving vehicle varies with its travelling speed and pavement surface characteristics. This variation in the moving load is typically ignored by conventional response analysis tools. However, 3D-Move analysis is considered loading characteristics with different travelling speed because vehicle speed influences pavement response computation such as strain and deflection.

2.11.2.4. Axle Configuration and Contact Pressure Distribution

In 3D-Move Analysis the pavement contact stress distribution is input data and assumes simple contact stress distributions such as circular or elliptical loaded areas with uniform vertical stress. However, in reality, the pavement contact stress distributions are non-uniform and more complex. The 3D-Move Analysis considers six types of loading conditions and classified into an option, each option has been predefined in different load cases. These are: - Option A: Pre-defined load cases, Option B: User selected Pre-defined Axle/ Tire Configuration (Uniform pressure), Option C: User selected Tire Configuration and Contact Pressure Distribution from Database, Option D: Semi-Trailer Truck Including Vehicle Dynamics, Option E: Special Non-Highway Vehicles, and Options F: User-Input Tire Configuration and Contact Pressure Distribution (3D-Move Analysis Software ver. 2.1 manual, 2013).

Option A: Pre-defined load cases:-There are 9 load cases included under this option and they represent many widely-used field cases. Users cannot modify the axle configuration and contact pressure distribution of any of the load cases.

Option B: User selected Pre-defined Axle/ Tire Configuration (Uniform pressure):- Here user can specify axle configuration and three types of contact pressure distributions (Circle, Ellipse, or Rectangle).

Option C: User selected Tire Configuration and Contact Pressure Distribution from Database:- In this case, VRSPTA and Kistler databases which have reported measured contact pressure distributions for a variety of tire types that included single and wide base tires under a tire pressure range of 220–1000 kPa and a tire load range of 26–106 kN are considered in both databases. The user is allowed to specify the axle configuration, and tire load, a contact pressure distribution can be assigned from the available databases.

Option D: Semi-Trailer Truck Including Vehicle Dynamics:- Under this option, the load distribution on various tires of the 18-wheel semitrailer and during braking is initially computed. Braking causes the vehicle to decelerate and the loads to transfer to the front of the vehicle and evaluate the contact pressure distribution.

Option E: Special Non-Highway Vehicles:- In this case, End Dump Truck and Forklift are included. Users can select an appropriate axle configuration and tire load from a database of the manufacturer's specifications.

Option F: User-Input Tire Configuration and Contact Pressure Distribution:- This option is entirely open to the user to define the contact pressure distribution. It may be uniform or non-uniform contact pressure distribution.

2.11.2.5. Characterization of Asphalt Materials

The asphalt layer material can be characterized as a linear elastic material or as a viscoelastic material. The dynamic modulus is required for the viscoelastic analysis. Therefore, 3D Move Analysis Software allows inputting the frequency sweep test data (E* and G* test data) of asphalt mixes in the analysis. It was taken into account the properties of the viscoelastic material in the analysis and can be input in three different ways but the above software is not considered:

Dynamic Modulus Lab Data:- Asphalt materials properties can be specified using the dynamic modulus lab data. The 3D-Move Analysis incorporates the master curve, which enables the input of dynamic modulus at any selected pavement temperature in the analysis.

Witczak Model:- If no test data are available, then use a predictive equation to develop the master curve. This equation can predict the dynamic modulus of asphalt mixtures as a function of temperature, rates of loading, and aging conditions from information that is readily available from aggregate gradation, volumetric, and binder properties of the mixture.

User-Defined Materials Properties:- A set of data of |E*| as a function of frequency can be specified by the user. Other input variables (Poisson's and damping ratios) can be either specified as constants or as a function of frequency.

2.11.2.6. Performance Models

The current version of 3D-Move analysis software is equipped with two pavement prediction models: NCHRP 137A and VESYS performance models. These models primarily have cracking and rutting distress modes. However, the software listed above is not considered such a type of condition.

NCHRP 1-37A performance prediction model consists of six distress modes:

- AC (Asphalt Concrete) Top-down cracking
- AC (Asphalt Concrete) Bottom-up cracking
- AC (Asphalt Concrete) Rutting
- Base Rutting
- Subbase Rutting and
- Subgrade Rutting.

VESYS Model

The VESYS model is a well-documented probabilistic and mechanistic flexible pavement analysis computer program series. This model also has been incorporated into 3D-Move Analysis software to predict the structural responses and the integrity of flexible pavements. VESYS model for flexible pavements consists of a primary response model and a damage model. The primary response model is defined as the time-dependent state of stress, strain, or deformation existing in the pavement. The primary response model represents the pavement system by an n-layer semi-infinite continuum such that the upper n-1 layers are finite in thickness while the bottom layer is infinite in extent. All layers are infinite to a horizontal extent. The materials in each layer are assumed to be isotropic and homogeneous. While, the damage model in the VESYS model consists of three independent models to predict the accumulation of pavement cracking, rutting, and roughness within a given design period.

Under such circumstances, 3D-Move performs much more computationally efficient than the other moving load models based on the finite element method.

The following major assumptions used by the software are:

- The domain is composed of horizontal layers of uniform thickness, which can be of different materials.
- Each layer can be either linear elastic or linear viscoelastic with a set of uniform material properties (e.g., elastic modulus, Poisson's ratio, unit weight), which are time and space invariants.
- Layers are modelled as single-phase.
- The surface loads are assumed to move with constant speed (i.e., no acceleration) along the x-axis.

2.11.3. Material Type used by Software for Asphalt Layer

Two types of materials can be assigned by the software for the asphalt layer: these are elastic and viscoelastic. The properties required for the elastic materials are Young's modulus E and Poisson's ratio. The properties of the elastic material are constant and are not influenced by loading frequency. On the other hand, the viscoelastic material properties are influenced by loading frequency.

2.11.3.1. Linear Elastic Materials

The properties required for the elastic materials are Young's modulus E, Poisson's ratio, and damping ratio. The properties of the elastic material are constant and do not vary as a function of frequency.

Poisson's ratio - Usually Poisson's ratio of bituminous materials varies with temperature and it ranges between 0.15 and 0.50. In 3D-Move Analysis, Poisson's ratio can be provided by either Constant Poisson's ratio or Poisson's ratio from the model.

Constant Poisson's ratio - In this case, the user needs to input a value for Poisson's ratio between 0.01 and 0.495. Table 2.3 shows the typical Poisson's ratio ranges for asphalt materials.

Table 2.3 Typical Value for Poisson's Ratio

Temperature (°F)	Poisson's ratio
< 0	< 0.15
0 - 40	0.15 - 0.20
40 - 70	0.20 - 0.30
70 - 100	0.30 - 0.40
100 - 130	0.40 - 0.48
>130	0.45 - 0.48

Source: 3D Move Analysis Software Manual

Poisson's ratio from the model- In this case, Poisson's ratio is estimated from the following equation.

$$\mu_{ac} = 0.15 + \frac{0.35}{1+e^{(a+bE_{ac})}} \quad \text{.......................(2.4)}$$

Where μac = Poisson's ratio asphalt mixture at a specific temperature, Eac= Modulus of asphalt mixture at a specific temperature, and a,b = constants

And Typical value for a and b are ; a = -1.63, b = 3.84E-6

2.11.3.2. Viscoelastic Materials

The modulus properties of asphalt concrete are known to be a function of temperature, rate of loading, age, and mixture characteristics such as binder stiffness, aggregate gradation, binder content, and air voids. It has been shown that the role of temperature and rate of loading on the modulus can be accounted for by adopting a master curve approach (constructed at a reference temperature). In 3D-Move Analysis, there are three different options to provide inputs for viscoelastic materials. They are:

1. Dynamic Modulus data
2. Witczak Model
3. User-Defined Properties

1. **Dynamic Modulus Data-** In this option, properties of asphalt materials are defined by E* master curves. Master curves are developed using the principle of time-temperature superposition. These curves are constructed by shifting E* data concerning the time until the curves merge into a single smooth curve at a selected reference temperature. Then using shift factors, the E* curve for the analysis temperature is constructed. The dynamic modulus master curve can be represented by the following equation below:

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta+\gamma \log(t_r)}} \quad \text{............(2.5)}$$

Where, E* = dynamic modulus, tr = time of loading at the reference temperature, α,δ = fitting parameters, ß,γ = parameters that describing the shape of the sigmoidal function for a given set of data

δ = minimum value of E* and δ+α = maximum value of E*;

The master curve and shift factors also can be developed based on Asphalt Mixture Properties and Asphalt Binder Properties

- **Asphalt Mixture Properties-** To construct the master curve, laboratory test data along with asphalt binder properties are required. The user needs to select the number of temperature and loading frequency, inputs the E* data.
- **Asphalt Binder Properties-** The master curve and shift factors also can be developed based on the relationships between binder properties. These relationships are established based on A and VTS. The following equation shows the relationship between A, VTS, binder viscosity, and temperature:

$$\log\log\eta = A + VTS\log T_R \quad \text{……………..(2.6)}$$

Where, η = binder viscosity, T_R = temperature in Rankine at which the viscosity was estimated, A and VTS = regression parameters.

There are two levels of selection with the different options in 3D-Move Analysis to calculate these values.

Level 1

At this level, actual laboratory test data are required to calculate the A and VTS values. There are two options. They are:

- Superpave Binder test data and Conventional Binder test data.

Superpave Binder Test Data- In this case, binder test data which consists of binder shear modulus (G*), phase angle (δ), and temperature (T) for the loading rate of 1.59 Hz (10 rad/s) from dynamic shear rheometer tests are required to estimate A and VTS. To get the relationship between binder viscosities with temperature, first, binder stiffness will be converted to viscosity at each temperature by using the following equation:

$$\eta = \frac{1000 * G}{10}\left(\frac{1}{Sin\,\delta}\right)^{4.8268} \quad \text{……………. (2.7)}$$

Then using linear regression, A and VTS parameters will be calculated

Conventional Binder Test- In this case, conventional binder test data are used to calculate the A and VTS values. The following equation is used to convert the penetration data to viscosity.

$$\log \eta = 10.5012 - 2.2601 \log(pen) + 0.00389 \log(pen)^2 \quad \text{.........(2.8)}$$

Where, η = Binder Viscosity (Poise), Pen = penetration for 100 g, 5 sec loading, mm/10.

Level 2

In this level, A and VTS are estimated from binder grading. If no test data are available, then the user can select this level. There are three options available to calculate the A and VTS values. They are:

- Superpave Binder Grade
- Conventional AC Grade
- Conventional Penetration Grade.

Superpave Binder Grade - Superpave binder grade depends on the high and low temperature of the pavement. The recommended values for A and VTS are based on the grading and they are shown in *Table 2.4*

Table 2.4 Recommended RTFO A and VTS Parameters Based on Asphalt PG Grade

High T^0 Grade (°C)	Lower Temperate Grade (°C)													
	-10		-16		-22		-28		-34		-40		-46	
	VTM	A	VTM	A	VTM	A	VTM	A	VTM	A	VTM	A	VTM	A
46	-	-	-	-	-	-	-	-	-3.901	11.504	-3.393	10.101	-2.905	8.755
52	-4.570	13.386	-4.541	13.305	-4.342	12.755	-4.012	11.840	-3.602	10.707	-3.164	9.496	-2.734	8.31
58	-4.171	12.316	-4.147	12.248	-3.981	11.787	-3.701	11.010	-3.350	10.035	-2.968	8.976	-	-
64	-3.842	11.432	-3.822	11.375	-3.680	10.980	-3.440	10.312	-3.134	9.461	-2.798	8.524	-	-
70	-3.566	10.690	-3.548	10.641	-3.426	10.299	-3.217	9.715	-2.948	8.965	-2.648	8.129	-	-
76	-3.331	10.059	-3.315	10.015	-3.208	9.715	-3.024	9.200	-2.785	8.532	-	-	-	-
82	-3.128	9.514	-3.114	9.475	-3.019	9.209	-2.856	8.750	-2.642	8.151	-	-	-	-

Conventional AC Grade- The recommended values for A and VTS based on the AC grading are shown in *Table 2.5*.

Table 2.5 Recommended RTFO A and VTS Parameters Based on Asphalt Viscosity Grade

Grade	A	VTS
AC - 2.5	11.5167	-3.89
AC - 5	11.2614	-3.791
AC - 10	11.0134	-3.695
AC - 20	10.7709	-3.602
AC - 30	10.6316	-3.548
AC - 40	10.5338	-3.51

Conventional Pen Grade- The recommended values for A and VTS based on the Pen grading are shown in the *Table 2.6*.

Table 2.6 Recommended RTFO A and VTS Parameters Based on grading

Grade	A	VTS
40 - 50	10.5254	-3.5047
60 - 70	10.6508	-3.5537
85 - 100	11.8232	-3.621
120 - 150	11.0897	-3.7252
200 - 300	11.8107	-4.0068

E* Master Curve Development- 3D-Move Analysis uses 3rd party tool to construct an E* master curve. In 3D-Move Analysis, it completes all the required inputs for the master curve, "Update Graph" will be enabled by the program *Figure 2.5*. By clicking this button, the user initiates the 3rd party tool to develop the master curve. Once it completes the calculation, the program will display the master curve in the same window.

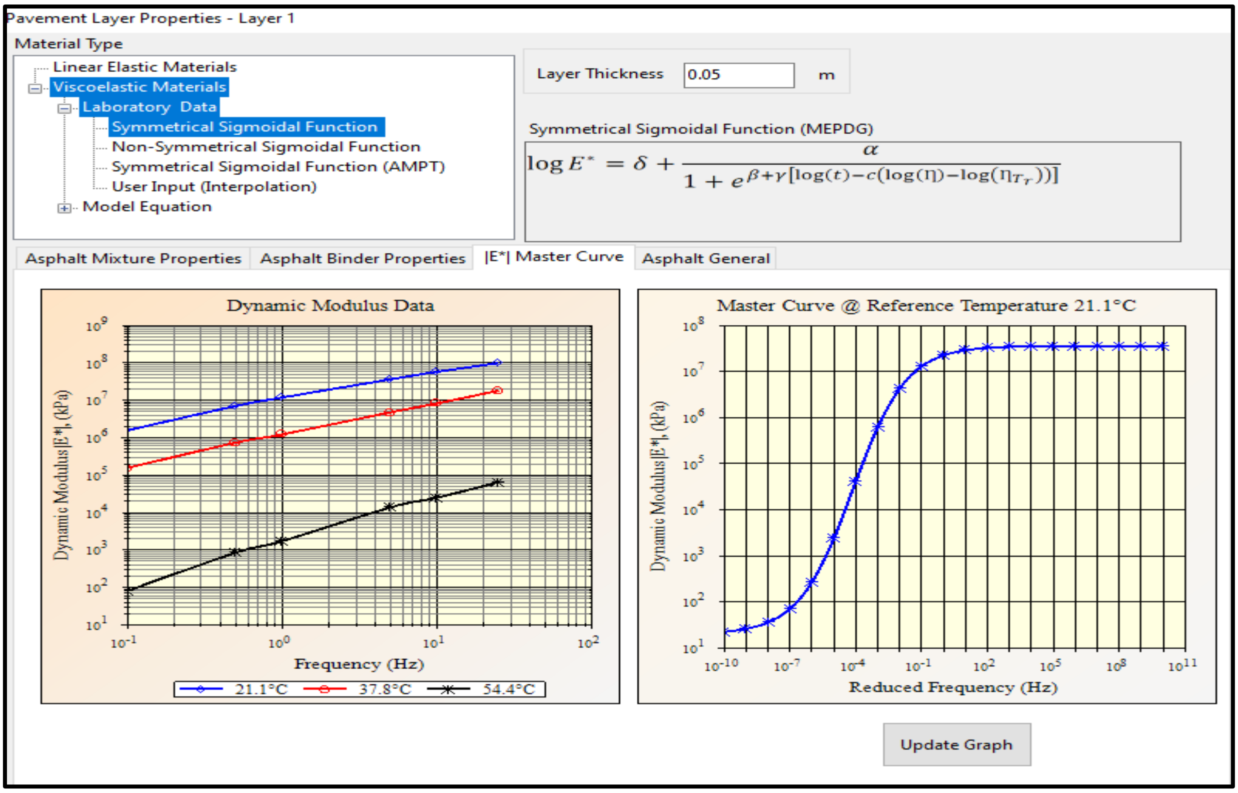


Figure 2.5 Screen shoot Input Window for Master Curve.

The next step is to develop E* curve at analysis temperature from *Figure 2.6*. Once the user inputs the analysis temperature, the program will display the E* curve associated with the analysis temperature on the same window.

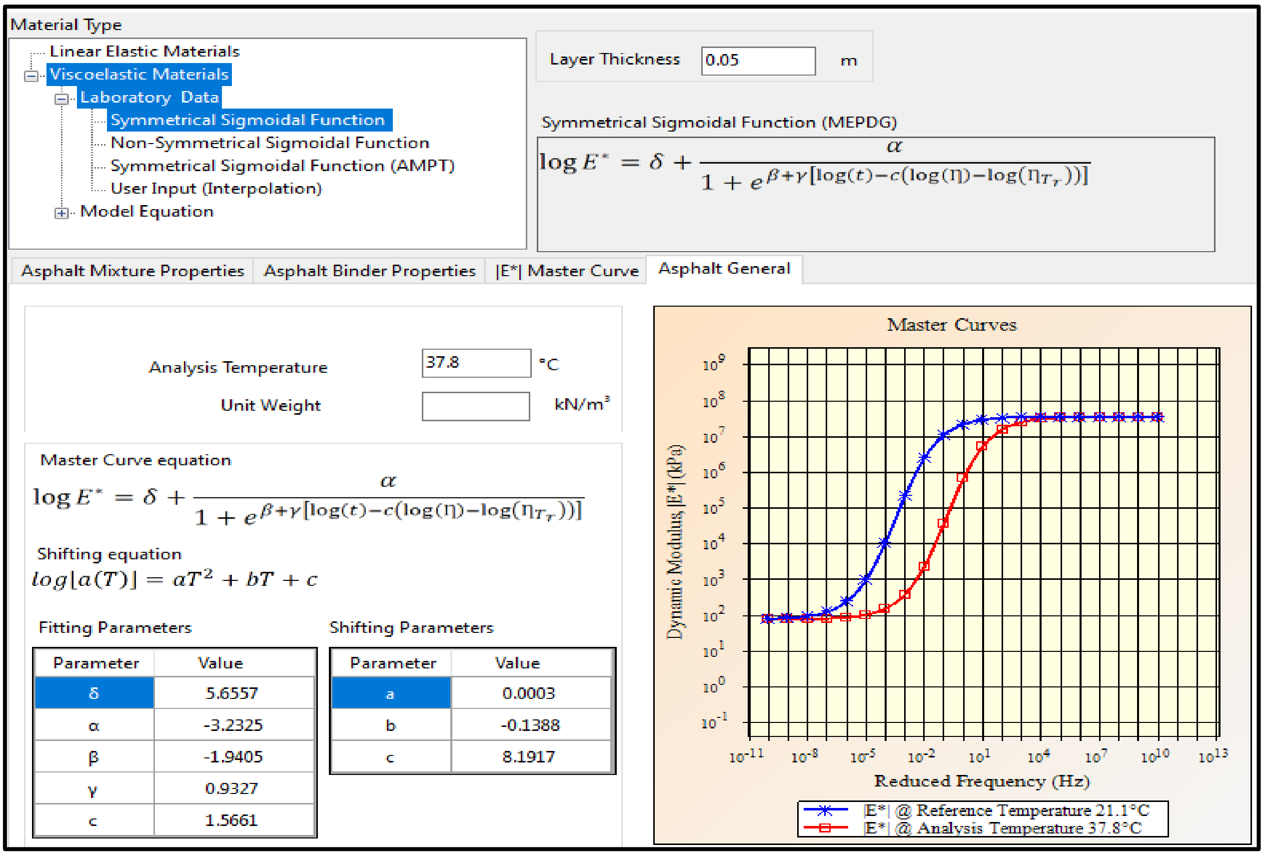


Figure 2.6 Screen shoot Input Window for E* Curve at Analysis Temperature

2. Witczak Model

If no test data are available, then the user can use this option to develop the E* curve. This option uses a predictive equation to develop the master curve. This equation can predict the dynamic modulus of asphalt mixtures as a function of temperature, rates of loading, and aging conditions from information that is readily available from material specifications or volumetric design of the mixture. The predictive equation is shown below:

$$\log E^* = 3.750063 + 0.02932(\rho_{200})^2 - 0.002841\rho_4 - 0.05809V_a - 0.802208\left(\frac{V_{beff}}{V_{beff} + V_a}\right) + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}}{1 + e^{(-0.603313 - 0.313351 log(f) - 0.393532\log(\eta))}}$$

……………………..(2.9)

$$\delta = -1.249937 + 0.02923\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a - 0.82208\left(\frac{V_{beff}}{V_{beff} + V_a}\right)$$

α $= 3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.00547\rho_{34}$

β $= -0.603313 - 0.313532 \log(\eta_{Tr})$

γ $= 0.313351$

t_r = reduced time of loading at reference temperature

η_{Tr} = asphalt RTFOT viscosity at the reference temperature, 10^6 Poise

Where:- E* = Dynamic Modulus, η = bitumen viscosity, f = loading frequency, Va = air void content, Vbeff = effective bitumen content, ρ34 = cumulative % retained on the 3/4 sieve, ρ38 = cumulative % retained on the 3/8 sieve, ρ4 =cumulative % retained on the No. 4 sieve, ρ200 = cumulative % retained on the No. 200 sieve.

3. User-Defined Materials

This option allows the user to input the E*, Poisson's ratio, and damping ratio as a function of frequency.

2.11.4. Features of 3D-Move Analysis Software

The main feature of 3D-Move analysis software are:-

- SI / US units.
- Static / Dynamic Analysis.
- Uniform Contact Pressure Distribution (Circle, Ellipse, and Rectangle).
- Non-Uniform Contact Pressure Distribution from Database.
- Semi-Trailer Truck including Vehicle Dynamics (Uniform / Non-Uniform Contact Pressure Distribution).
- Special Non-Highway Vehicle Loading (e.g. End-Dump Truck, Forklift).
- Dynamic Variation of Tire Load (Dynamic Loading Coefficient, DLC).
- Traffic Information.
- Dynamic Modulus, E* from lab data, or using Witczak Model for Asphalt Materials.
- Performance Models (NCHRP 1-37A and VESYS).
- Response Computations at an Array of Points.
- Text Output (Text files, Microsoft Excel files, and Graphical Output.

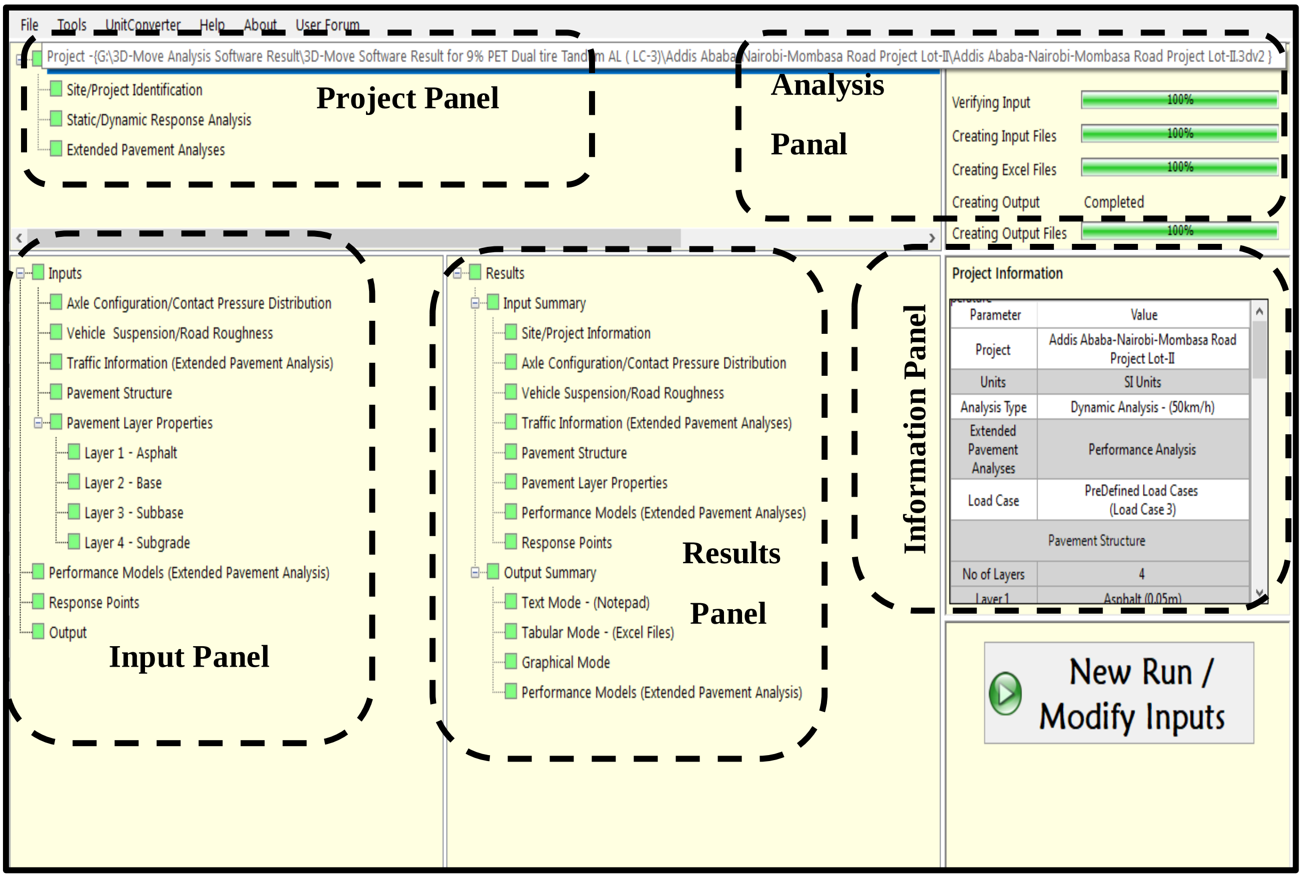


Figure 2.7 Screenshot of the Main window of 3D-Move Analysis after a new project is created.

2.11.5. Input Parameters for the software

The major inputs data that are used in the software are:

- Axle configuration and contact pressure distribution.
- Vehicle superposition and road roughness.
- Traffic information (extended pavement analysis).
- Pavement structure (Layer type, Material characterization, and thickness).
- Pavement layer properties (Asphalt, Base, Sub-base, and Subgrade layer).

2.11.6. Output Parameters from the Software

The final output from the software are:

- Text, Tabular, and Graphical Output (Stresses and Strains)
- Performance Models Results and pavement distress predictions (AC Top-Down Cracking, AC Bottom-Up Cracking, and AC Rutting)

2.12. Summary

Asphalt roads are the most popular and used roads around the world. Most of the Ethiopian roads also a flexible type of pavement. However, several structural defects occurred on the asphalt road due to the quality of material, climatic condition, traffic load, and other factors. That is why the major problem in the world. Based on these problems number of researchers to investigate an alternative solution according to local material availability and cost minimized. Grounded on the result of reviewed studies explain that the partial replacement of bitumen using the polymer (waste plastic) has a significant improvement on the rheological properties of asphalt binder in terms of increasing the Linear viscoelastic properties, rutting resistance at high temperature and performance grade of the bitumen. The results indicated from those studies clearly show the potential benefits of using waste plastic to improve the rheological properties of bitumen. In addition to this, possible to use an alternative waste material used as a filler material in the production of HMA mixture also shows an improvement in the mechanical properties of the mixture in terms of volumetric, marshal and flow value. However, due to climate, material variations, and availability, the use of coffee husk ash filler in HMA and PET plastic in the rheological improvement must be investigated using local materials, based on local specifications. Therefore, this study aims to investigate the effects of PET plastic on the rheological properties of bitumen and coffee husk ash filler in HMA specifically for Ethiopian local materials and climate. The study enhances and contributes significantly to the development of asphalt mix design using industrial waste materials and also provides better alternative material for conventional mineral filler and finally using 3D Move Analysis Software to evaluate the performance of pavement with and without PET plastic.

3. MATERIAL AND METHODS

3.1. Study Area

The research was conducted at Hawassa city in Yirgalem Construction plc laboratory room and Addis Ababa Institute of Technology highway laboratory, Addis Ababa Ethiopia.

3.2. Study Design

The study used an experimental research method to answer the research question and meet the objectives. During the research work the following phases were applied: - The first phase was sample collection. At this phase, Bitumen, Crushed aggregate, and coffee husk were collected. The second phase was laboratory testing. This phase is further classified into three steps.

- **Step-I Quality testing-**The quality of the sample was tested from each component of the materials, the physical properties of the samples were compared with the ERA specification, and checked the materials that fulfil the ERA specification.
- **Step-II Mix Design-** During the marshal mix design step, three types of mixtures were designed. a mixture of normal HMA with stone dust as a filler, a mixture of coffee husk ash as a filler in HMA, a combination mixture of coffee husk and shredded plastic at different ratios with HMA. The HMA mixture was tested and calculated the stability flow and volumetric properties according to marshal mix design and assessed their properties and check the properties they are suitable as wearing course material.
- **Step-III Rheological properties-** Using Superpave asphalt physical tests equipment and determining rheological properties of asphalt binder by adding a different proportion of shredded plastic in the asphalt binder. These are the Rolling thin film oven test (RTFO) and Dynamic shear rheometer (DSR).

3.3. The framework of the study

The general framework of the study is shown below in *Figure 3.1*

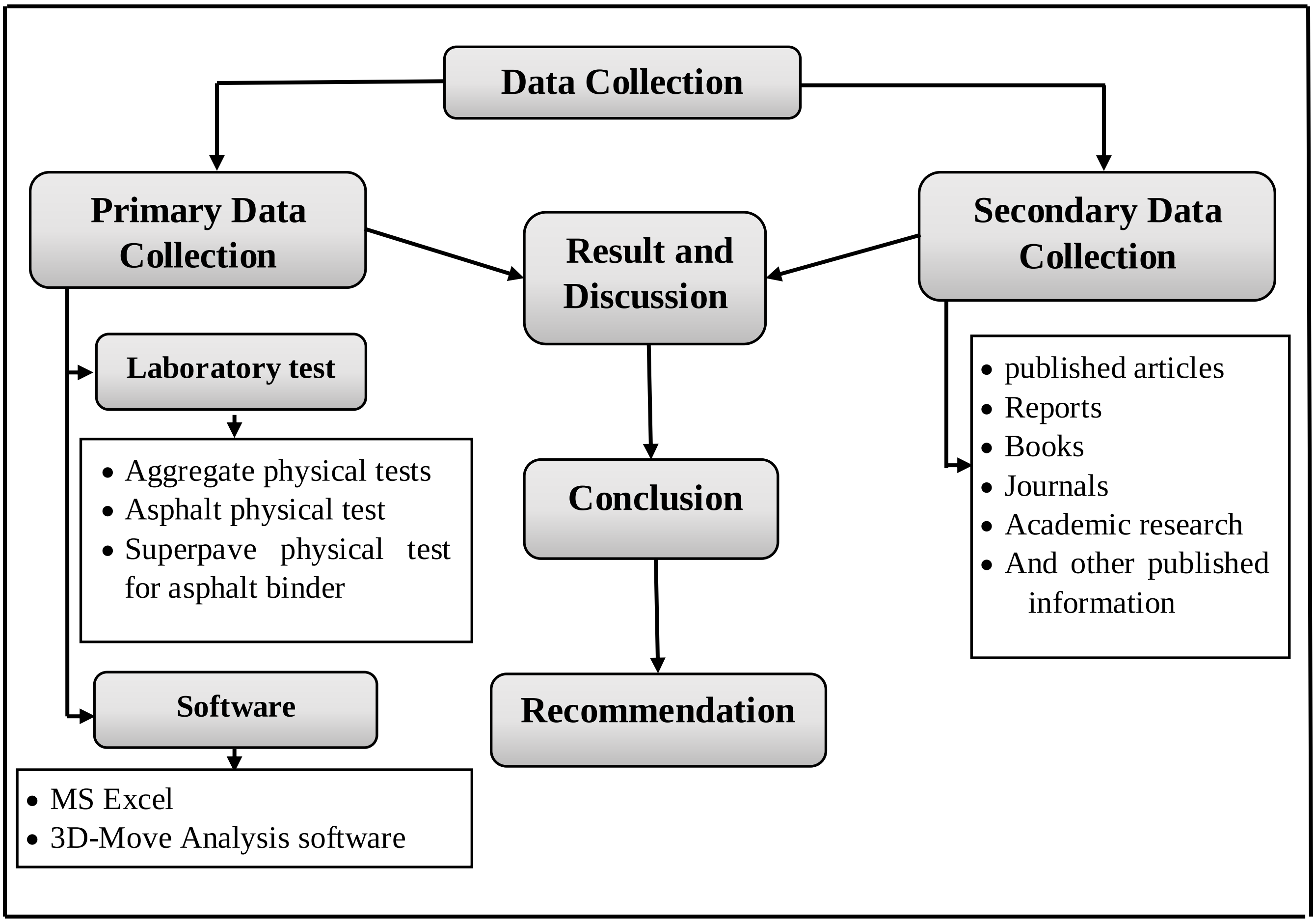


Figure 3.1 The major framework of the study

3.4. Sampling Technique

Purposive and quartering sampling techniques were used to sample preparation for conventional and Superpave asphalt physical test and marshal test. The purposive sampling technique is a non-probabilistic sampling method that occurs when elements selected for the sample are chosen by the judgment of the researcher and the quartering method means first divide the sample into four segments, the diagonally opposite of which are rejected. Then after the laboratory test to be undertaken to assess the material suitability. All the tests were performed according to the ERA standard.

3.5. Source and Data Collection

In this research, both primary and secondary sources of data were used. Primary data were collected from Laboratory test results. Whereas the secondary data were collected from published articles, reports, books, journals, academic research, and other published information. Secondary data may either be published or unpublished data.

3.6. Sample Collection

- **Shredded plastic Polyethylene terephthalate**

Shredded Polyethylene terephthalate plastic is taken from COBA Impact Manufacturing PLC located in Nefassilk Lafto Subcity at Addis Ababa Ethiopia. Their physical properties, such as gradation parameters and specific gravity were determined as shown in Table 3.1.

Table 3.1 Physical properties of Polyethylene terephthalate

Property of PET Plastic	
Plastic-type	Polyethylene terephthalate(PET)
Sieve Size (mm)	4.75-2.36mm
Density (g/cm^3)	1.38
Melting point	260^{o}c

Source: (Sojobi et al., 2016)

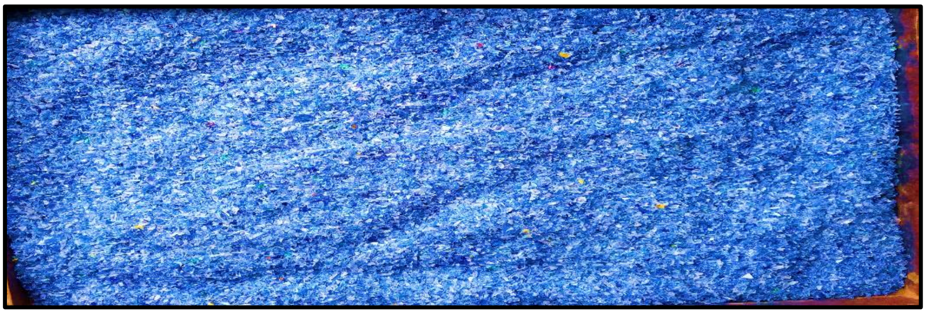


Figure 3.2 Shredded plastic passing through 4.75mm and retained on 2.36mm sieve size

- **Aggregate**

The Aggregate is taken from Hawassa city at Yirgalem Construction material production stock. The collected samples were transported to the Yirgalem Construction highway laboratory room to conduct the tests.

- **Coffee Husk-ash as filler material**

The dry coffee husk was collected from Yirgalem town in the Sidama region. It is found at 255 Kilometers distance from Addis Ababa and 35Km from Hawassa city. Then the coffee husk burned in the furnace at 550 °C for 4 hours to make ash and taken as filler.

- **Asphalt Binder**

Bitumen 60/70 penetration grade was taken from the Yirgalem construction batching plant in Hawassa city and used material to prepare the specimens. The collected materials were transported to Addis Ababa Institute of Technology, Highway laboratory room Addis

Ababa, Ethiopia. The criteria to select a 60/70 penetration grade is used in the project site above 27°C.

3.7. Experimental program

This investigation was based on the experiment as the main procedure to achieve the research objectives. Standard procedures (AASHTO, ASTM, ERA, and Tanzania manuals) were followed in the preparation of the Marshal Mix design, asphalt cement physical test, and Superpave physical test for asphalt binder. Several tests were conducted to measure the physical properties of the hot mix asphalt constituent materials and to compare them with recognized standards. In addition, the properties of modified hot mix asphalt were evaluated and compared with unmodified asphalt binders. Both the modified and unmodified were compared with recommended standards.

3.7.1. Raw Material

The materials used for this study are described in table Table *3.2*

Table 3.2 The raw material used for the study

Material		Function
Raw Materials	Coarse aggregate	Distribute the vehicle loads to the subgrade by grain-to-grain contact through the pavement structure safely.
	Fine aggregate	To improve the density and strength of the HMA mix.
	Filler Material (Coffee husk-ash)	Coffee husk-ash is used to fill the voids in the mixture and dense aggregate skeleton and form asphalt mastic.
	Bitumen	For binding and over-coating the aggregate.
	Shredded plastic (PET)	It is used as an aggregate coat and bitumen modifier of asphalt mixture.

Source: (Manju et al., 2017; Mosa, 2017; Nega et al., 2013; Sarir et al., 2019)

3.7.2. Mixing of waste plastic with asphalt Procedure

Shredded plastics were mixed with asphalt in two ways: By dry and wet process.

3.7.2.1. Dry Process

The shredded waste plastic that passing the sieve size of 4.75mm was used. The mixed aggregates were heated in a chamber at a temperature of 180^0C. (ERA Manual, 2013) recommend that the heated temperature of aggregate exceeded the compaction temperature of asphalt mixture by a range of 10-30^{0}C and also used a temperature of 150^{0}C heated asphalt binder for 60/70 grade asphalt binder. Then the collected plastic was put into the chamber and heated along with the aggregate. Until the plastic gets coated uniformly over the aggregate within 30min-45min. It was given an oily coated to the aggregate mixture. In the same way, the asphalt was heated at a temperature of 150^{0}C (Chukka & Carr, 2016). When this process is done to attain good binding and to prevent weak bonding because the plastic has a double binding property and improves the surface properties of the aggregates, practical in all types of climates, emission of toxic gases cannot be possible below 270^{0}C. When the plastic heated above 270°c it gets decomposed and above 750°c they get burnt to produce toxic gas. The main important thing that must be kept in mind is monitoring the temperature. In the next step, the hot aggregate mix was mixed with the hot asphalt at a temperature between 140^{0}C to 170^{0}C (Bale, 2011). This dry process was used for marshal mix design with the addition of shredded PET plastic.

3.7.2.2. Wet Process

The shredded plastic that is remained after passing through the 4.75mm sieve size used. The wet process used in the melt-blending technique. Asphalt heated in the oven till fluid condition and shredded plastic slowly added the temperature between 150-160^{0}C. The mix continued for 45-60min to produce homogenous mixtures. The polymer modified asphalt was sealed in containers and stored for further testing. Finally, Conventional and performance tests were used as penetration, softening point and viscosity, and Superpave physical test (Appiah et al., 2016).

3.7.3. Laboratory sample tests

This research evaluated the properties of HMA prepared in the laboratory using different percentages of shredded plastic from 3, 6, 9, and 12% by total weight of the optimum asphalt content and fully replaced stone dust with coffee husk ash. The materials used in the mixture include coarse, intermediate, and fine aggregates, filler material, shredded waste plastic Polyethylene terephthalate, and asphalt binder.

Several tests were conducted to measure the physical properties of the hot mix asphalt constituent materials. The materials test was taken for this study listed below:

3.7.3.1. Aggregate physical tests

Aggregates are used to produce hot mix asphalt mixtures. The aggregate selected for this study was crushed rock obtained from the Yirgalem construction stoke area of three different sizes (19mm, 9.5mm, and 4.75mm). To define the properties of these aggregates, some laboratory tests were conducted.

Table 3.3 Aggregate physical tests

No.	Aggregate Physical test		Test Method
1	The grain size distribution of Aggregates		AASHTO T 27
2	Specific gravity and water absorption of aggregates.	Specific gravity and water absorption of Coarse and Intermediate aggregate	AASHTO T 85-91 and ASTM C 127-88
		Specific gravity and water absorption of fine aggregate	AASHTO T 84-94 and ASTM C 128-88
3	Aggregate Shape Test	Flakiness index test	BS 812: Section 105.1: 1989
		Elongation index test	BS 812: Section 105.1: 1989
4	Aggregate Crushing Value Test-ACV		BS 812: Part 110: 1990
5	Los Angeles abrasion test-LAA		AASHTO T 96-94
6	Ten Percent Fines Value-TFV		BS 812-111:1990

Results of the aggregate properties are presented in **Appendix A - Materials Quality Test Results.**

3.7.3.2. Preparation of Coffee husk ash filler

The collected dry coffee husk was burnt under controlled conditions or by furnace kept at a temperature 550^0C for 4 hours to obtain the ash. After complete burning, the ash is allowed to cool for another 24 hours, the burnt ash was taken out and sieved by 0.075mm, and the fractions passing through the sieve were used throughout the test. The sieved ash was stored in a container to avoid absorb moisture. Figure 3.3 shows the product obtained after the process of coffee husk.

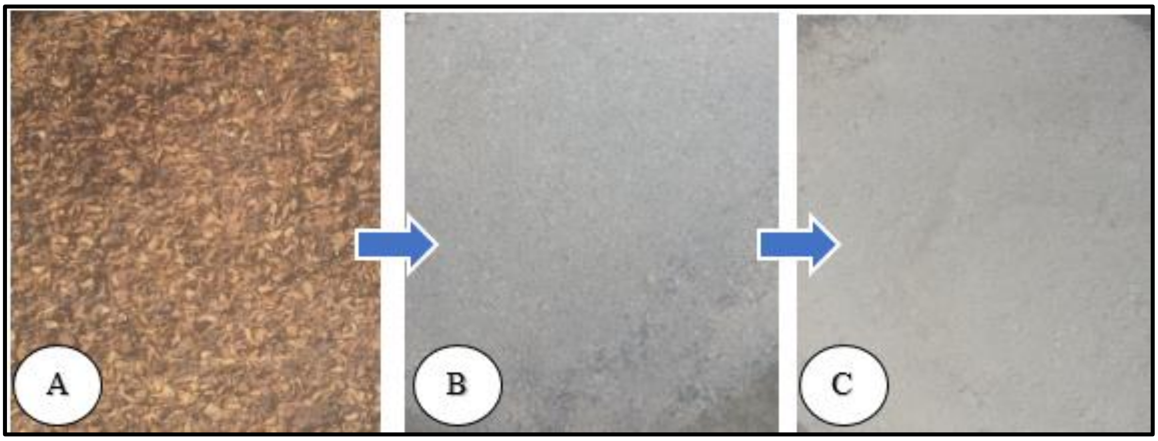


Figure 3.3 Dry coffee husk (A), Burned Coffee husk (B), and Coffee husk ash after Sieved (C)

3.7.3.3. Asphalt Binder physical tests

In this test, two Asphalt performance measuring methods were conducted. The first is the recently advanced equipment-developed performance measuring method by Superpave system (Superpave physical tests for asphalt binder). These are the Rolling thin film oven and Dynamic shear rheometer test. While the second is conventional performance measuring techniques (Conventional asphalt physical tests).

A. Procedures for Rolling Thin Film Oven Test (RTFO)

According to AASHTO T 240, a rolling thin film oven test (RTFO) follow the basic steps in aging asphalt binders. RTFO aging procedure is a conditioning step that simulates the construction aging of asphalt binders (Arabani & Shabani, 2019). Additional tests are performed on the residue from the test. The rolling thin film oven *Figure 3.4* consists of an oven chamber with a vertical circular carriage. Sample bottles rest in the carriage and the assembly rotates about the carriage center. A fan circulates air in the chamber. At the bottom of the rotation, an air jet blows hot air into the sample bottle.

Figure 3.4 The Main components of RTFO test Equipment

Where 1= Touch screen display, 2= Start-Stop button, 3= Door handle, 4= Squirrel cage fan, 5= Circular carriage with spring clips sample bottle container, and 6= Coiled tube for air jet.

Step-1 Turn on the main switch to preheat the RTFO for a minimum of 2 hours before testing with the fan on and the door closed.

Step-2 Heat the asphalt binder sample with and without PET plastic in a suitable container until the sample is sufficiently fluid to pour.

Step-3 Pour into each glass bottle 35 ± 0.5g of binder and Cool to room temperature.

Step-4 Carefully but quickly arrange the bottles in the carriage so that it is balanced with the oven at operating temperature, 163 ± 0.5^{o}C, place eight bottles containing asphalt into the circular carriage. Oven temperature is displayed on the touch screen display and will drop during the placement of the bottles, after 10 min achieved to the operating temperature.

Step-5 Immediately closed the door and begin the rotation of carriage assembly at a rate of 15 ± 0.2 rpm. As soon as start the airflow at a rate of 4000 ± 200 ml/min for the condition of 85min.

Step-6 After the 85-minute conditioning period, turn the oven off and stop the carriage, air flow, and circulating fan

Step-7 Remove the bottles, one by one, keeping them horizontal as they are removed from the carriage to prevent spillage and to facilitate immediate pouring. After each bottle is removed, pour the sample from the bottle into a suitable container. The container should have a large enough capacity to hold all material to be for further tested.

B. Procedures for Dynamic Shear Rheometer Test (DSR)

AASHTO T-315-10 “Standard Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)” and it is used to characterize the viscoelastic nature of the binder from 4-88°C temperature ranges. The thickness of the asphalt binder sample inserted between the fixed plate and oscillating spindle depends on the test temperature. Hence, for a temperature in the range of 4^0C - 40^0C required samples 8 mm in diameter plate is used with a 2mm gap between the upper and lower plates. Whereas, for high temperature greater than 46^0C required samples 25 mm in diameter plate with a 1mm gap between the upper and lower platens used (Cho & Im, 2014). The DSR apparatus used is as shown in *Figure 3.5*.

Figure 3.5 Dynamic Shear Rheometer Setup

Previous to testing a sample, a gap measurement verification procedure, called setting the zero-gap, conducted to ensure that the micrometer reading and the actual gap between the platens are the same. The zero-gap is set by lowering the upper plate in small increments until the upper and lower plates just touch, or reach zero gaps. The micrometer wheel was set to zero when the zero gaps between the platens have been achieved. Before setting the zero-gap, the temperature controller was turned on and the environmental chamber was preheated, or cooled, to the desired test temperature. The zero-gap was then set after the medium surrounding the platens stabilizes at that temperature. The accuracy of the gap measurement is directly related to the accuracy of the specimen evaluation, hence this procedure is essential.

Upon conducting the DSR test, the asphalt specimen was properly placed on the fixed plate allowing the upper plate to squeeze the asphalt specimen. The upper plate was lowered such that the gap between the two plates was 0.05 mm greater than the test gap. The excess asphalt was squeezed from between the platens and removed around the periphery of the platens then the sample bulged so that the gap between the upper and lower platens 2mm for the 8mm plate and 1mm for the 25mm plate. After the asphalt sample was correctly placed in the DSR and the test temperature appears stable, it allowed for a few minutes for the temperature of the specimen to equilibrate to the test temperature before conducting the test.

Sample preparation

To prepare the sample, the first step was to heat the asphalt binder upto $110^{O}C$ until it becomes fluid. Then shredded plastic was added 3%, 6%, and 9% by weight of asphalt binder, producing a total of 3 mix types with the controlled mix. Upon adding shredded plastic, the mix was contentiously stirred for 45-60min at a constant temperature to ensure good homogeneity. Then each sample was aged using Rolling-thin Film Oven (RTFO) following AASHTO T 240. Then the heated asphalt was poured into a mold and allowed to cool until solid enough to be removed from the mold. After removal from the mold, the asphalt disk was placed between the fixed plate and the oscillating spindle of the DSR for testing.

Accordingly, Four main types of tests were conducted using DSR in this research to determine the binder properties. These are:-

I. Performance grade determinations

This binder specification system works based on the climate at which the pavement is expected to serve by evaluating the contribution of the binder in resistance to permanent deformation, low temperature cracking and fatigue cracking in asphalt pavements. According to the Superpave, to carry out performance grade determination new set of tests of physical properties at a range of temperatures must be carried out. The performance grade

(PG) of the binder is designated as PGxx-yy, where xx represents the average seven days maximum temperature and yy represents the minimum temperature. Thus the types of samples were original binder and RTFO aged binder. And the test plates used were 25mm in diameter for a 1mm thickness of the specimen by considering higher temperature test because of the unavailability of pressure ageing vessel (PAV) to carry out long term ageing To determine the performance grading of asphalt binder, the basic rheological parameters (complex shear modulus(G*), phase angle (δ) and the rutting parameter (G/sin(δ)) were determined for the unaged and RTFO aged samples by DSR over a specified temperature (52, 58, 64, 70, 76, 82, and 88°C increase or decrease the temperature by increments of 6°C). According to the Superpave design limit, the minimum value of G/sin(δ) is 1kPa for unaged and 2.2kPa for RTFO aged samples as explained in AASHTO M320.

II. Amplitude Sweep Test (AST)

Using the rheometer software to applied a frequency at 10 rad/sec or 1.59 Hz. The test was conducted at three different temperatures i.e. at 21.1, 37.8, and 54.4°C using an 8mm diameter plate size. The dynamic rheological properties were tested by measuring the required shear stress to achieve a preset strain level for both aged and unaged mixes. The strain level should be large enough so that it is measurable and also small enough so that the required stress does not exceed the capacity of the testing device or damage the sample. The controlled stress level for AST was from 0.1KPa to 90KPa. The complex shear modulus G* versus strain plot was used to determine the linear viscoelastic region.

III. Frequency Sweep Test (FST)

Frequency sweep tests were also performed on the same temperatures as AST i.e. at 21.1, 37.8, and 54.4^{0}C. The tests were run on an 8mm parallel plate after the samples were allowed to equilibrate for a few minutes at each temperature before testing. The shear strain was applied for all the samples and the frequency range used was 0.1Hz to 25Hz.

- **Master Curves of Complex Shear Modulus**

In constructing the master curves using the time-temperature superposition principle at different temperatures and loading times, in terms of stiffness (shear complex modulus & Phase angle), determining temperature shift factors and constants at a specific reference temperature compared to a reference temperature, which is in our case 21.1 °C. The data at any other temperatures were shifted concerning the time until various curves overlap almost perfectly to form a single master curve (Al-Haddad, 2015). Modulus Master Curves were developed using Microsoft excel solver to best fit the sigmoid function for all asphalt binders. The sigmoid function to best fit the obtained G* data from the DSR is carried out by changing the sigmoid constants and the shift factor. The sigmoid function to calculate the complex shear modulus is represented below (Tadele & Quezon, 2021);

$$\text{Log}\,|G^*| = a + \frac{b}{1+e^{c\log fr+d}} \quad \text{Or} \quad \text{Log}\,|G^*| = \Delta + \frac{\alpha}{1+e^{(\beta+\gamma\log(fr)+Tr)}} \text{.........(3.1)}$$

The logfr obtained from the equation, $\text{Log}(f_r) = \text{Log}(f) + aT$

Where: a(Δ), b(α), c(β-γ) and d($T_{21.1}$): Sigmoid function constants, fr: Reduced frequency,

|G*|: Complex shear modulus, F: Frequency, aT-Temperature shift factor

William-Landel-Ferry (WLF) empirical relationship shown below was proposed to link the shift factor for each flow curve to the master curve, based on the time-temperature superposition to obtain the shift factor.

$$\log_{aT} = \frac{-C_1+(T-T_{ref})}{C_2+(T-T_{ref})} \text{............................(3.2)}$$

Where T is temperature, T_{ref} is the reference temperature, C_1 and C_2 are taken as constants.

- **Phase Angle Master Curve**

A relatively similar procedure followed for phase angle master curve like modulus master curves and with a slightly different form of the generalized logistic function, the binder phase angle master curves were developed as follows. The sigmoid /logistic function was;

$$\delta = 90bd\ \frac{\text{Exp}^{(c+d(\log fr))}}{\left(1+\text{Exp}^{(c+d(\log fr))}\right)^2} \text{.........................(3.3)}$$

IV. Multiple Stress Creep and Recovery (MSCR)

Before conducting the MSCR test performance grade determination was conducted to decide the test temperatures for the MSCR test using the same device. Then the repeated shear creep loading test was performed. Constant shear stress of 0.1 KPa and 3.2 KPa was used for samples having a 25mm diameter using a 1mm gap between the platens. The shear loading and unloading were applied for a total of 10 seconds i.e. 1-second creep load followed by 9 seconds recovery or rest period. During each cycle, the asphalt binder reaches a peak strain and then recovers before the next cycle stress is applied again. The permanent strain is then accumulated. The test was started with the application of a low-stress 0.1 KPa for 10 creep/recovery cycles, i.e. sample conditioning then another 0.1KPa with 10 creep/recovery cycles were repeated then the stress was increased to 3.2 KPa. This analysis is carried out considering the main purposes of Rutting prediction, an indication of elastic response and specification preparation of a binder. To evaluate those properties the main parameters are non-recoverable creep compliance, percent creep compliance difference and the percent elastic recovery (Tadele & Quezon, 2021).

- The percent recovery value ($R_{0.1}$, $R_{3.1}$) indicates an elastic response and stress dependency of modified and non-modified asphalt binder. It measured how much sample returns to its original shape after a given load was removed.

$$R_{0.1} = \frac{Sum(\varepsilon_{r0.1Kp},N)}{10} \quad and \quad R_{3.1} = \frac{Sum(\varepsilon_{r3.1Kp},N)}{10} \ldots\ldots\ldots\ldots(3.4)$$

- Non-recoverable creep compliance (Jnr) is a residual strain in the specimen after creep and recovery divided by applied strain. It measures the stress sensitivity of a given aged binder.

$$\text{Jnr(Kpa at 0.1Kp or 3.2Kp)} = \frac{\varepsilon 10}{\text{0.1Kp or 3.2Kp}} \ldots\ldots\ldots\ldots\ldots\ldots(3.5)$$

- Percent elastic recovery difference (%) is the variation of elastic difference in percent between elastic recovery at 0.1kpa and 3.2kpa applied strain.

$$\text{Percent elastic recovery} = \frac{\text{Strain at 1sec} - \text{Strain at 10 sec}}{\text{Strainat 1sec}} * 100 \ldots\ldots\ldots(3.6)$$

- Percent difference in non-recoverable creep compliance (Jnr_{diff})

$$Jnr_{diff} = \frac{(Jnr3.2Kp - Jnr0.1Kp)}{Jnro.1Kp} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.7)$$

C. Conventional Asphalt Binder Physical Tests

Different tests were performed according to AASHTO test Standards to characterize the properties of asphalt binder mixed without and with different percentages of shredded plastic Polyethylene terephthalate a range of 3-9% by the increment of 3% by the weight of the binder. First, the asphalt binder was heated at a temperature of 150^0C then the different percent of shredded plastic was added to the asphalt binder and mixed 45-60min at a temperature 150-160^{0}C to ensure good homogeneity, then the following different tests were performed.

- **Penetration test**

According to AASHTO T 49, a 100g sample of asphalt binder with 3%, 6%, and 9% by weight of sample shredded plastic was heated in an oven for enough time to completely soften and uniformly mixed. Then it was transferred into a penetration test cup and allowed to cool to room temperature for 1 hour. The sample was then placed in water with a temperature controller set to 25^{0}C and allowed to condition for about 1 hour. It was then removed, dried the water quickly, and placed under the needle of the penetrometer. Then three readings were taken for a single penetration cup after placing the tip of the penetrometer needle precisely at the surface of the cup before the instrument was started. The average of the three sample values was recorded.

- **Ductility test**

According to AASHTO T 51, the asphalt binder mixed with 3%, 6%, and 9% by weight of sample shredded plastic was heated and poured in the mold assembly placed on a plate. The samples were cooled in the air at room temperature for about 30min and then put in a water bath at 27℃ temperature. The mold has removed on both sides and hooked on the machine, then assembly contained sample was kept in a water bath of the ductility machine range of 80-90 minutes and the machine was operated. Finally, measured a ductility value, the distance from the point up to break off the thread, and recorded in cm.

- **Softening point test**

According to AASTO T 53, the asphalt binder mixed with 0, 3, 6, and 9% PET shredded plastic by weight of binder was heated and poured into two small brass rings, and allowed to cool. A heated knife blade was used to trim the surface of the samples to the level of the brass rings. The prepared samples were then conditioned in a temperature controller at 4℃ for at least 30 minutes before the test. A steel 3.55gm ball-bearing was centered on each specimen and placed in a glass jar. An electric heater and thermometer were fitted into the beaker filled with clean, distilled water. The temperature at the instant when each of the balls and samples touches the bottom plate of support was recorded as softening point value. The average of two read values was taken and rounded to the nearest whole degree.

Table 3.4 Asphalt cement physical test

No.	Asphalt cement physical test	Test Method
1	Penetration Grade test	AASHTO T 49
2	Ductility test for asphalt	AASHTO T 51-94
3	Softening point test	AASHTO T53-06

Results of the bitumen properties are presented in **Appendix A- Materials Quality Test Result.**

3.8. Marshal Tests

Marshall Mix Design method was used to determine the optimum asphalt content and evaluate the stability, flow, and volumetric properties of the mixtures in the laboratory. The Marshall stability of the mix is defined as a maximum load carried by a compacted specimen at a standard test temperature at 60^0C. The flow value is the deformation at which the Marshall Test specimen undergoes during the loading up to the maximum load, in a 0.25 mm unit.

For this experimental work, three scenarios were conducted. First, the HMA mixture is prepared by a normal mix used stone dust filler. Second, prepared the HMA mixture fully replaced stone dust filler with Coffee husk ash i.e. 3% by weight of the total mixture. Third, prepared a combination of coffee husk ash and shredded plastic on the HMA. It was done by different content of shredded plastic added on the HMA i.e. 3%, 6%, 9%, and 12% by weight of optimum bitumen content. The HMA mixture tested and calculated the stability flow and volumetric properties according to marshal mix design and assessed their properties and check the properties they are suitable as wearing course material.

3.8.1. Marshal Mix Preparations

Before conduct, the laboratory tests for the marshal tests first prepared test specimens for the laboratory tests. It was followed by the main laboratory test procedure to achieve the research objectives. Asphalt, selected aggregate, filler material, and waste PET plastic was used to prepare the specimens.

The first step in conducting the marshal test is the blending of aggregates. Asphalt mixture requires the combining of two or more aggregates, having different gradations, to produce an aggregate blend that meets the gradation specifications for asphalt mix. Available aggregate materials, coarse aggregate (9.5mm-19mm), intermediate aggregate (4.75mm-9.5mm), fine aggregate (0-4.75mm), and filler, were integrated to get the proper gradation within the allowable limits according to ERA specifications using the trial and error method. And also the selected nominal maximum stone size was 19mm because to consider heavy traffic load.

Table 3.5 Particle Size Distribution for wearing Coarse (ERA Manual, 2013)

Sieve Size (mm)	Nominal maximum stone size (19mm) Percentage passing sieve
25	100
19	90-100
9.5	56-80
4.75	35-65
2.36	23-49
0.3	5_19
0.075	2_8
Bitumen Content (%)	4_10

The Blended percentage of each size of aggregates was determined by trial and error method and computed and compared to specification limits. If the calculated graduation is within the allowable limits, no further adjustments need to be made; if not, an adjustment in the proportions must be made and the calculations must be repeated. The trials are continued until the percentage of each size of aggregate is within allowable limits. An optimum aggregate proportion that meets both criteria was then selected. The aggregate blending proportion 23%, 27%, and 50% prepared, for each fractional size of 9.5mm-19mm, 4.75mm-9.5mm, and 0-4.75mm, respectively. Aggregates blending results are presented in Figure 3.6

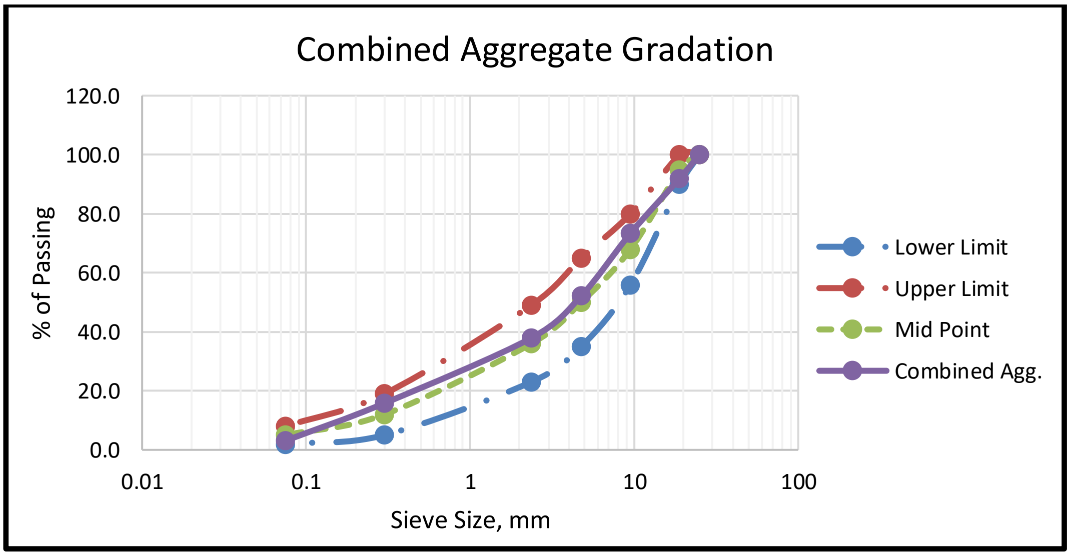


Figure 3.6 Combination of Different aggregate gradation

After this, prepared marshal mix by weight of the mixture, first, calculating the percentage of retained of each size of aggregate in each sieve size then converted into a retained weight on each fraction size. The retained weight on each fractional size in each sieve size is the product of the total weight of the HMA mixture, the percentage composition of each fraction, and the percentage of retained of each size of aggregate. Then finally, calculated Retained Weight on each fraction size by Weight of Mixture. Detailed calculations are presented in **Appendix C – Marshal Mix Test Results.**

3.8.2. Sample Preparation

The mix design was handled by preparing three samples for each bitumen content. For all mixes, five different bitumen contents selected by 0.5% increment. Hence, a total of forty-five Marshall Specimens were prepared and tested for this research. A sample consists of a combined aggregate weighing 1200gm. The aggregate and the asphalt are then mixed with a temperature of 160 – 170^{o}c. The mixed samples were placed into a mold and compacted with 75 blows on each side by a 4.5kg hammer falling from 457mm height.

Form the total 45 samples prepared for the first, second and third scenarios. In the first scenario, each 1200 gm in weight was prepared using five different asphalt contents 3.8%, 4.3%, 4.8%, 5.3%, and 5.8% by 0.5% increment, i.e. three samples for each asphalt content to have an average value. According to the ERA standard protocol (2013) First, the aggregate and binding materials were heated at a temperature of 170-180^{0}C and 150^{0}C, respectively. Then, these ingredients were mixed with a temperature of 140-170^{0}C. The percentage by weight of asphalt content for all mixes taken for the total weight of the mixture. MS-2 Asphalt Institute, (2014) stated that the mixture was placed in the preheated mold and compaction temperature for an asphalt mixture 135-155^{0}C compacted 75 blows on either side of the specimen by a 4.5kg hammer falling from 457mm height. After compaction, the specimen was allowed to cool and removed from the mold using an extrusion jack. Under the Marshal procedure, each compacted test specimen were subjected to the determination of unit weight, void analysis, and stability and flow tests. Then, plotted the graph to determine the values of each respective specimen prepared. The above procedure was repeated for the second scenario the, only difference between them has replaced stone dust filler with coffee husk ash filler material.

After obtaining OBC, the third scenario done with the same procedures. 15 samples were prepared by considering four different proportions of PET shredded plastic 3%, 6%, 9%, and 12% by the weight of OBC by using a dry process method.

3.8.3. Volumetric Analysis

The term “volumetric,” as applied in the asphalt industry, essentially uses measurements of an asphalt mixture by both mass (M) and volume (V) to determine various percentages (P). The volumetric properties of a compacted paving mixture are important criteria by which the quality of an asphalt mixture is evaluated. The volumetric properties are determined using the mass and/or volume measurements of a mixture and its constituent components such as a binder, aggregate, air void. The relationship between mass and volume is determined by the material’s specific gravity (G). Specific gravity is a dimensionless number defined as the ratio of the density of a material to the density of water.

The properties that are to be considered include the theoretical maximum specific gravity Gmm, the bulk specific gravity Gmb, percentage of voids in total mix VTM, percentage volume of bitumen V_b, percentage void in mineral aggregate VMA, percentage voids filled with asphalt VFA, Effective asphalt content P_{be} and film thickness. The volumetric component diagram of HMA is shown in Figure 3.7.

During the design process, various laboratory tests are used to determine the specific gravity of the asphalt mixture and its components. The standard nomenclature for most volumetric properties uses:

- A beginning capital letter to identify the property type;
- Followed by a subscripted lowercase letter identifying the material; and
- Sometimes followed by a second subscripted lowercase letter giving more detail about the nature of the property.

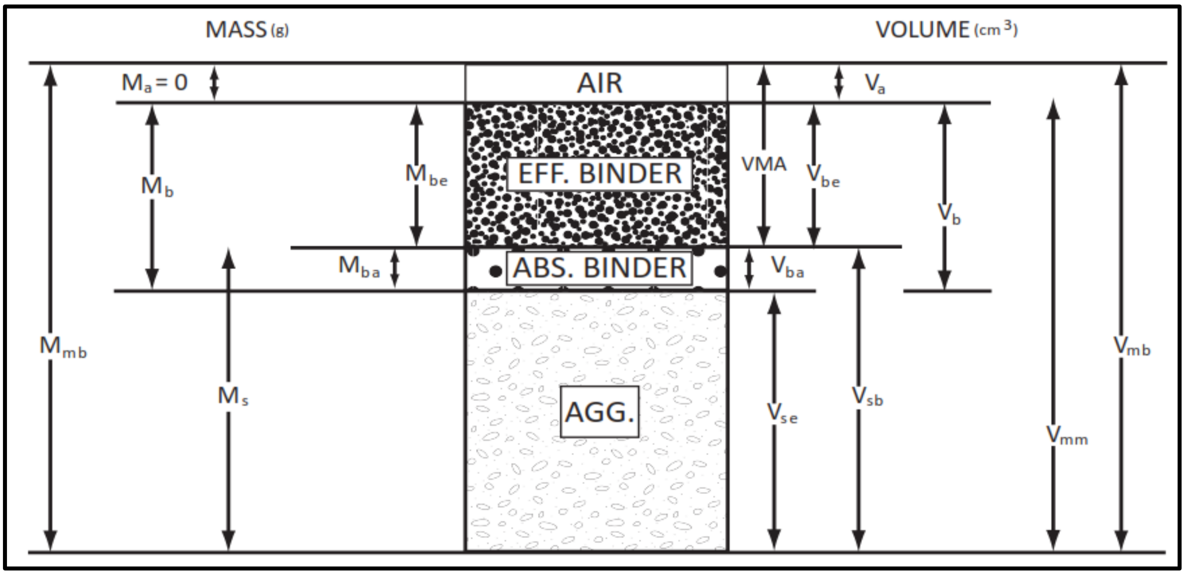


Figure 3.7 Mass and volume relationships in asphalt mixtures (MS-2 Asphalt Institute, 2014)

All the components are defined below

VMA = Volume of voids in mineral aggregate, V_{mb} = Bulk volume of compacted mix V_{mm} = Void less volume of HMA mix, VFA =Volume of voids filled with asphalt, V_a = Volume of air voids, V_{ba} = Volume of absorbed asphalt binder, V_{be} = Volume of effective asphalt binder, V_b = Volume of asphalt binder, V_{sb} = Volume of mineral aggregate (by bulk specific gravity), V_{se} = Volume of mineral aggregate (by effective specific gravity)

M_{mb} = Bulk Mass of a mixture, M_{be} = Mass of effective asphalt binder, M_{ba} = Mass of absorbed asphalt binder, M_s = Mass of aggregate, M_a = Mass of air = 0, M_b = Mass of asphalt binder

The Mixture volumetric parameters include Percentage of voids in the total mix (VTM), Percent void in mineral aggregate (VMA), Percent voids filled with asphalt (VFA), Percent aggregate (P_s), Percent Binder (P_b), Percent Binder Effective, (P_{be}), Percent Binder Absorbed (P_{ba}) (ERA Manual, 2013).

- **Percentage of voids in the total mix (VIM)**

The volume of air voids in a compacted mixture is the total volume of air void throughout a compacted paving mixture, expressed as a percent of the bulk volume of the compacted paving mixture. It can be determined by using the equation below:

$$VIM = 100 * \left[\frac{G_{mm} - G_{mb}}{G_{mm}}\right] \text{…………..…} (3.8)$$

Where, V_a = Air voids in the compacted mixture, percent of total volume, G_{mm} = maximum specific gravity of paving mixture, G_{mb} = bulk specific gravity of the compacted mixture

- **Percent void in mineral aggregate (VMA)**

VMA is the volume of intergranular void space between aggregate particles of the compacted paving mixture. It includes air void and volume of asphalt not observed into the aggregate and expressed as a percent of the total volume.

$$VMA = 100 - \left[\frac{G_{mb} * P_s}{G_{sb}}\right] \text{……….. (3.9)} \quad \text{used for percent by wt. of the total mix}$$

$$VMA = 100 - \left[\frac{G_{mb}}{G_{sb}} * \frac{100}{100 + P_b} * 100\right] \text{……. (3.10)} \quad \text{used for percent by wt. of aggregate}$$

Where, VMA = voids in the mineral aggregate, G_{sb}= bulk specific gravity of total aggregate, G_{mb} = bulk specific gravity of the compacted mixture, P_s = aggregate content, percent by mass of total mixture, and P_b = percent of a binder.

- **Percent voids filled with asphalt (VFA)**

The voids filled with asphalt, VFB, is the percentage of VMA that is filled with bitumen. It is calculated using:

$$VFA = 100 * \left[\frac{VMA-VIM}{VMA}\right] \text{…………..… (3.10)}$$

Where: VFB = voids filled with bitumen (percent of VMA) VMA = voids in mineral aggregate, percent of bulk volume VIM = air voids in the compacted mix, percent of total volume.

- **Percent aggregate (P_s)**

The total percentage of aggregate in the asphalt mixture expressed as a percentage of the total mix mass

$$P_s = 100 - 100 * P_b \text{………………… (3.11)}$$

Where Ps = percentage of aggregate and Pb = percentage of bitumen content by total mix.

- **Percent Binder Effective, (P_{be})**

The functional portion of the asphalt binder that stays on the outside of the aggregate and is not absorbed into the aggregate, expressed as a percentage of the total mix mass.

$$P_{be} = P_b - \left[\frac{P_{ba}}{100}\right] * P_s \text{ ………………… (3.12)}$$

Where, P_{be} = Effective binder content of a paving mixture, P_b = percent of bitumen content by total mixture, P_{ba} = percent of binder absorbed, and P_s = percent aggregate by total mix weight.

- **Percent Binder Absorbed (Pba)**

The portion of the asphalt binder that is absorbed into the aggregate, expressed as a percentage of the total aggregate. It is calculated by:

$$P_{ba} = 100 * G_b \left[\frac{G_{se}-G_{sb}}{G_{se}*G_{sb}}\right] \quad (3.13)$$

Where, P_{ba} = Binder absorption, G_b = Specific gravity of bitumen, G_{se} = Effective specific gravity of aggregate, and G_{sb} = Bulk specific gravity of aggregate. All Detail calculations and formulas for marshal tests are presented in **Appendix C.**

3.8.4. Optimum Bitumen Content Determination

The Marshall test used to determine the optimum binder content. Five different percentages of asphalt were examined to determine the optimum percentage of asphalt for the aggregates used, which include 3.8%, 4.3%, 4.8%, 5.3%, and 5.8% by weight of the mix with three samples for each percentage. Finally, the ERA, (2013) flexible pavement design manual used to determine the OBC by determining asphalt content corresponding to 4% air void and fix the value of stability, density, flow, VMA, and VFA. ERA specification 2013 manual generally expressed the percent of air void in the mixture is in the range of 3-5%.

To summarize the total number of a sample taken in this research shown in *Table 3.6*

Table 3.6 Total number of sample used in this research

Type of Test	Sample Type (%PET content)	Sample Condition (BRTFO & ARTFO)	Test Replicate or Sample	Temperature replicates	No of Test
Penetration	4	2	3	-	24
Ductility	4	2	3	-	24
Softening Point	4	2	2	-	16
Total No of Conventional Test sample =					64
PG determination	4	2	2	3	48
AST at 21.1, 37.8, & 54.4oC	4	2	2	3	48
FST at 21.1, 37.8, & 54.4oC	4	2	2	3	48
MSCR at 58, 64, & 70oC	4	1	2	3	24
Total No of Superpave physical Test sample =					168
Marshal Test for SD Filer	1	-	15	-	15
Marshal Test for CHA Filer	1	-	15	-	15
Marshal Test for PET+CHA Filer	5	-	3	-	15
Total No of Marshal Test sample =					45
Total No of Test Sample in this Research =					**277**

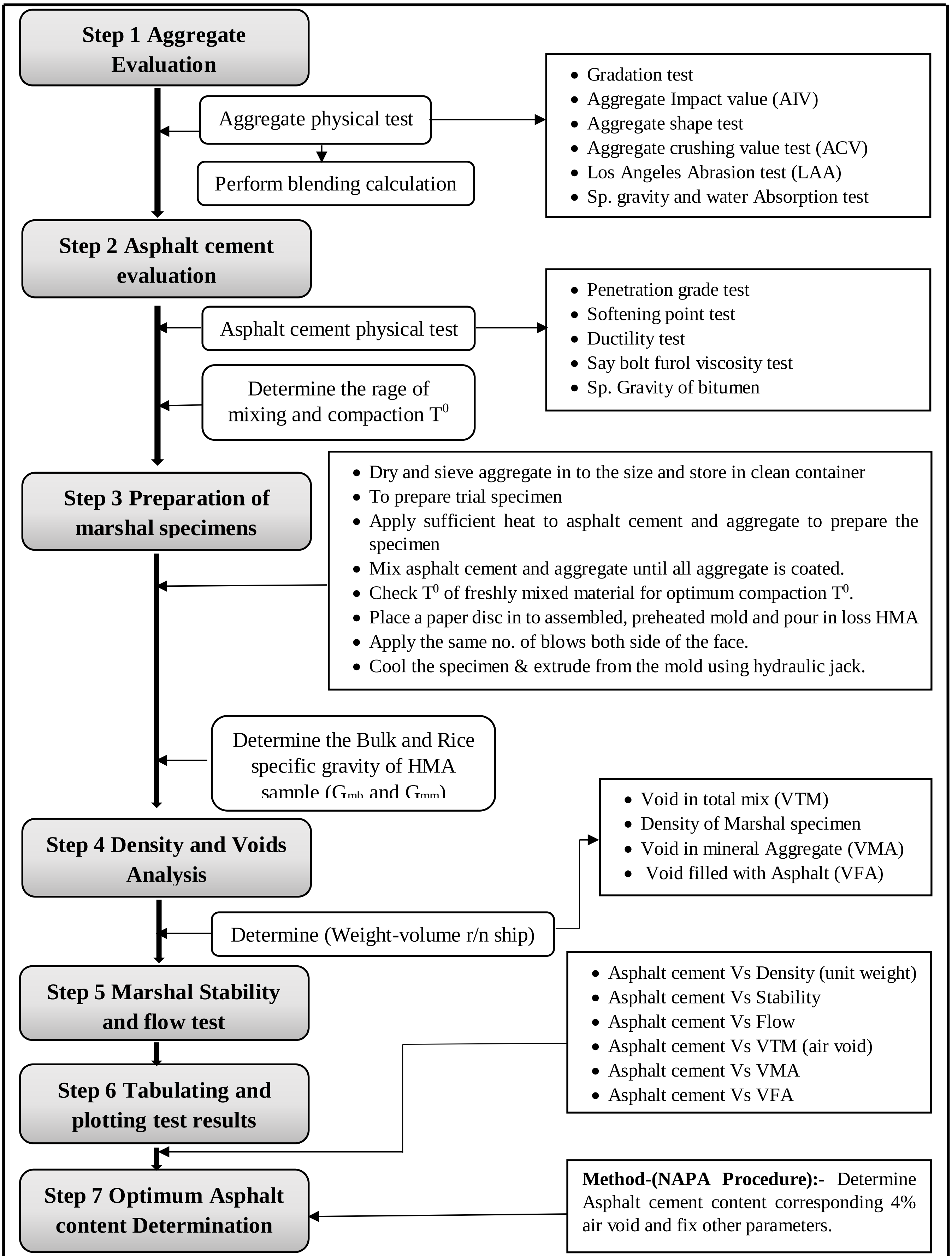


Figure 3.8 General Procedure of marshal mix design

3.9. Research Variable

In this research, the variables were categorized into two. These are the dependent and independent variables.

A. Dependent Variable

The dependent variable used:-

- Rutting, Linear Viscoelastic, Top-Down and Bottom-Up cracking

B. Independent Variable

The independent variable used:-

- Rheological properties of asphalt binder (Shear Modulus and Phase Angle)
- Durability (RTFO).
- Time-Temperature and Loading condition

3.10. 3D- Move Analysis Software

Before evaluation the performance of pavement using this software first select the route and collect the available data from the recommended office. The selected route was conducted in Hawassa City, by selecting the road stretch from Tikur wuha (Bishan Guracha) to Hawassa University Institute of Technology that covers a length of approximately 5.7 km and is located about 275 km from Addis Ababa along the road connecting Ethiopia and Kenya. It is found in Sidama Region on the eastern shore of Lake Hawassa. It is approximately located between 7°02'51" N Latitude and 38°29'44" E Longitude, and at an average elevation of 1716m above mean sea level.

Source: (Google Earth Map, Access date: April 21, 2021 G.C)

Figure 3.9 Snapshot of location map from Tkur wuha to Hawassa University Institute of Technology

Finally, a typical route selected from tikur wuha to hawassa institute technology along from Addis Ababa-Nairobi-Mombasa road corridor. A typical 4-layer flexible pavement structure was used in the analysis, consisting of a 5cm AC layer, 25cm base course, 30cm Sub-base, and subgrade. The design load considered a standard dual tire single axle (Load case-2), Dual tire tandem axle, super single tire tandem axle load, and a Special non-highway Vehicle (Caterpiller) which consists of a Single axle-single tire in the front axle and tandem axle-single tire in the rear axle with 3 million repetitions during the design period of 20 years. Then, changes were made to the asphalt concrete layer material properties depending on the lab test results (E* and Phase angle). 3D Move Analysis software allows user to incorporate $|E|$ and $|G|$ test results into the analysis as Level 1inputs. Other parameters in the base course, sub-base and subgrade layer are used constantly for all condition.

The major Step-by-Step Guide to 3D-Move Analysis steps that followed below:

Step 1: Open 3D-Move Analysis software

Step 2: Getting Started with a New Project- Once a new project is created, define the unit system to be used with the problem under consideration. The 3D-Move Analysis program is capable of analyzing a problem in any one of the two different unit systems. They are US Customary Units (in, lb, °F, mph); and SI Units (m, kN, °C, km/h).

Step 3: Site/ Project Identification

Step 4: To define the type of analysis, click either Static or Dynamic Analysis in the main window

Step 5: To specify the vehicle loading condition by click Axle Configuration and Contact Pressure Distribution in the main window.

Step 6: Dynamic Loading Coefficient (DLC)- Click Vehicle Suspension and Road Roughness in the main window to specify a DLC.

Step 7: Pavement Structure- Details on Pavement Structures can be input by clicking Pavement Structure in the main window.

Step 8: Defining Pavement Layer Properties- The material properties can be entered by clicking each layer displayed under the Pavement Layer Properties.

Step 9: Response Points -To define the response points, click the Response Points in the main window

Step 10: Run Analysis -Since all inputs are provided, Click Run Analysis to perform the response analysis.

Step 11: Input Summary – Microsoft Excel Format. 3D-Move Analysis creates comprehensive input and outputs summary files in Microsoft Excel Format after completing the execution. Input summary Excel file can be accessed by clicking titles under Input Summary in the main window.

Step 12: Output Summary – For dynamic analysis, the output can be accessed in three different ways (Text Format, Microsoft Excel Format, and Graphical format).

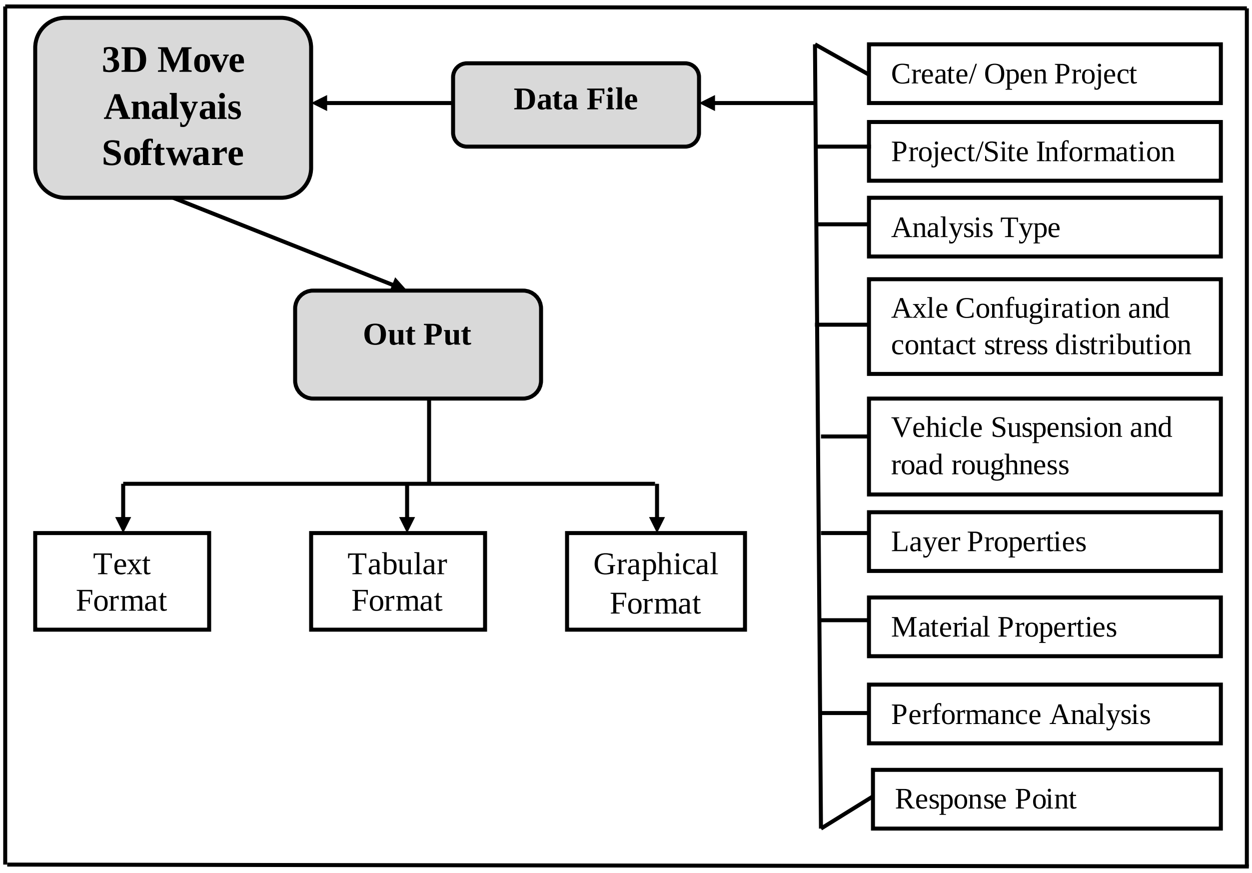


Figure 3.10 General Procedure of 3D-Move Analysis Software

4. RESULTS AND DISCUSSIONS

This chapter presents the test result of the asphalt binder property that changes when the addition of Polyethylene terephthalate (PET) plastic and used coffee husk ash as a filler in the hot mix asphalt properties then compared and discussed the result with other different researches made on. Then the results of the test are used to conclude the rheological properties and performance of both PET and coffee husk ash materials and their ability to meet specifications.

4.1. The effect of PET plastic on The Rheological Property of asphalt Binder

4.1.1. The Effect of PET plastic on the Amplitude Sweep Test (AST)

Test results from the DSR amplitude sweep test are presented graphically with the Log-Log scale of complex shear modulus (G*) in the Y-axis and Strain (%) in the X-axis below. The main reason for conducting the AST is to determine the linear viscoelastic range (LVER). The limit of LVER is determined as the point beyond which the complex modulus decreases to 95% of the measured value at zero-strain (Agirre-Olabide et al., 2014). A typical LVER is determined in *Figure 4.1* for 0% PET plastic before rolling thin film oven test (BRTFO) at 21.1^{o}C.

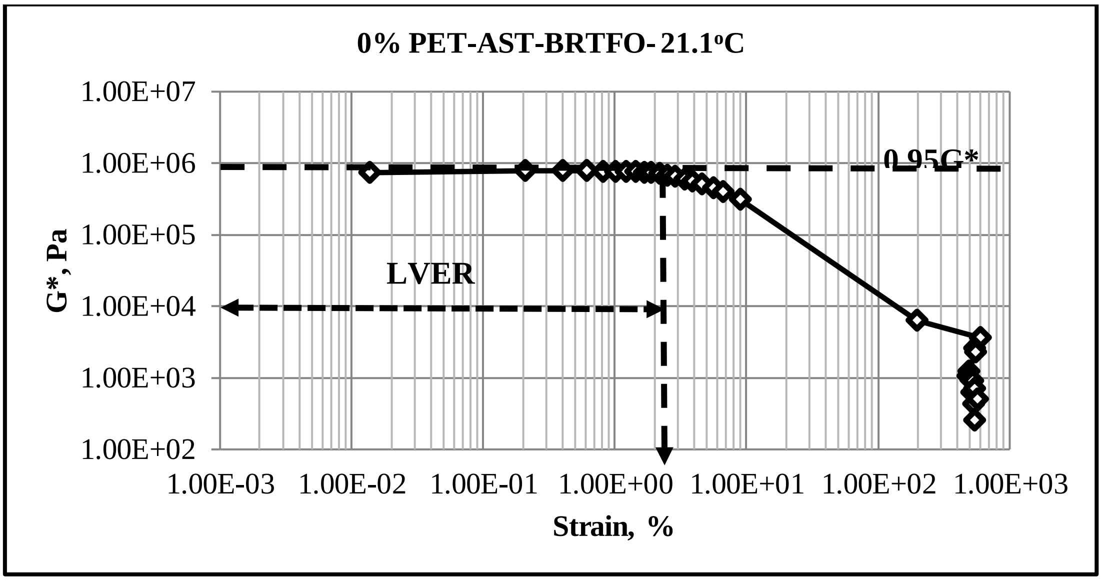


Figure 4.1 A typical LVE range for 0% PET plastic BRTFO at 21.1^{o}C

The addition of a different percentage of PET plastic influenced the rheological property of asphalt binder. As it can be seen from the figure below PET plastic influencing the linear visco-elastic Range (LVER) in asphalt binder. Which is an increase as compared with the original asphalt binder.

Table 4.1 Visco-elastic region for unaged and aged binder mixes

Binder Type	Temperature oC	Before RTFO			After RTFO		
		G*	0.95G*	Limiting Strain, %	G*	0.95G*	Limiting Strain, %
0% PET	21.1	7.05E+05	6.70E+05	2.49	1.68E+06	1.60E+06	2.38
	37.8	4.21E+04	4.00E+04	23.0	4.21E+04	4.00E+04	16.04
	54.4	2.89E+03	2.74E+03	64.7	5.01E+03	4.76E+03	70.47
3% PET	21.1	1.79E+06	1.70E+06	6.00	3.31E+06	3.14E+06	5.16
	37.8	2.66E+06	2.53E+06	33.3	4.00E+06	3.80E+06	18.35
	54.4	3.88E+03	3.69E+03	83.8	1.47E+05	1.40E+05	42.14
6% PET	21.1	2.64E+06	2.51E+06	7.5	3.86E+06	3.67E+06	6.23
	37.8	1.04E+05	9.88E+04	50.2	2.78E+05	2.64E+05	55.37
	54.4	9.68E+03	9.20E+03	87.1	9.40E+03	8.93E+03	86.12
9% PET	21.1	7.64E+07	7.26E+07	4.12	1.11E+07	1.05E+07	5.75
	37.8	2.71E+08	2.57E+08	34.5	7.15E+06	6.79E+06	35.77
	54.4	2.78E+08	2.64E+08	73.5	1.37E+04	1.30E+04	79.14

Based on *Table 4.1* it is observed that the amplitude sweep test enables to recognition of the change in stiffness and LVE range due to the three factors these are temperature, the content of PET plastic, and ageing. At 21.1°C temperature the limiting strain value for each sample is minimum and the effect of the modifier is not significant but at 54.4°C temperature, higher limiting strain values are observed as the material gets less stiff and also it is possible to say there is considerable change in limiting strain values before and after RTFO ageing. The linear viscoelastic region of the pure bitumen was less than the linear viscoelastic region of the modified binders as shown in all Figures. This shows that the sample structure of the modified binder is undisturbed on high strain or deformation than the pure bitumens internal structure. The structures of bitumen samples, pure or modified, are disturbed at the end of the LVE region. The LVE region of pure bitumen had disturbed before the modified bitumen at all temperatures. This has shown that the modifiers had successfully modified and increased the pure bitumen linearity during deformation.

The test results were indicated that the PET plastic modifiers and temperature have affected the stiffness and strain of asphalt binder. *Figure 4.3* and *Figure 4.4* show that the complex modulus of pure and modified binders have decreased as the temperature increase and vice versa this shown that the stiffness of a binder has affected by temperature. And also increase G* values with the increase in the percentage of PET plastic. This because PET plastic changes the rheological properties of the binder by increase the total resistance of the binder to deformation when repeatedly sheared.

As observed from *Figure 4.2*, a long LVE region has scored at a binder modified at 6% PET plastic modifier. This means that at 6% PET plastic content was a better performance than the other modifiers according to the LVE region. Detailed AST results are presented in **Appendix B**.

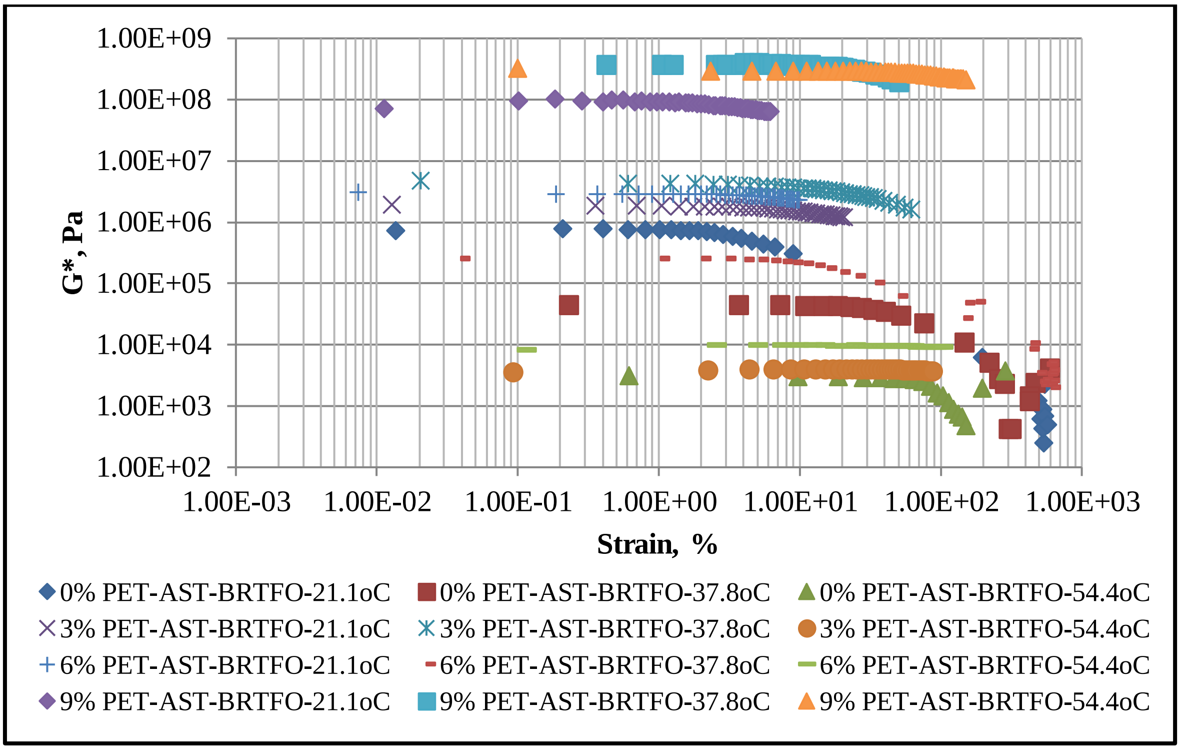


Figure 4.2 The effect of PET Plastic on the AST-BRTFO Asphalt binder

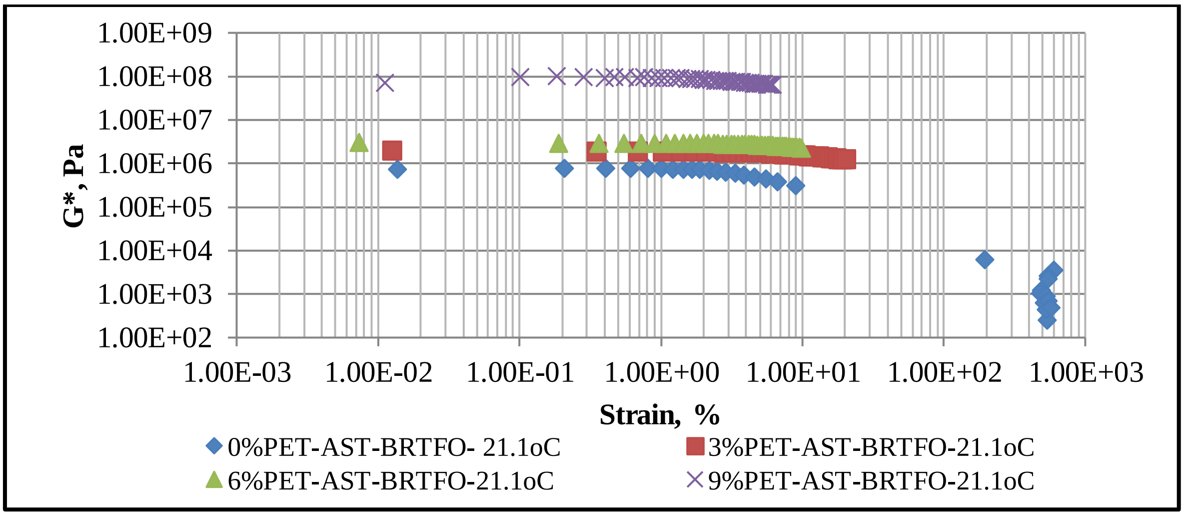


Figure 4.3 The effect of PET Plastic on the BRTFO Asphalt binder at 21.1°C

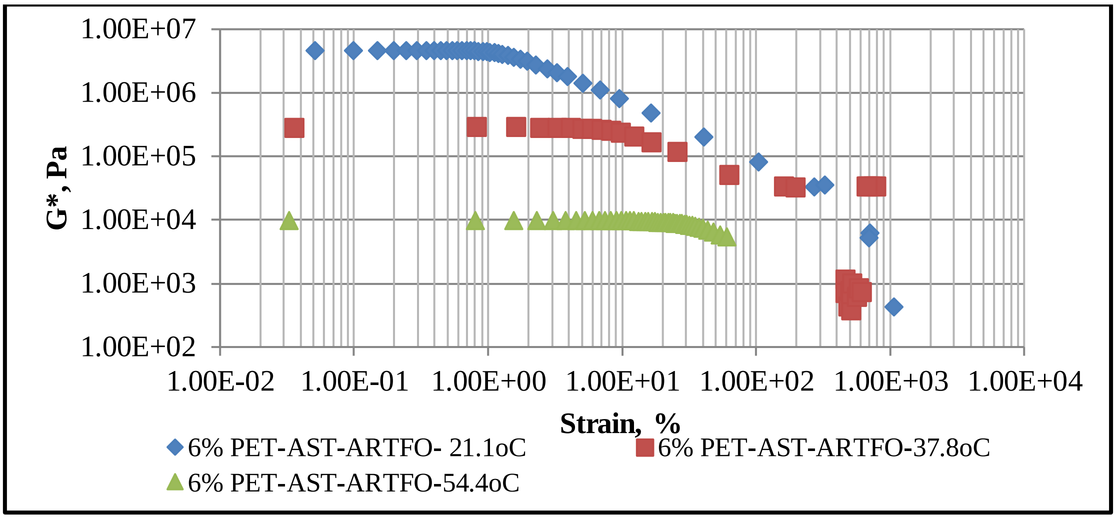


Figure 4.4 A typical Complex Modulus versus strain for 6% PET plastic ARTFO

Furthermore, the linear viscoelastic region of unaged bitumen was greater than that of aged bitumen. The effect of ageing bitumen on the LVE region is shown in *Table 4.1*. it shown that the LVE region of unaged pure bitumen has changed from 2.47 and 23.0% to 2.38 and 16.0% after RTFO aged at 21.1 and 37.8℃, respectively. This shows the structure disturbance of RTFO aged pure bitumen was reached before the RTFO aged modified bitumen shown in *Figure 4.5*.

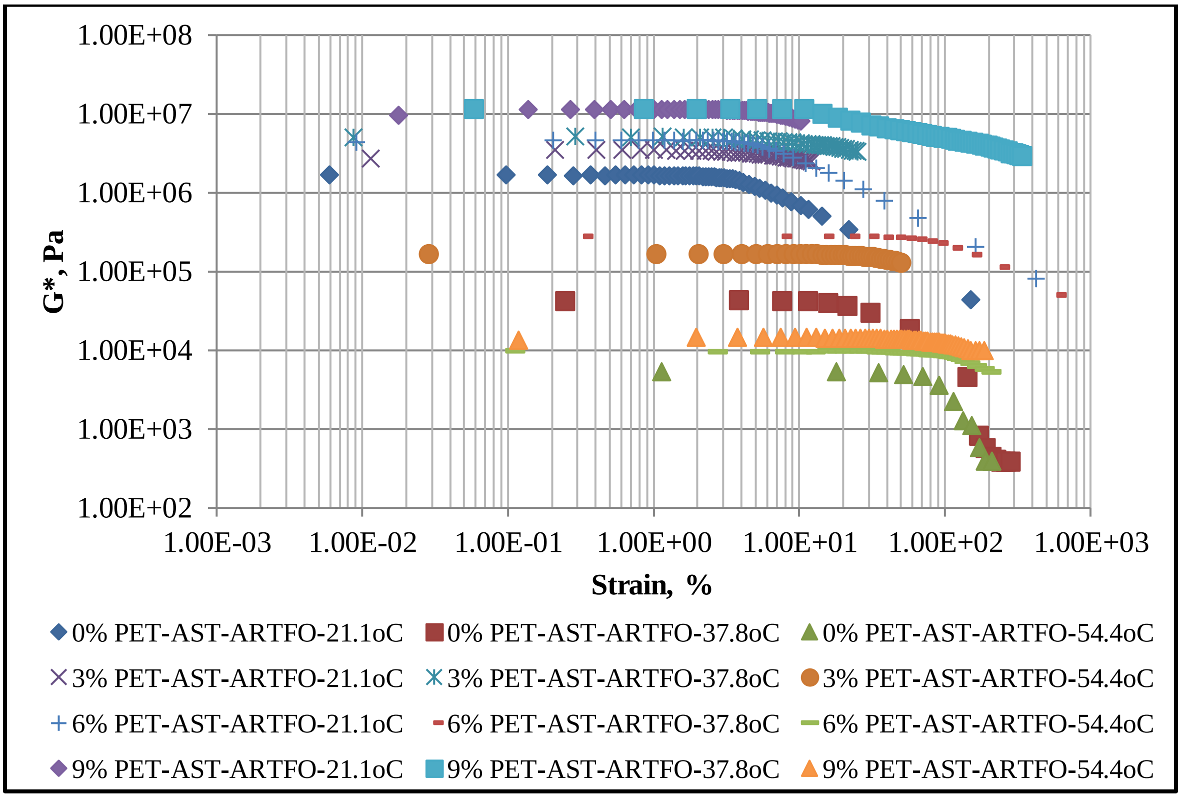


Figure 4.5 The effect of PET Plastic on the AST-ARTFO Asphalt binder

4.1.2. The Effect of PET Plastic on the Performance Grade (PG) Determination Test

To grade an unknown binder, begin DSR testing at 52˚C. Determine the results of G*/sin δ. If the sample fails at 52˚C, increase or decrease the temperature by increments of 6˚C until the value of G*/sin δ ≥ 1.00 kPa for unaged binder and G*/sin δ ≥ 2.2 kPa for aged binder. The highest temperature where G*/sin δ ≥ 1.00 kPa will determine the starting value for the PG grade. The high-temperature test result includes relevant parameters for the determination of the performance grade for both unmodified and modified binders at high temperature were organized in *Table 4.2* before and after RTFO respectively.

Table 4.2 PG Determination Test Result for Original and RTFO Residue Binder

Binder Type	Aging Status	Temp. ºC	G*, Kpa	δ, º	G*/Sin(δ) at 1.59Hz, Kpa	Criteria G*/Sin(δ) at 1.59Hz, Kpa	PG	PG at Final High Temperature
0% PET plastic	Unaged	52.0	3.10E+03	85.51	3.12	≥ 1	PG58-xx	PG58-xx
		58.1	1.23E+03	86.67	1.24			
		64.1	5.54E+02	87.46	0.56			
	Aged	52.0	6.62E+03	83.07	6.70	≥ 2.2	PG58-xx	
		58.0	2.77E+03	85.25	2.79			
		63.99	1.13E+03	86.71	1.14			
3% PET plastic	Unaged	58.08	2.52E+03	85.54	2.54	≥ 1	PG58-xx	PG58-xx
		64.06	1.08E+03	86.81	0.79			
	Aged	58.04	1.46E+04	73.16	15.42	≥ 2.2	PG70-xx	
		64.04	6.52E+03	76.75	6.76			
		70.04	3.82E+03	79.19	3.92			
		75.99	1.60E+03	82.23	1.62			
6% PET plastic	Unaged	58.05	3.94E+03	81.58	4.01	≥ 1	PG64-xx	PG64-xx
		64.10	1.71E+03	83.88	1.73			
		68.88	8.42E+02	85.42	0.85			
		75.14	8.22E+02	85.49	0.83			
	Aged	58.03	1.60E+04	72.54	16.94	≥ 2.2	PG70-xx	
		64.11	1.07E+04	74.61	11.19			
		70.12	4.55E+03	78.39	4.68			
		76.02	2.09E+03	81.18	2.13			
9% PET plastic	Unaged	58.11	1.72E+04	71.32	18.37	≥ 1	PG76-xx	PG70-xx
		64.03	8.00E+03	75.04	8.36			
		70.14	3.38E+03	78.69	3.47			
		76.01	1.60E+03	81.59	1.63			
	Aged	58.11	1.72E+04	71.32	18.37	≥ 2.2	PG70-xx	
		64.03	8.00E+03	75.04	8.36			
		70.14	3.38E+03	78.69	3.47			
		76.01	1.60E+03	81.59	1.63			

Table 4.2 shown that the PG of the original binder has no variation at 3% but improved from PG 58 to PG 64 and PG 70 when the bitumen is modified at 6 and 9% PET plastic. This means that the content of PET plastic has increased, the PG also increased. The increment in PG clearly shows that modifying bitumen with PET plastic material increases the stiffness of the asphalt binder. Here it is understood that the basic rheological parameter, complex shear modulus (G*) is increasing as the percentage content of PET increases and decreased the phase angle. The increase in complex shear modulus values indicates that the bitumen added additional shear strength with the addition of PET.

Figure 4.6 shown the shear modulus (G*/sin(δ)) versus temperature graphs obtained for modified by PET plastic. The lower the complex shear modulus (G*) value of the bitumen means the lower the stiffness value and easily deformed. The phase angle is also an indicator of viscous deformation of bitumen relative to elastic deformation, the phase angle values of the modified binder decreased compared to the pure bitumen. This indicates that when the phase angle decreased, the viscous deformation decreased and the elasticity of the bitumen getting increased. As the values of complex shear modulus (G*) increased and the values of the phase angle (δ) decreased, the value of the rutting parameter, G*/sin(δ) increased. The higher this value is, the greater the resistance of the modified binder to rutting at high temperatures. Based on this it is possible to improve the rutting performance due to the modifier.

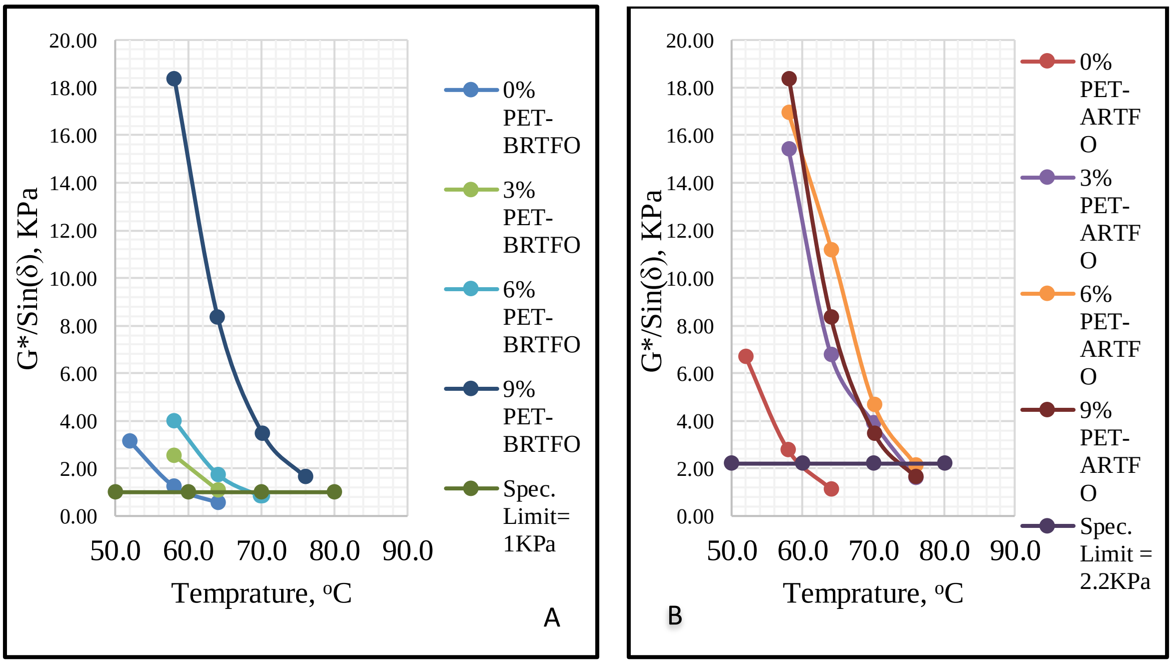


Figure 4.6 (a) & (b) G*/sin(δ) Vs Temperature for BRTFO and ARTFO bitumen samples

4.1.3. The Effect of PET Plastic on the Frequency Sweep Test (FST)

The sweep or variation in frequency was set from high to low (25Hz-0.1Hz) in an increasingly damaging effect. Frequency sweep test results at 21.1°C, 37.8°C, and 54.4°C for all samples both aged and unaged were determined. The Figures below present the DSR frequency sweep test results in complex modulus Vs Frequency and phase angle Vs Frequency for a typical PET plastic percentage.

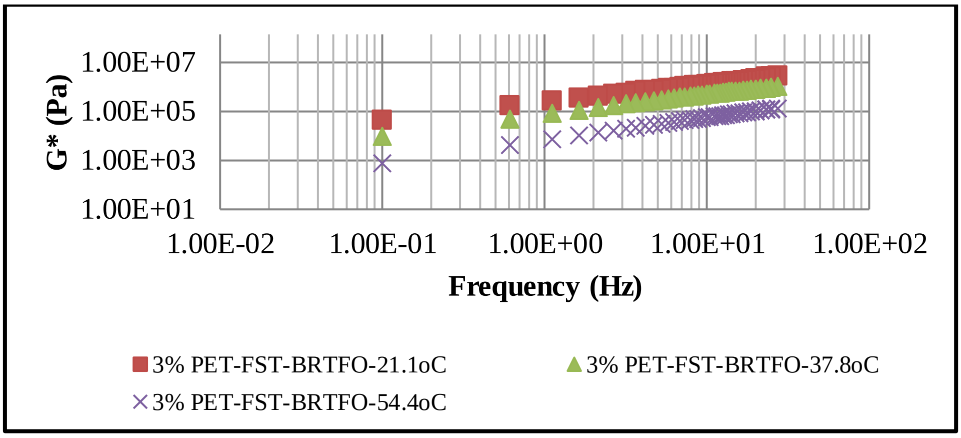


Figure 4.7 Complex modulus versus frequency for unaged binder at 3% PET

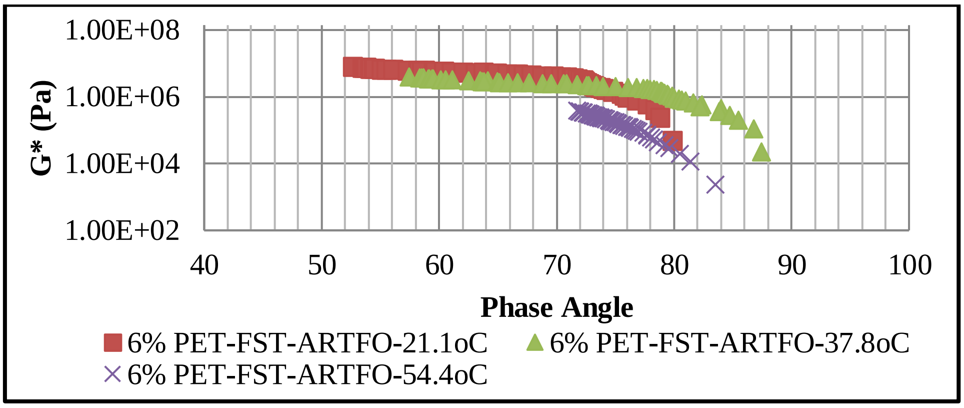


Figure 4.8 Complex modulus versus Phase Angle for Aged binder at 6% PET

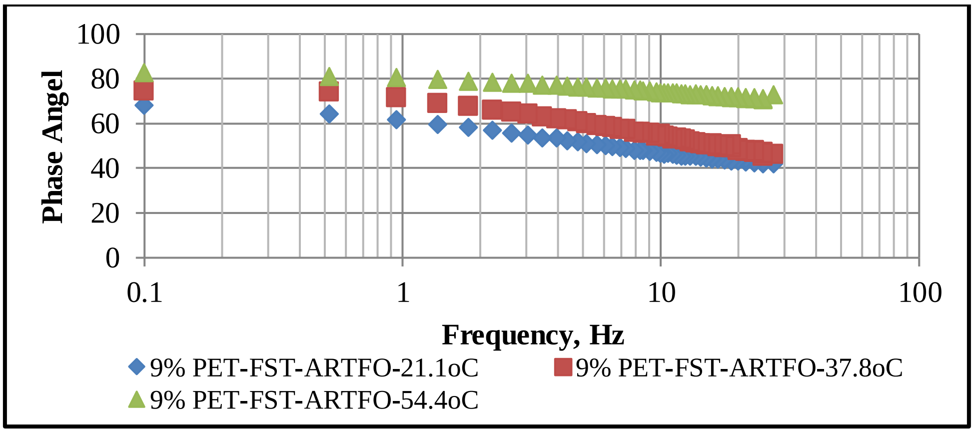


Figure 4.9 Phase Angle versus Frequency for Aged binder at 9% PET

Figure 4.7, *Figure 4.8*, and *Figure 4.9* indicate that the relation between complex modulus Frequency and phase angle. The test result shows that complex modulus values have increased with the increase in frequency, while decreased with the increase of the temperature is based on the fact that the material is in the plastic region at low frequencies (high values for phase angle). And the phase angle values have decreased with the increase in frequency, while increased with the increase of the temperature. The higher the complex modulus value and low phase angle are, the stiffer the asphalt binder and thus more resistance to rutting and the lower the δ value, the more elastic the asphalt binder by increase G*/Sinδ, a parameter for rutting resistance. So, it is more resistant to permanent deformation. More figures related to FST results are presented in **Appendix B.**

4.1.3.1. Master Curves of Complex Shear Modulus

Modulus Master Curves were developed using Microsoft excel solver to best fit the sigmoid function for all asphalt binders. The sigmoid function to best fit the obtained G* data from the DSR is carried out by changing the sigmoid constants and the shift factor.

Accordingly, the DSR data from the three test temperatures (21.1°, 37.8°, and 54.4°C) were used to construct the master curves for asphalt binder and asphalt binder containing PET plastic both aged and unaged, and the following shift factors have been developed to construct the master curves for complex modulus.

Table 4.3 Shear Modulus Sigmoid Coefficients and Temperature Shift Factors

Ageing condition	PET content (%)	Alpha (α)	Beta (β)	Delta (Δ)	Gamma (γ)	$T_{21.1}$	$T_{37.8}$	$T_{54.4}$
BRTFO	0	34.08	-0.96	-19.48	0.17	0.00	-0.93	-1.60
	3	51.00	-3.53	-44.08	0.49	0.00	-1.82	-3.12
	6	64.03	-3.10	-56.31	0.37	0.00	-1.82	-3.12
	9	45.79	-3.10	-37.19	0.49	0.00	-1.82	-3.12
ARTFO	0	38.71	-2.74	-30.63	0.34	0.00	-1.19	-2.25
	3	41.12	-0.82	-23.28	0.11	0.00	-1.82	-3.12
	6	68.08	-3.01	-59.20	0.31	0.00	-1.82	-3.12
	9	44.69	-3.92	-36.10	0.68	0.00	-1.82	-3.12

Table 4.3 shows the parameter α is defined as the minimum stress level that would cause the damage; Δ+α is defined as the maximum stress that would cause instantaneous damage, and the ß and γ are described as the shape of the sigmoidal function. All of these values vary for each binder type. As for the temperature shift factors, a is zero for all the binder types because all the parameters are shifted to 21.1°C. Whereas for 37.8 and 54.5^{0}C the values are

all negative because the stiffness parameters are shifted to a reduced temperature which is 21.1ºC. *Figure 4.10* and *Figure 4.11* below present the complex modulus master curves for aged and unaged binders.

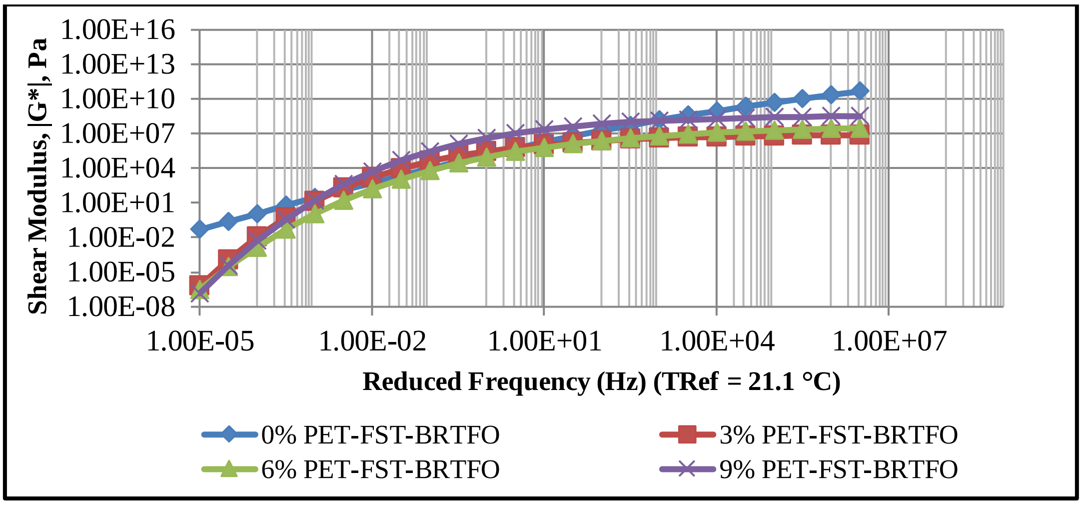


Figure 4.10 Shear Modulus Master Curve before RTFO

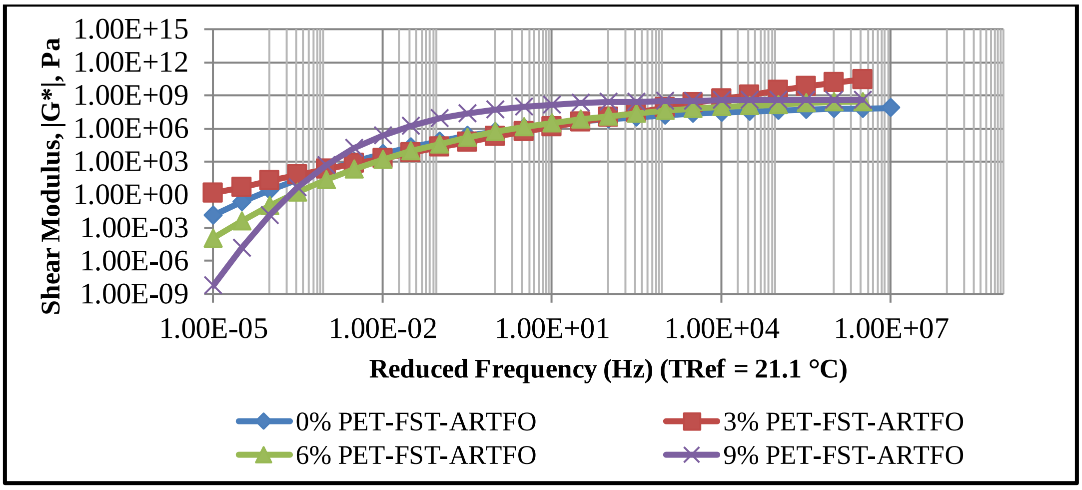


Figure 4.11 Shear Modulus Master Curve after RTFO

Figure 4.10 and *Figure 4.11* show the main rheological parameter, complex shear modulus (G*) behaves as a result of temperature, frequency related to loading rate (Speed), the content of modifier, and ageing. For all binders are almost similar pattern shear stiffness decreases as temperature increases. At high temperature and low frequency, the smaller value of shear modulus occurs but at a lower frequency and higher temperature, the modulus increases as the modifier increases. This exhibit the PET plastic modifier improves the complex shear modulus of the virgin binder at higher temperatures. This improves the rutting problem at high temperatures due to slow-moving traffic and pavement performance against this distress by increasing the stiffness of the binder.

4.1.3.2. Phase Angle Master Curve

A relatively similar procedure followed for phase angle master curve like modulus master curves and with a slightly different form of the generalized logistic function, the binder phase angle master curves were developed. Frequency sweep data were also used for the construction of phase angle Master curves. Phase angle Master curves are shown below.

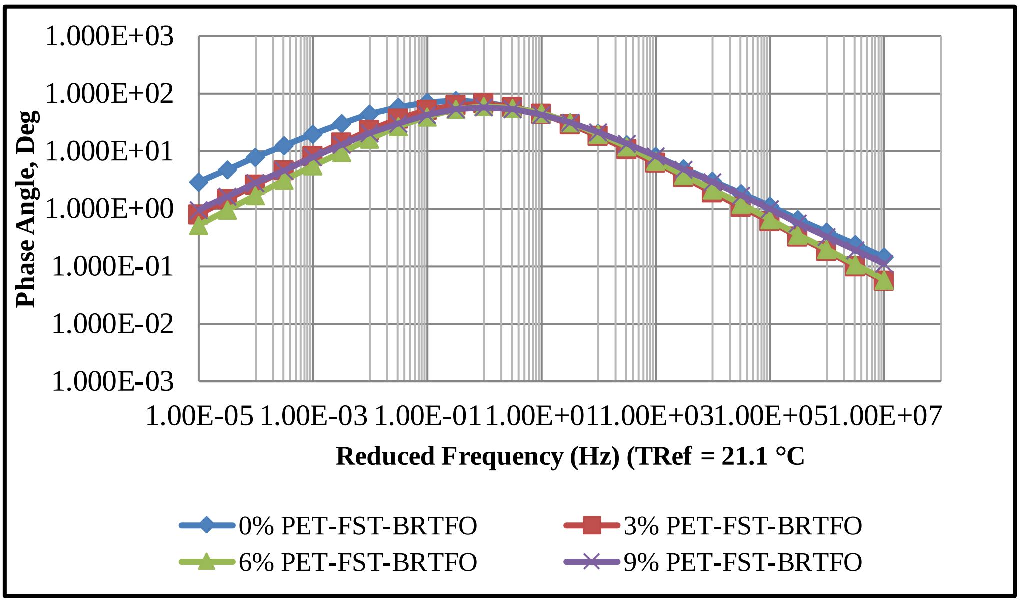


Figure 4.12 Phase Angle Master Curve before RTFO

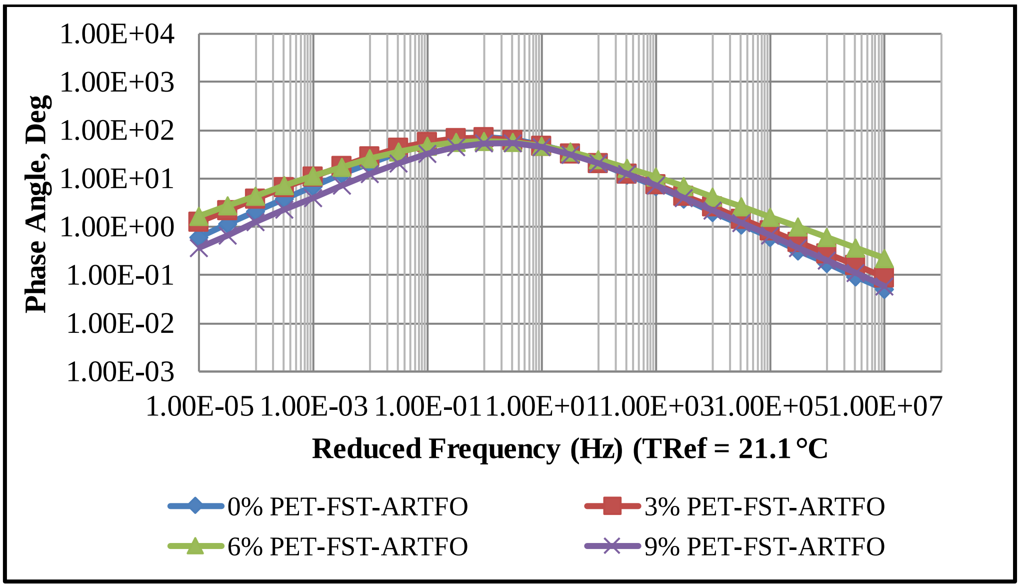


Figure 4.13 Phase Angle Master Curve after RTFO

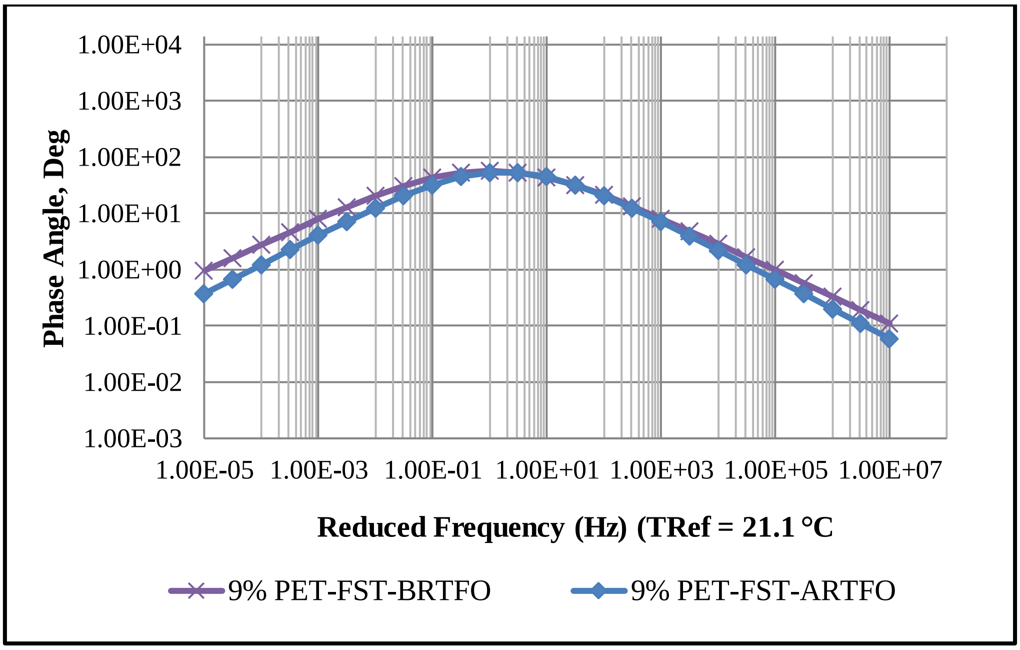


Figure 4.14 The effect of PET on the phase angle Master Curve ARTFO

Figure 4.12 and *Figure 4.13* exhibit that the decrease in phase angle at high temperature and low frequency as the content of the PET plastic increases compared with the virgin binder. This indicates an increase in elastic modulus when the phase angle decreased. And also the *Figure 4.14* show that compared phase angle master curves before and after RTFO ageing, it is possible to say that the virgin bitumen is more sensitive for aging than modified binders this indicated that PET plastic changes the rheological properties by decrease the phase angle with increase the shear modules of material and resist the higher temperature rutting deformation. Additional FST related results are presented in **Appendix B.**

4.1.4. The Effect of PET Plastic on the Multiple Stress Creep and Recovery Test (MSCR)

The MSCR test result (software output) contains huge data to represent in tables here. Therefore, the test result is organized graphically as shown in *Figure 4.15* and *Figure 4.16*.

Figure 4.15 below show that the total strain was influenced by the addition of PET plastic. As the PET plastic percentage increases the total strain value decreases showing improving behaviour. Detailed MSCR test result presented in **Appendix B.**

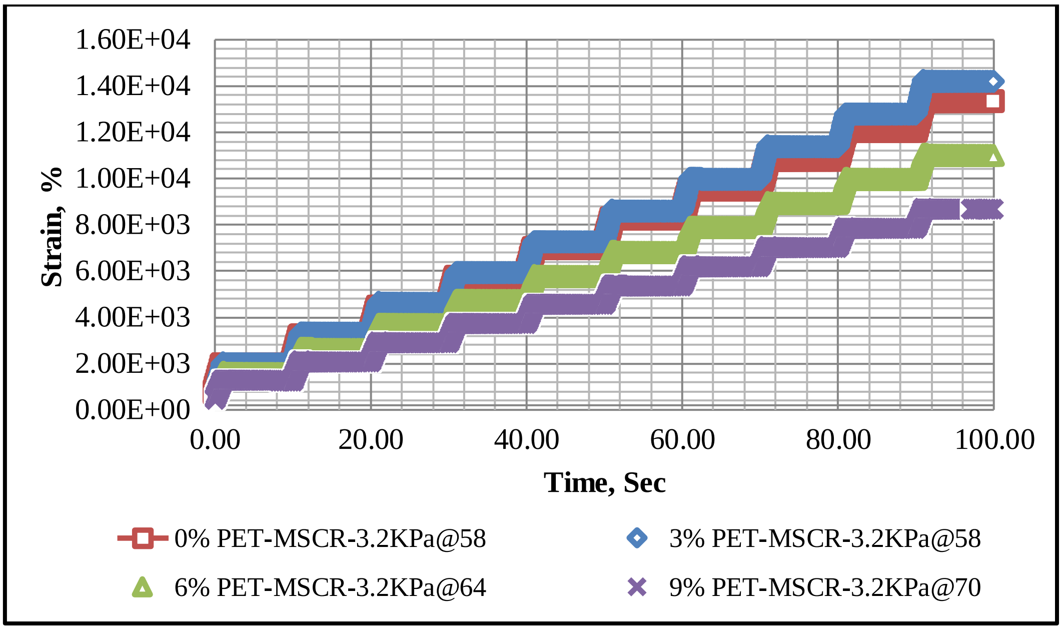


Figure 4.15 The Effect of PET plastic on the strain value at 3.2KPa

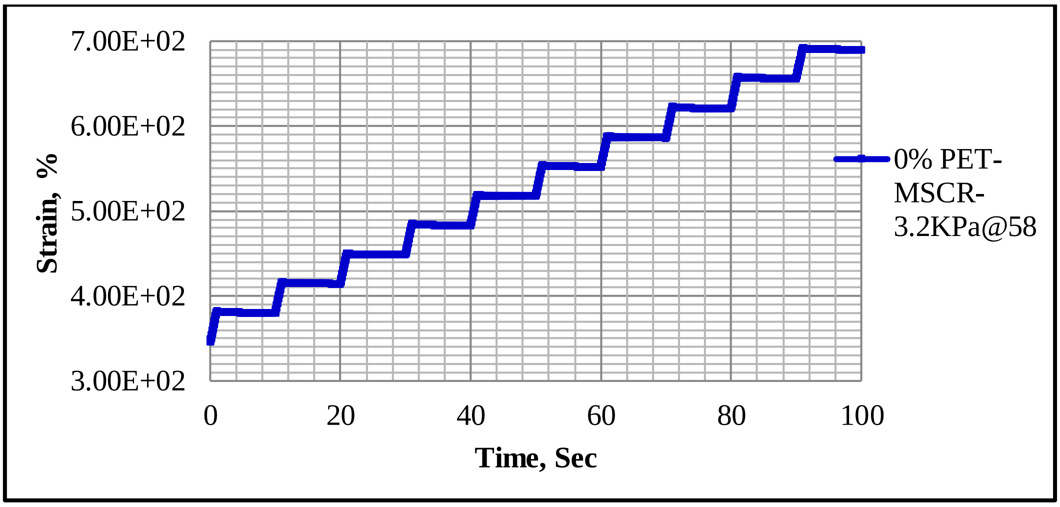


Figure 4.16 MSCR Graph for 0% PET Modified Binder

MSCR test was conducted after the determination of the performance grade of each sample. The PG is then used to establish the MSCR test temperatures as prepared in *Table 4.4*.

Table 4.4 MSCR Test Temperatures Based on PG

PET Content, %	PG	Test Temprature, ^{O}C		
0	58	52	58	64
3	58	52	58	64
6	64	58	64	70
9	70	64	70	76

For this study, repeated shear creep testing was conducted at three temperatures 58°, 64° and 70 °C after the determination of performance grade for binder mix.

The analysis is carried out to predict the Rutting effect by the indication of elastic response and specification preparation of a binder. To evaluate those properties the main parameters are non-recoverable creep compliance, percent creep compliance difference and the percent elastic recovery. Non-recoverable creep compliance (Jnr) is calculated as non-recoverable strain divided by applied stress (0.1kpa or 3.2kPa). Percent difference in creep compliance is computed as (Jnr at 3.2kPa – Jnr at 0.1kPa) / Jnr at 0.1kPa multiplied by 100. And the percent elastic recovery is computed as (strain at 1sec – strain at 10sec) *100 / strain at 1sec, which must be greater than 25% or 35% depending on traffic volume. The calculated values are organized using the separate table for each type of binder as follows.

Table 4.5 Analyzed Jnr and percent Recovery for Modified Binder

%PET	**0%**	**3%**	**6%**	**9%**
Temperature (°C)	58	58	64	70
Percent Recovery at 0.1kPa	5.32	9.96	12.19	12.77
Percent Recovery at 3.2kPa	0.79	1.68	2.33	3.46
Percent Recovery difference (%)	85.10	83.10	80.93	72.94
Jnr at 0.1kPa	3.38	3.32	2.66	2.15
Jnr at 3.2kPa	3.88	3.93	3.20	2.51
Jnr Difference (%)	14.74	24.83	20.09	16.67

Now it is possible to evaluate all the binders by contrasting the calculated basic MSCR parameters concerning the limits of those parameters described under the standard specification for performance graded asphalt binder using the MSCR test, AASHTO M 322.

Table 4.6 Binder Specification Requirement Based on MSCR grade limit

Traffic Designation	Traffic Level, ESAL'S	Load Rate (Km/h)	Max Jnr3.2, Kpa-1
Standard Traffic "S"	< 3 million	>70	4
Heavy Traffic "H"	> 3 million	20 to 70	2
Very Heavy Traffic "V"	> 10 million	<20	1
Extremely Heavy Traffic "E"	> 30 million	<20	0.5

Source:(https://www.fhwa.dot.gov/pavement/materials/pubs/hif11038/hif11038.pdf, Access Date: May 21, 2021, at 2:12 AM)

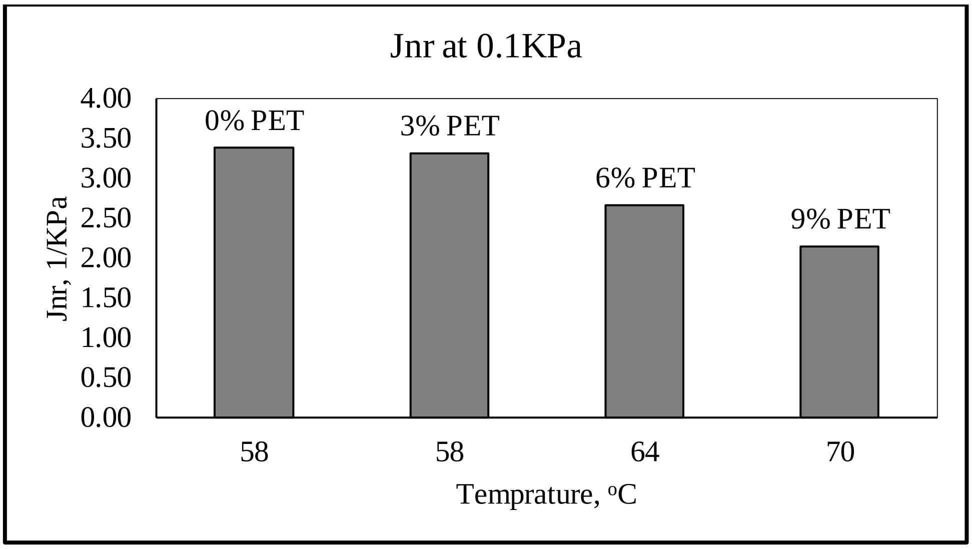


Figure 4.17 Jnr at 0.1KPa at Representative Test Temperatures with different PET

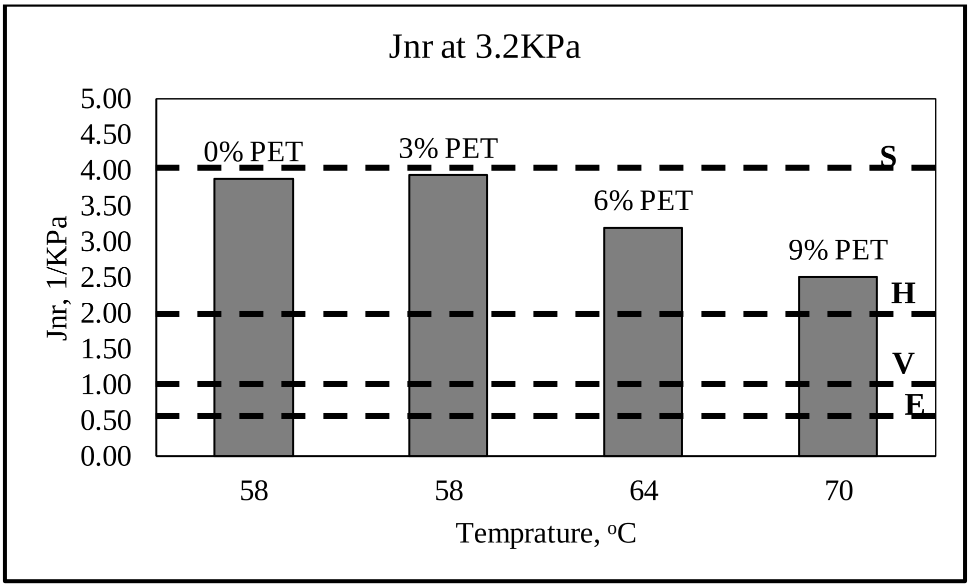


Figure 4.18 Jnr at 3.2KPa at Representative Test Temperatures with different PET

The non-recoverable creep compliance (Jnr) at 3.2kPa is the fundamental parameter to evaluate rutting potential. Based on AASHTO M 332 as described above in *Table 4.6* the horizontal lines denoted by S, H, V and E in *Figure 4.18* represents the standard Jnr values related to traffic. The result shows that the original bitumen (0% PET) and 3% PET containing bitumen have almost similar Jnr results. These two binders with test temperature 58°C have not to be used for heavy and extremely heavy traffic on the test temperatures. Both the binders can be used for standard traffic at pavement design temperature 58°c. And also 6% PET containing bitumen at a test temperature of 64°C has a lower Jnr value and it approaches heavy traffic comparing with the original binder and 3% PET modifier bitumen. At 9% PET modified binder with test temperatures of 70°C can be used for heavy traffic and approaches to very heavy traffic compare with 0, 3, and 6% PET plastic. This exhibit that the PET plastic modifier improves the rheological properties of a binder. This means the PET content increase with a decrease in the non-recoverable (permanent) shear strain of the binder and increasing the recoverable shear strain. Therefore, at 9% PET plastic content shows a significant improvement for rutting resistance at a test temperature of 70°C.

The other important parameter is elastic recovery, and hence comparison of percent recovery of 0.1 and 3.2KPa. In both cases, the percent recovery increases as the percentage of binder replacement with PET plastic increases.

4.2. The Effect of PET on the conventional properties of Unaged Asphalt binder

4.2.1. The Effect of PET plastic on the Penetration Test

The test results of the asphalt binder 60/70 grade prepared at the various percentage of PET plastic 0, 3, 6, and 9 by total weight of asphalt binder. *Figure* 4.19 represents the effect of variable concentrations of PET plastic on the penetration properties of asphalt binder and presented that decreased penetration grade values by 10.2%, 25.8%, and 31.5% with an increase in the concentration of PET plastic by addition of 3%, 6%, and 9% of PET, respectively, as compared to the original bitumen. This shows the increase in stiffness and consistency of bitumen. The stiffness of the bitumen can be advantageous as increases the stiffness of the material and improve the rutting resistance of the mix and also the penetration grade value of 3% PET modified bitumen is not a significant change from the original bitumen grade range. However, When 6% and 9% PET by weight of bitumen mixed, the penetration result become 49.3mm and 45.5mm respectively, which is closer to bitumen grade 40/50. This mix can be suitably used in hotter climatic conditions, especially in regions where the temperature differential is substantially higher. *Figure* 4.19 shows the linearity equation (PT= -2.4488%PET + 66.253) indicate that one percent of PET plastic adding decreased 2.4488 unit value of penetration grade from the original asphalt binder. This study result is in agreement with the findings of Mahrez & Karim, (2010) as the reported penetration grade decrease as the PET content increases in the mix and it Reduced by 14%, 21%, 30% and 35% in penetration values with the addition of 2%, 4 %, 6% and 8% of PET, respectively, as compared to the original bitumen. The result of this study is also in line with M. Ahmad & Mahdi, (2015) for the blend of PET plastic with asphalt binder and the most effective percentage of Polyethylene Terephthalate (PET) waste was obtained between 10 to 12% by weight of the bitumen decreased by 10.81 to 13.96 % from original grade.

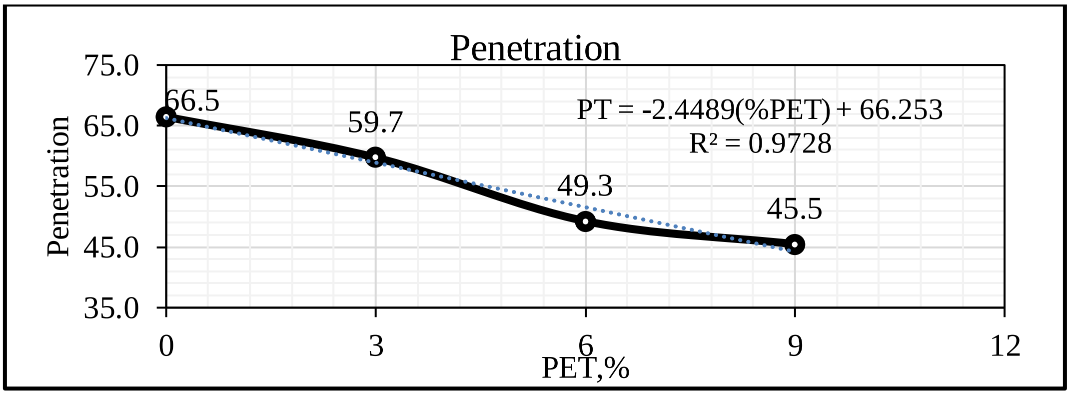


Figure 4.19 Penetration test result with different PET

4.2.2. The Effect of PET plastic on the Ductility Test

Figure 4.20 shows the effect of various percentages of bitumen modified PET plastic on the ductility value of bitumen. The ductility of the original bitumen value showed slightly reduced by 5.6% with 3% PET plastic and gradually decrease with the increase in the addition of a different percentage of PET plastic reduced by 31.6% with 6%. At 9% PET plastic the ductility value is highly reduced by 40.1% compared with the original bitumen. But still, now the value is satisfied with the ERA specification. This shows that the adhesive property of the modified binder decrease with an increase in the concentration of PET due to low interlocking between excessive polymer molecules with bitumen and makes bitumen stiffer. Therefore, the optimum percentage of this PET plastic is necessary for the desired value of ductility of bitumen that is needed for construction work. *Figure* 4.20 shows the linearity equation (DT= -6.9%PET + 145.63) indicate that one percent of PET plastic adding decreased 6.9 unit value of ductility value from the original asphalt binder. The result is in line with the work of Tunde et al., (2020) observation data that the ductility of plain bitumen decreases 1, 3, 13, 18, 25, 32, 38, 44 and 52 cm with the addition of PET from 1-17 % respectively, as compared to the plain bitumen.

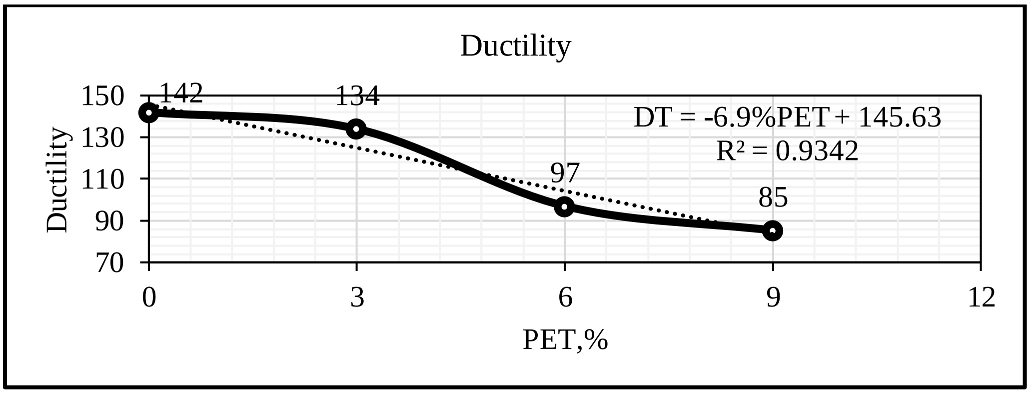


Figure 4.20 Ductility test result with different PET

4.2.3. The Effect of PET plastic on the Softening point Test

Figure 4.21 show that softening point value increases with increasing PET plastic content. This phenomenon indicates that the resistance of the binder to the effect of heat is increased and reduced its tendency to soften in hot weather. Thus, with the addition of PET, the modified binder become less susceptible to temperature changes. Consequently by using PET plastic waste in a bituminous mix, the rate of rutting decrease due to the increase in softening point. *Figure* 4.21 shows the linearity equation (SF= 1.533%PET + 47.65) indicate that one percent of PET plastic adding increased by 1.533 unit of softening value from the original asphalt binder. The finding of this study is in close agreement with the work of M. E. Abdullah et al., (2017) that the plastic waste content is 1.5% by weight of bitumen, the softening point is 53ºC and when plastic waste is added up to 6%, the softening point reaches up to 56 ºC. It said that when the softening point is increasing the reduction of susceptibility at a high temperature. M. Ahmad & Mahdi, (2015) reported that the addition of PET plastic in the bitumen increases the softening point value from 45 ^{0}C for plain bitumen to 56 ^{0}C for PET modified bitumen. It has also been observed that the increase in the softening point of plain bitumen was significant when PET is added in percentages from 10 to 14%. He saide that the increment in the value indicates the resistance of the binder to the effect of heat is increased and reduce its tendency to soften in hot weather.

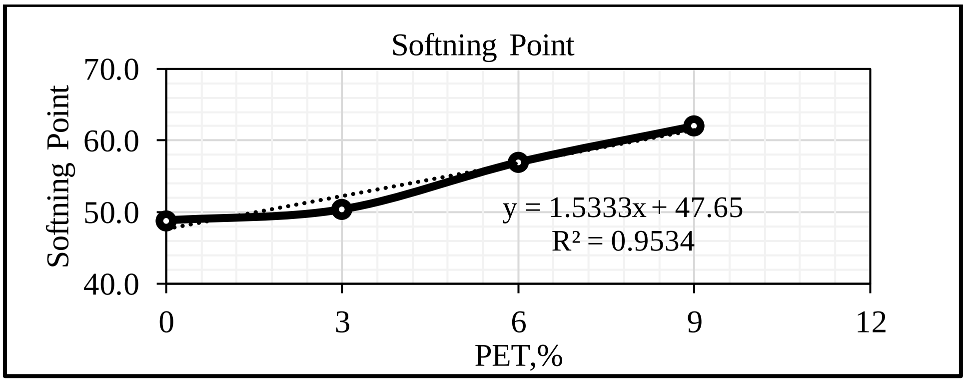


Figure 4.21 Softening Point test result with different PET

4.2.4. The Aging Effect of Bitumen on the conventional tests using RTFO

After ageing the modified bitumen Penetration, Ductility and softening point tests were conducted for each percentage of mixes for the 0%, 3%, 6%, and 9% PET plastic. After ageing, the result of penetration, ductility, and softening point value decreased because of both air oxidation, polymerization (molecules combined and form large molecule), and loss of more volatile components. *Figure* 4.22, *Figure* 4.23, and *Figure* 4.24 Showed that the gap between the aged and unaged value of penetration, ductility, and softening point decreased with increased concentration of PET plastic. This shows that the original asphalt binder becomes highly aged when exposed to air and heat compared with modified asphalt binder and decreased the ageing character because the plastic concentration reduced the amount of asphalt binder content and decrease the oxidation and loss of volatile components.

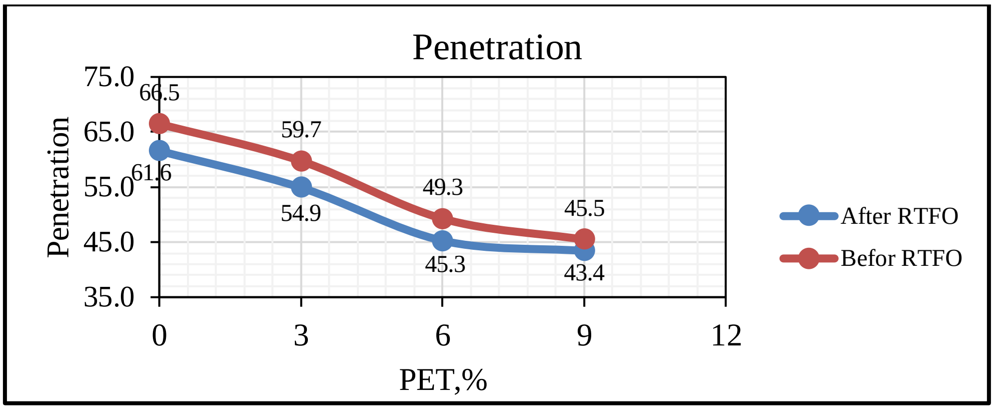


Figure 4.22 Comparison between penetration for aged and unaged binder

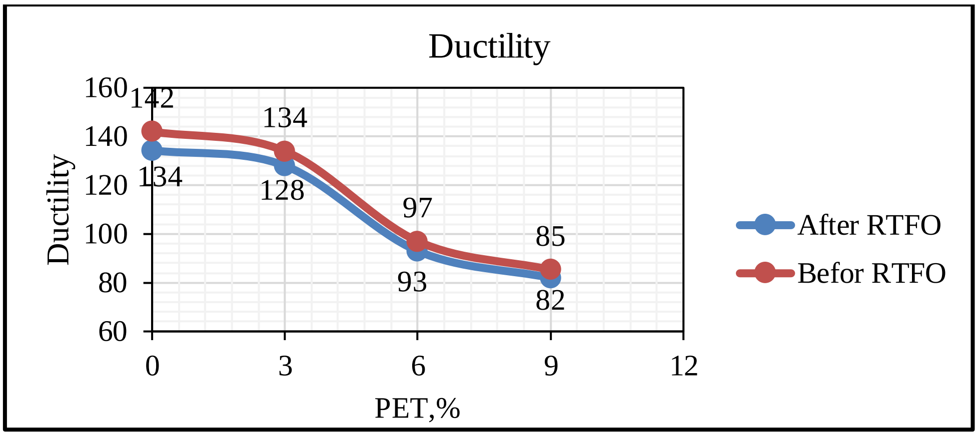


Figure 4.23 Comparison between Ductility for aged and unaged binder

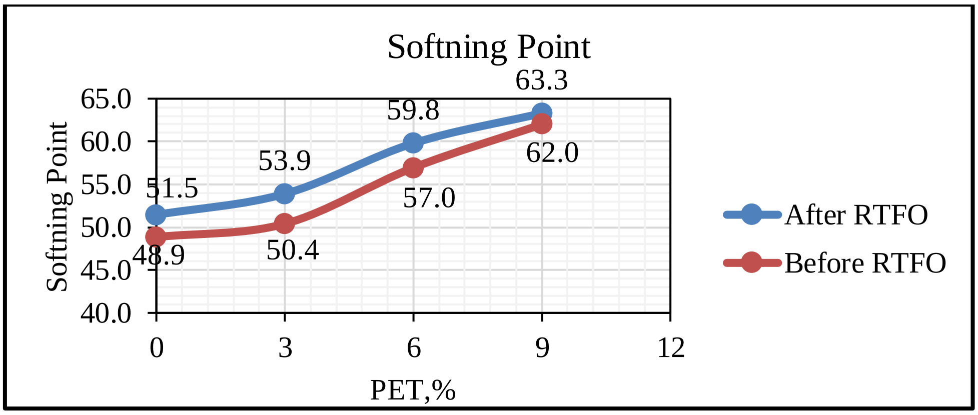


Figure 4.24 Comparison between Softing Point for aged and unaged binder

4.3. Effect of fillers on the Marshal Properties of HMA mixture

The results of Marshal Tests on bituminous mixes prepared by using coffee husk ash (CHA) and stone dust filler (SD) and compared with each other at the same optimum percentage 3.2% by weight of aggregate from job mix formula. From the test results, optimum asphalt content (AC), stability, flow, density, void in the total mixture (VTM), void in the mineral aggregate (VMA) and void fill by asphalt binder (VFA) was selected and compare values. *Table 4.7* and *Table 4.8* indicate the properties of mixtures obtained for the two filler mixtures. The effect of mineral fillers on the properties of the asphalt mixtures discussed in subsequent sections. Detailed laboratory test results are presented in **Appendix C** of the paper.

Table 4.7 Marshall Properties of Mixture with varies percent of AC with SD filler

Bitumen Content (%)	Stability (KN)	Flow (mm)	Density (g/cm3)	VTM (%)	VMA (%)	VFA (%)
3.8	19.23	2.87	2.22	8.69	15.50	43.93
4.3	19.71	3.13	2.24	7.42	15.38	51.78
4.8	21.33	3.17	2.26	5.91	15.07	60.79
5.3	22.04	3.77	2.27	4.83	15.15	68.15
5.7 (OBC)	**19.32**	**4.06**	**2.28**	**3.88**	**15.16**	**74.41**
5.8	18.64	4.13	2.28	3.64	15.16	75.98
ERA Spec.	**Min. 9KN**	**2-3.5**	**N/A**	**4**	**Min.13**	**65-75**

Table 4.8 Marshall Properties of Mixture with varies percent of AC with CHA filler

Bitumen Content (%)	Stability (KN)	Flow (mm)	Density (g/cm3)	VTM (%)	VMA (%)	VFA (%)
3.8	10.91	2.77	2.13	12.39	18.97	34.70
4.3	11.95	3.00	2.17	7.79	15.76	50.59
4.8	13.98	3.17	2.20	6.31	15.46	59.17
5.3	14.54	3.37	2.23	4.44	14.83	70.04
5.4 (OBC)	**14.41**	**3.41**	**2.25**	**4.01**	**14.85**	**71.45**
5.8	13.87	3.60	2.26	3.43	14.97	77.11
ERA Spec.	**Min. 9KN**	**2-3.5**	**N/A**	**4**	**Min.13**	**65-75**

4.3.1. Effect of filler on the Optimum Bitumen Content

As mentioned in the methodology part the optimum Binder Content (OBC) was found out by taking Asphalt cement content corresponding to 4% air void and fix other parameters.

Table 4.7 and *Table 4.8* showed that the corresponding optimum bitumen content for the mix was found to be 5.7 and 5.4% at 4% air void with stone dust and coffee husk ash filler respectively. This result shows that the OBC of SD filler is higher by 0.3% than CHA filler. To observe that the SD filler is highly finer than the coffee husk ash filler. This may occur most of the void between the aggregate skeleton in the mixture is excessively filled by SD filler and reduced the air void that recommends and become denser, raise the stability, and loses the flexibility of the pavement. Additionally, the SD filler absorbs more bitumen content this leading to an increase in the OBC of a mixture, this occurred due to SD filler has a higher surface area to absorb more bitumen. It forms more asphalt-mastic and followed high lateral flow compares with CHA filler. Due to this situation, the surface of the pavement becomes cracked and permanent deformation during the heavy vehicle moved on the surface of the pavement before the design period. This study result is in agreement with the work of Muniandy et al., (2013) reported that the optimum binder content increased as the particle size of filler material decreased by an increase in the interfacial area per unit volume of mineral fillers and a higher bitumen absorption ability. This increases asphalt binder demand. Furthermore, this research showed that an alternative by-product material was used as a filler in the production of HMA to satisfy the ERA specification. Baby et al., (2017) reported that the use of 6.2% of glass powder by weight of aggregate as the filler material is the optimum dosage to obtained 5.5% optimum OBC compared with normal filler lime and cement. He concludes that glass filler increases the Stability value from 9.669KN to 14KN. This shows that the addition of glass powder makes the bituminous mix stiffer. Ararsa et al., (2019) test results indicated that the Optimum Bitumen Content of about 5.1% of the total Hot Mix Asphalt, and the optimum percentage of Sub-base Course Dust (SCD) indicated 6%. And conclude that at 6% Sub-base course dust, obtained 13.9KN and 4.5% stability and total air void in the mixture, this shows higher air void is provided compared with stability value. Due to these, structural defects may have become occurs when heavies vehicles move on the pavement structure.

4.3.2. Effect of filler on Marshal Stability and flow

From *Table 4.7* and *Table 4.8* an optimum asphalt content of 5.7% and 5.4% (by weight of aggregate) was obtained for the mixtures which contain stone dust and coffee husk ash as filler material. At these optimum values, the Marshall Stability values and flow values were 19.32 KN, 4.06 mm, and 14.41 KN, 3.41 mm respectively. *Figure 4.25* show that the Marshall stability value obtained for stone dust and coffee husk ash filler is increased by

114.7% and 60.1% respectively from the minimum ERA specification (Min. 9KN) this is a much higher value compared with non-conventional filler (Coffee husk ash) because of stone dust fillers is highly finer than the coffee husk ash filler and extremely fill the void between the aggregate skeleton in the mixture and increase the stability value besides non-conventional filler specimens are found to show optimum stability value (14.41 kN) compared to conventional filler(stone dust) specimens (19.32 kN). Hence the higher value of stability exhibit pavement becomes denser and loses the flexibility of the pavement and approaches to rigid. Due to this situation, the surface of the pavement becomes cracking due to mixture brittleness during the life of the pavement and permanently deformed through the heavy vehicle moved on the surface of the pavement. Furthermore, *Figure 4.26* shown that at 5.7% of OBC, the flow value of SD filler is 4.06 mm which is out of ERA specification (2-3.5%), and at 5.4% of OBC, the flow value of CHA filler is 3.41 mm which is within the range of the specification. This effect at the higher temperature, the fineness filler mix with asphalt binder and form highly flexible mastic-asphalt pavement that develops laterally moved then permanent deformation becomes occurs while the heavy vehicle moves on the pavement. Rahman et al., (2012) reported that the optimum bitumen content of 6.2% and 6.1% by weight of aggregate for the mixtures which contain Brick dust and cement as filler material respectively exhibit slightly higher stability and flow value 11.50 kN and 3.20 mm, compared to cement filler specimens 10.70 kN and 2.80 mm correspondingly. Moreover, Ararsa et al., (2019) stated that 5.1% of optimum bitumen content contains Sub-base dust as filler material that at 6% of sub-base dust filler, and the stability and flow values is 13.90 kN and 3.66 mm from figure 7 and 8.

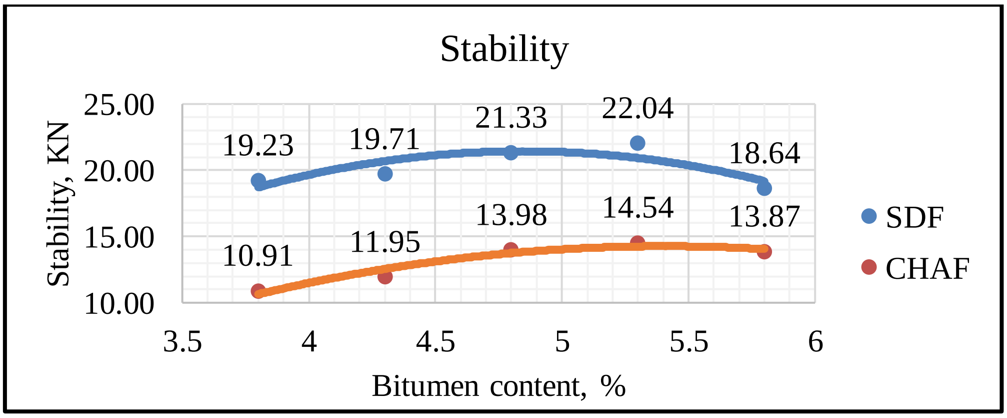


Figure 4.25 Effect of SD and CHA filler type on Marshal Stability

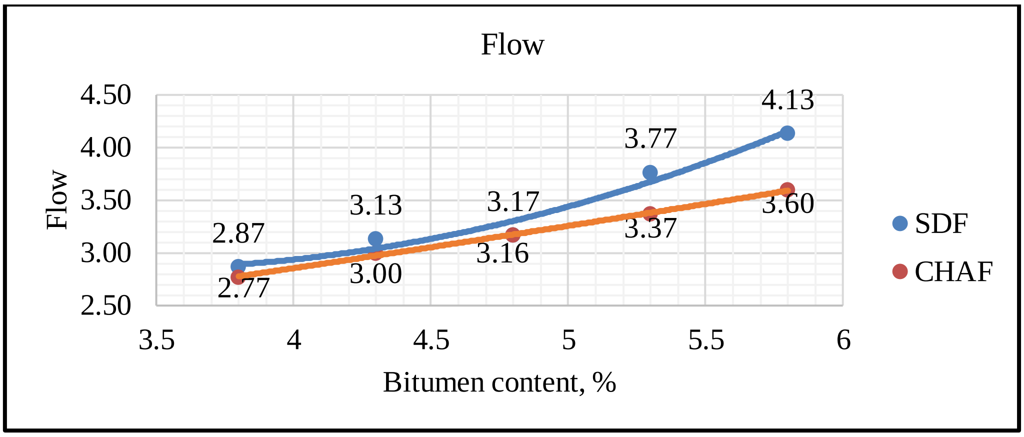


Figure 4.26 Effect of SD and CHA filler type on flow value

4.3.3. Effect of filler on the Density of a Mixture

The effect of both filler type on the unit weight of compacted mixes is shown in *Figure 4.27*. It is shown that at 3.2% filler content, the coffee husk ash filler of a mixture possessed a slightly lower unit weight value (2.25 g/cm^3) as compared with stone dust filler (2.28 g/cm^3). This showed that the size of the SD filler is finer and able to fill more void in the mix. Due to this, increase the density of the mixture as compared with CHA filler. However, the mix becomes stiffer that needs greater compaction effort (Bohara & Tamrakar, 2017). The result of this study is similar to the work of Rahman et al., (2012) shown that at 6.2% of OBC obtained the lower density value of 2.27 gm/cm^3 with Brick dust filler compare with conventional filler (Cement filler) and at 6.1% OBC obtained the higher density value of 2.32 g/cm^3 with conventional filler. Therefore, the structural defect that occurred on the pavement structure and affect the pavement performance when the value of density highly increased.

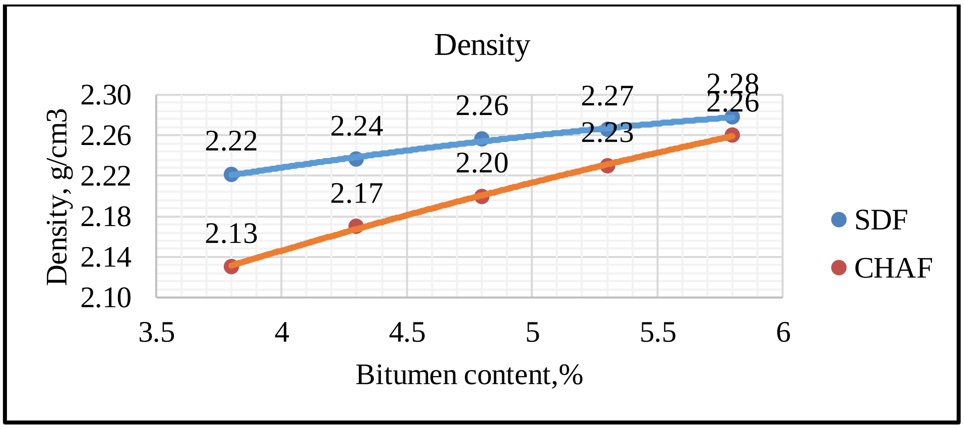


Figure 4.27 Effect of SD and CHA filler type on the Bulk specific gravity of the mixture

4.3.4. Effect of filler on the Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt binder (VFA)

The void space between aggregate particles of a compacted mix is referred to as VMA and is used to indicate the available space for bitumen to adequately coat each aggregate particle and VFA is the void space in the mineral aggregate that is filled with bitumen. This parameter describes the richness of a bituminous bound mix (Mohammed & Fadhil, 2018). As the ERA manual (2013) specifies the minimum VMA based on the nominal maximum particle size used and the design air voids of a mix, the requirement for a typical mix containing 19mm aggregate size and 4% air voids is selected as a Minimum of 13% and VFA value is the range of 65% to 75%. This criterion is important for the durability of mixes and related to the effective asphalt content in the mix. If the percentage of voids filled with asphalt is lower than the limit indicated, there less asphalt film around the aggregate particles. Lower asphalt films are more subjected to moisture and weather effects where they can be detached from the aggregate particles and subsequently lower performance. On the other hand, if the limit is exceeded, more voids are filled with asphalt than required for durability. This can be explained as the asphalt film around aggregate particles is thicker and lower voids than required are left. This increased amount of effective asphalt results in bleeding and lower stiffness of the mix (Tefera et al., 2018). As shown in *Figure 4.28* is used to indicate the available space for bitumen to adequately coat each aggregate particle. It can be seen that the VMA values of SD and CHA filler (15.16% and 14.85%) are considerably 0.31% slightly higher than CHA filler. This exhibit the difference in the filler size of the two materials. In addition, *Figure 4.29* shows that the VFA value of SD and CHA filler is 74.41% and 71.45% respectively. This also noticeably 2.96% slightly higher than CHA filler. However, the result is not a significant difference on both filler material on the VMA and VFA. Al-Shamsi et al., (2017) defined VMA as intergranular void space in a compacted asphalt mix. It is the volume of air voids and the volume of effective asphalt and an important parameter in hot mix asphalt (HMA) mix design. It is said that an optimum VMA requirement is necessary to ensure adequate binder content and durability for HMA. The result of this study is comparable to the work of Rahman et al., (2012) shown that at 6.2% of OBC obtained the VMA value of 19.25% with Brick dust filler and at 6.1% OBC obtained the VMA value of 15.5% with cement filler. Sutradhar et al., (2015) used three types of filler material (fine sand and stone dust, Waste concrete dust, and Brick dust) then reported that at OBC of these materials is 5.3%, 5.33%, and 5.37% the value of VMA and VFA is

18.92% and 82.23%, 16.40% and 79.20%, 20.30% and 82.23% respectively. Those values are exceeded from the limit, more voids are filled with asphalt than required for durability and the asphalt film around aggregate particles is thicker and lower voids. This increased amount of effective asphalt results in bleeding and lower stiffness of the mix.

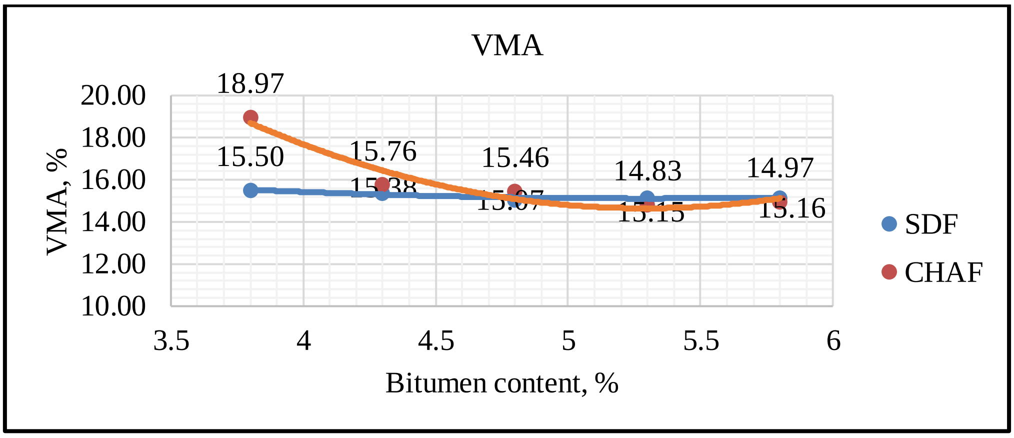


Figure 4.28 Effect of SD and CHA filler type on Void in the Mineral Aggregate (VMA)

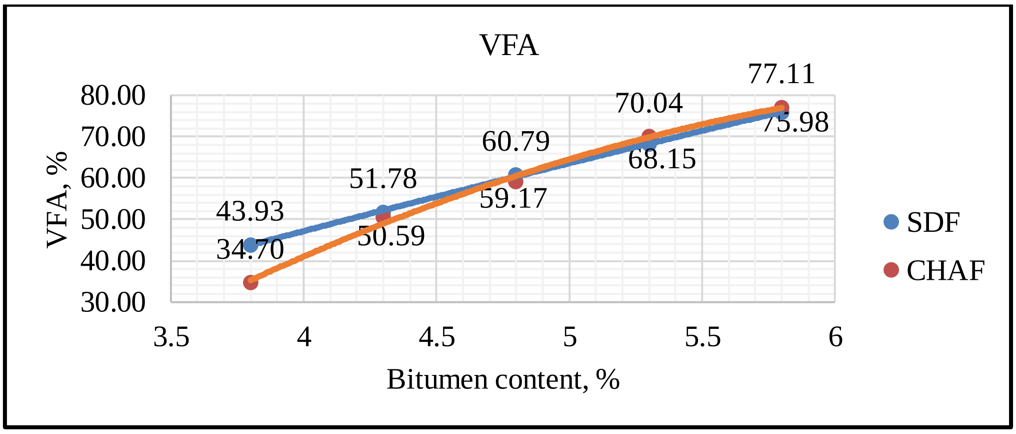


Figure 4.29 Effect of SD and CHA filler type on Voids Filled with Asphalt (VFA)

4.4. The Effect of PET plastic on the Marshal Properties of HMA

4.4.1. The Effect PET on the Optimum Bitumen Content

As mentioned in the methodology part the optimum Binder Content was found out by taking bitumen content corresponding to 4% air void and fix other parameters. And from *Table 4.8* the corresponding optimum bitumen content using coffee husk ash filler for the mix was found to be 5.4% at 4% air void. After obtaining OBC, to evaluate the effect of adding shredded plastic Polyethylene terephthalate(PET) to asphalt mixture samples by considering four different proportions of shredded plastic 3%, 6%, 9%, and 12% of waste plastic by the weight of OBC by using a dry process method. Detailed laboratory test results are presented in **Appendix C** of the paper. *Table 4.9* shown that the percentage of PET plastic increased with decreased the optimum bitumen content and modifies the Marshal properties of a mixture. The combination of optimum PET plastic and bitumen content is 6.8% by weight of OBC and 5.04% by weight of aggregate at 4% air void was the optimum value to achieve a good performance of mixture. This shows at this optimum can use PET plastic used as a bitumen modifier to achieve the ERA specification.

Table 4.9 Marshall Properties of Mixture with varies percent of PET plastic with CHA filler

% of PET plastic by wt. of OBC	OBC Content (%), Pb	Stability (KN)	Flow (mm)	Density (g/cm3)	VTM (%)	VMA (%)	VFA (%)
0	**5.40**	**14.30**	**3.50**	**2.256**	**3.90**	**15.62**	**75.06**
3	5.25	14.26	3.37	2.251	3.87	15.66	75.29
6	5.08	15.57	3.03	2.266	3.94	14.98	73.69
6.8 (OPET)	**5.04 (OBC)**	**15.90**	**2.96**	**2.268**	**4.00**	**14.83**	**73.01**
9	4.92	16.79	2.77	2.276	4.17	14.42	71.12
12	4.75	13.90	2.97	2.280	5.14	14.74	65.12
ERA Specification		**Min. 9KN**	**2-3.5**	**N/A**	**4**	**Min. 13**	**65-75**

OPET= Optimum Polyethylene terephthalate, OBC= optimum Bitumen content

4.4.2. Effect of PET plastic on the Marshal Stability and flow

Figure 4.30 exhibits the effect of PET plastic contents on Marshall Stability. This show that increases the PET plastic with decrease the OBC in the mixture, Marshall Stability also increases up to 9% value of PET content and then decline. This is because voids at lower asphalt content and higher PET content are too high and the void to be filled with PET plastic as coated over the aggregate and then separate aggregate interlock hence effect tend to reduce the stability values. The stability value of modified bitumen (Bitumen with PET) is 15.9 KN which is higher than unmodified bitumen 14.3 KN by 1.6 KN variation. This indicates that the upgrading of stability of asphalt mixes with optimum PET plastic established a better bond between bitumen and PET plastic coated aggregates due to intermolecular bonding, these intermolecular magnetism improved the stability of the mix.

In addition, flow refers that the vertical deformation of the sample (measured from the start of loading to the point at which stability begins to decrease). High flow values generally indicate a plastic mix that will experience permanent deformation under traffic, whereas low flow values may indicate a mix with higher than normal voids and insufficient asphalt for durability and one that may experience premature cracking due to mixture brittleness during the life of the pavement. *Figure 4.31* shows flow results of mix decreased with the percentage of PET plastic increase while the increase the flow value at the stability decline. This is because of the decline of bitumen content, the PET plastic particles overcoating the aggregate which rise the void in the mixture then the loss of the intermolecular magnetism of the pavement. However, as excessive PET content decreases the stability, it increases the flow consequently. The flow value covers from 2.97mm to 3.5mm with the range of PET plastic from 0% to 12%. The result shows that flow in the mixture is within the range of the ERA specification. At optimum PET plastic 6.8% and 5.04% OBC, the flow value of modified bitumen is 2.96mm which is lower than unmodified bitumen 3.5mm by 0.54mm difference. This study result is in agreement with the work of *Prasad et al., (2013)* indicates that the Marshall stability and flow value at 5.3% optimum PET plastic and 5.03% OBC is 23.47 KN and 2.31mm compared to the mix without plastic, which is 16.2 KN. Furthermore, Neha et al., (2015) reported that the stability value increased with increased the percentage of PET plastic from 5-10% then decline at 15%. He concludes that at 10% optimum PET plastic gets the maximum stability value of 19.21 KN compared with unmodified bitumen. Mosa, (2017) stated that stability values increase with the increase of PET content from 0-0.5% by 0.2% increment then decrease with an increase of PET content up to 1.1%. At 0.5% optimum PET content and 5.5% of OBC produces 16.5KN and 10.5mm stability and flow value which is stiffer mixtures compared to14.7KN and 11.7mm unmodified respectively. However, excessive PET content possibly increases the heterogeneity of the mixture and reduces stability. In addition to this, Marshall Flow values decrease with an increase of PET contents up to 0.3% then increase with an increase in PET content. As mentioned, adding PET produces stiffer mixtures which decrease the flow. However, as excessive PET content decreases the stability, it increases the flow consequently. The result of this study is also in line with Dalia et al., (2015) as reported that the flow of modified asphalt mixture (2.12mm) is lower than the conventional asphalt mixture value(3.8mm). This shows that the flow decreases continuously as the PET plastic modifier content increases at 8% of PET plastic, then increase at 10% PET plastic content.

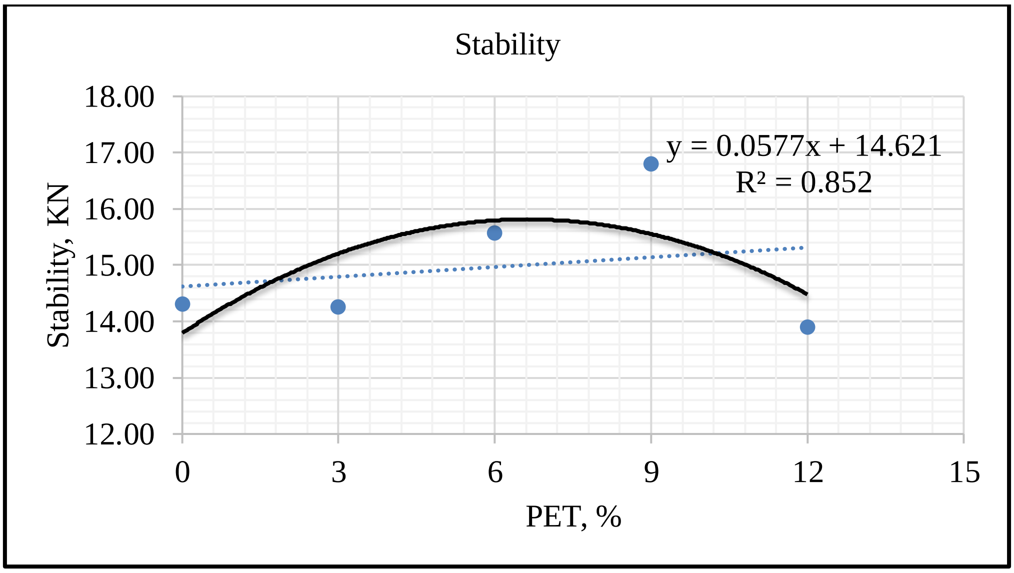


Figure 4.30 Effect of PET plastic with CHA filler on the Marshal Stability

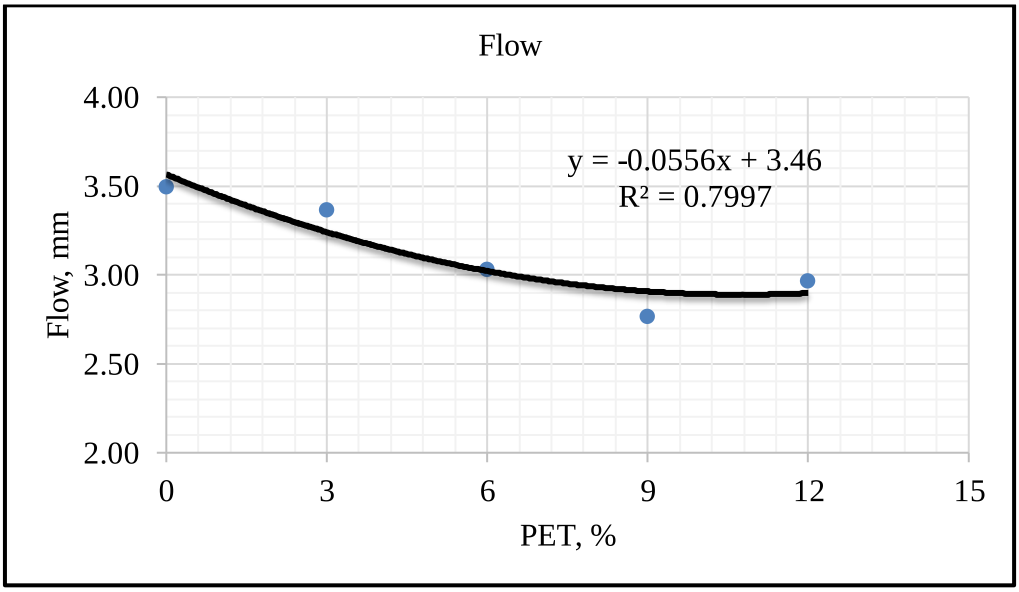


Figure 4.31 Effect of PET plastic with CHA filler on the Flow value

4.4.3. Effect of PET plastic on the Density of a Mixture

The effect of PET plastic content on the unit weight of the mixture is shown in *Figure 4.32*. The bulk specific gravity of the mixture initially decrease and then increase with an increase in the percentage of PET plastic. The density value of modified bitumen (Bitumen with PET) is 2.268 g/cm^3 which is slightly higher than unmodified bitumen 2.256 g/cm^3 by 0.012 g/cm^3 variation. This is because PET plastic is a higher unit weight compared to the binder used to coat the aggregate, hence increase the unit weight of a mixture. This study result is in agreement with the work of *Moghaddam et al., (2013)* stated that bulk specific gravity (Gmb) of the mixture increased by adding PET plastic then the trend decreased at higher plastic contents. Further, mixtures manufactured at higher asphalt contents showed to have higher Gmb. Gmb is 2.315 gm/cm^{3}for the mixtures with 0.4% and 7% optimum plastic bottle and asphalt content, respectively compared with 2.295gm/cm^3 unmodified bitumen. This study result is in disagreement with the work of Hidayat et al., (2019) for the highest density value was 2.24 gm /cm^3 in the mixture without added PET plastic, while the addition of PET plastic decreased. The addition of 1% PET content obtained the density value of 2.23gm/cm^3, 1.5% PET content of 2.22 gm/cm^3 and 2% PET content of 2.21 gm/cm^3. The result showed that the density value of concrete asphalt mixture without PET addition was higher than the mixture with PET addition. So that the major problem to observe from this work was PET plastic mixed with asphalt at the temperature of 115^{0}C, the added plastic was not perfectly melted which still in the form of fine fibres and it forms a heterogeneous mixture. However, this research used temperature between 150-160^{0}C continued for 45-60min to produce homogenous mixtures. Likewise, the type of filler (carbide waste filler-a compound of carbon with a metal or other element: *silicon carbide*.) may be another factor that used in the mixture that is why the variation has happened between us. The result of this study is also in line with Dalia et al., (2015) as reported that the maximum bulk density of PET plastic modified asphalt mixture (2.375 g/cm^3) is higher than the conventional asphalt mixture value (2.355 g/cm3) at 8% of PET plastic by weight of 5% of OBC.

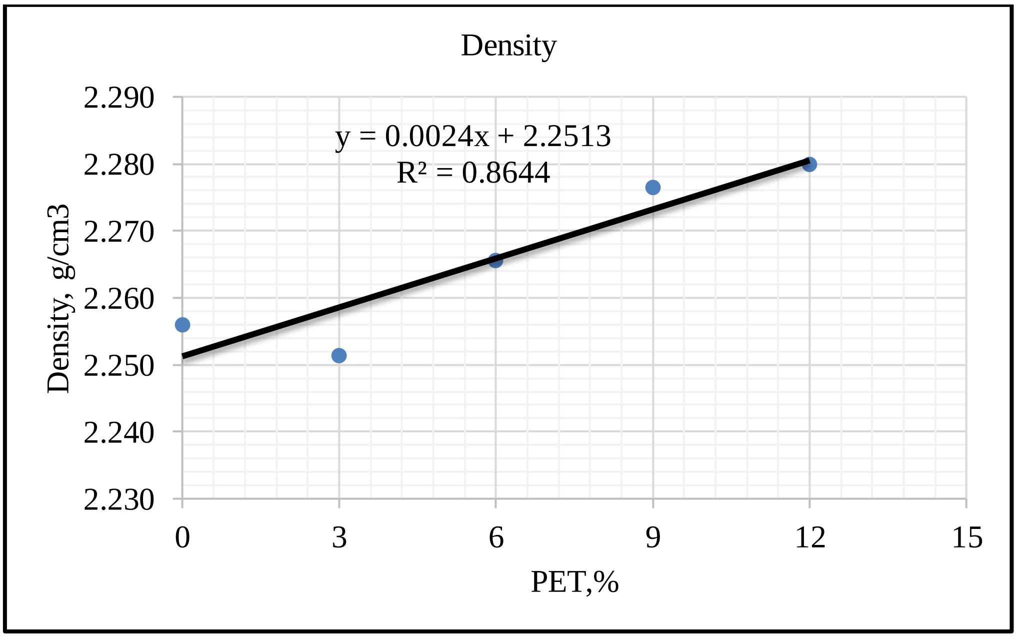


Figure 4.32 Effect of PET plastic with CHA filler on the Bulk specific gravity of the mixture

4.4.4. Effect of PET plastic on the Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt binder (VFA)

Minimum VMA is necessary for mixtures to accommodate enough asphalt content so that aggregate particles can be coated with adequate asphalt film thickness. This consequently results in a durable asphalt paving mixture (Al-Shamsi et al., 2017). It can be seen from *Figure 4.33* the lower VMA in the mixtures with an increase in the PET plastic content. Here the modified asphalt mixes are generally the lower VMA value than conventional asphalt mix. This indicates that the void here may fill with this PET plastic and it reduces the mineral aggregate void. In general, the VMA percentage of the modified asphalt concrete mixtures (14.83%) is lower than the conventional asphalt concrete mixture value of (15.62%) at 6.8% optimum PET plastic, the results are satisfying ERA specification (Min. VMA = 13%). In addition to this, the effect of PET plastic content on the voids filled with the asphalt property of the mixture is indicated in *Figure 4.34*. This indicates that the VFA percentage of the modified asphalt decrease with an increase in the percentage of PET plastic. The Voids filled with conventional asphalt binder value (15.62%) is greater than PET plastic contents (14.83%) at 6.8% optimum PET plastic. According to ERA specification for VFA is in the range of 65% - 75%. This criterion is vital for the durability of mixes and is related to the effective asphalt content in the mix. If the percentage of voids filled with asphalt is lower than the limit indicated, less asphalt film around the aggregate particles. Lower asphalt films are more subjected to moisture and weather effects where they can be detached from the aggregate particles and subsequently lower performance. On the other hand, if the limit is exceeded, more voids are filled with asphalt than required for durability. This can be explained as the asphalt film around aggregate particles is thicker and lower voids than required are left. This increased amount of effective asphalt results in bleeding and lower stiffness of the mix (Tefera et al., 2018). The result of this study is also in line with Dalia et al., (2015) stated that the VMA percentage of the modified asphalt concrete mixtures(16.21%) is lower than the conventional asphalt concrete mixture value of (16.9) at an 8% optimum PET plastic, the (VMA) decreases continuously as the PET plastic modifier content increases. According to Hidayat et al., (2019) reported that the increase of PET levels affecting the decreased value of VFA. It influenced by the mixing plastic that was not perfectly flowed and still in the form of fiber covering with asphalt. It reduces the amount of asphalt that should fill the void in the mixture. The highest VFA value was at 0% PET content of 76.27%. Meanwhile, the level of 1% PET decreased by 9.09%, so that the VFA value of 67.18% was obtained. Then, 1.5% PET content with the value of 62.07% and 2% PET content had decreased by 5.38%, so that VFA value is 56.59%. Based on these results, it can be concluded that the levels 0% and 1% had fulfilled the requirements.

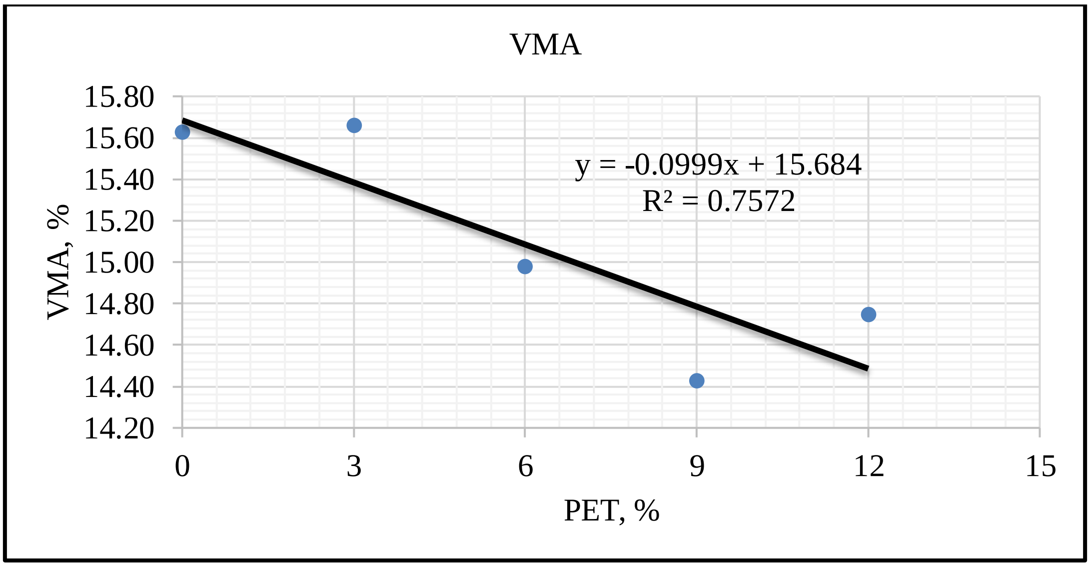


Figure 4.33 Effect of PET plastic with CHA filler on the Void in the Mineral Aggregate (VMA)

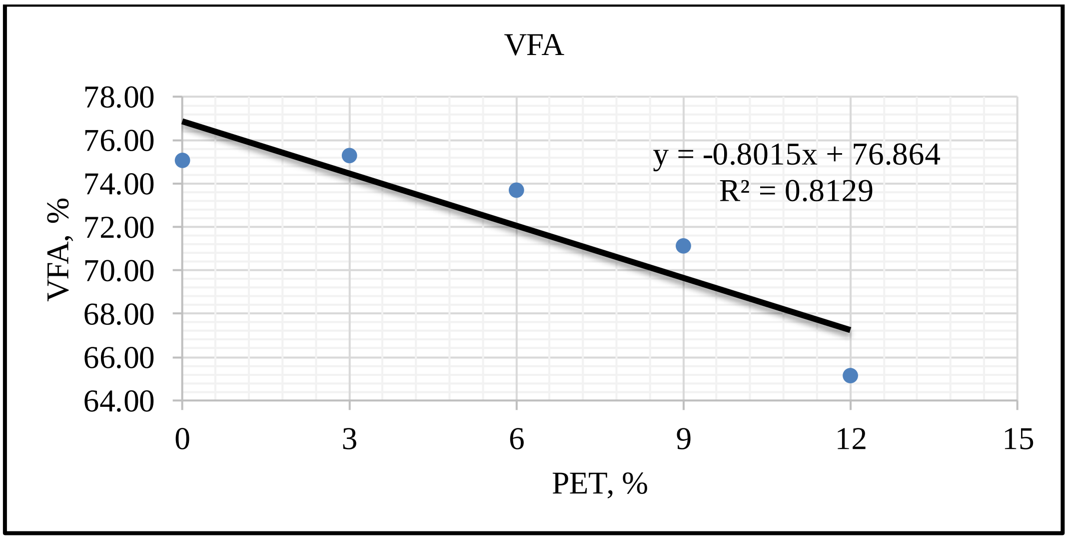


Figure 4.34 Effect of PET plastic and CHA filler on Voids Filled with Asphalt (VFA)

4.5. The Numerical Results of Pavement Response with and without PET plastic

In this section, to compare the performance of plastic pavement with the normal pavement by considering the effects of loading a moving tire on the pavement by a recreation of dual tire single, dual tire tandem, Super single tire tandem-axle and Special non-highway vehicle loading conditions.

Table 4.10 The AC Rutting, Top-Down, and Bottom-Up Cracking values of varies load case

Load Case	**Distress Type**	**% PET Plastic**			
		0	**3**	**6**	**9**
Dual Tire Single Axle Load (Load Case-2)	AC Rutting (mm)	9.04	9.08	5.16	2.97
	AC Top Down Cracking (%)	30.40	32.90	13.50	9.87
	AC Bottom UP Cracking (%)	28.80	27.60	19.60	4.71
Dual Tire Tandem Axle Load (Load Case-3)	AC Rutting (mm)	9.20	9.07	4.34	2.73
	AC Top Down Cracking (%)	23.54	25.70	11.94	11.01
	AC Bottom UP Cracking (%)	29.47	31.04	5.69	3.75
Super single Tire Tandem Axle Load (Load Case-7)	AC Rutting (mm)	7.69	7.60	4.34	2.49
	AC Top Down Cracking (%)	23.45	20.56	11.30	8.88
	AC Bottom UP Cracking (%)	10.53	9.24	5.74	4.29
Special Non-Highway Vehicle, End Dump Truck (Caterpillar)	AC Rutting (mm)	5.53	6.78	5.03	2.37
	AC Top Down Cracking (%)	10.67	15.03	8.89	4.62
	AC Bottom UP Cracking (%)	1.29	0.82	0.76	0.62

Table 4.10 show that the 3D-Move analysis software result output for asphalt binder with different PET content and load case. Results showed that AC Rutting, top-down, and bottom-up cracking decrease with an increase in the PET contents. *Figure 4.35* shows that the rutting depths in the asphalt concrete layer decreased with the increase of PET plastic content. At 0 and 3%, plastic content has no significant variation while at 6 and 9% PET content the rutting failer decrease with increase the PET. Furthermore, Dual tire single axle load has higher pavement damage than another load case because the load is concentrated and higher load effect on the pavement while super single tire tandem axle load has a lower effect than other because it has more than two axel and single wider tire base. Due to this, a higher contact surface area and leads to the distribution of the load uniformly and decrease the pressure. In addition to this, the rutting resistance of a modified binder is better than a conventional binder. *Figure 4.36* also shown that the results for top-down cracking followed the same trend as rutting, at 3%PET plastic has slightly higher than the conventional binder. However, at 6 and 9%, PET plastic content have a lower top-down value and best performance. This indicates that the PET plastic modifier binder has better resistance to top-down cracking at the top of the pavement by decreasing High surface horizontal tensile stresses due to truck tires, age hardening of the asphalt binder resulting in high thermal stresses in the HMA, and modify the stiffness of the upper layer caused by high surface temperatures. *Figure 4.37*

indicate that HMA pavement has a better resistance of bottom-up cracking when the PET content increase. This means that the modified binder of HMA pavement has a better resistance of tensile bending stresses at the bottom of the HMA layer and then progress up to the surface of the pavement. Super single tire tandem axle load has a lower value than others while Special non-highway vehicle end dump truck has a less significant effect on the bottom-up cracking effect.

Overall results of the analysis showed that asphalt binders with higher PET plastic content have the best performance since the stiffness of the asphalt mixes would increase with the addition of PET plastic. Thus, making asphalt mixes perform better and its resistance to distresses higher.

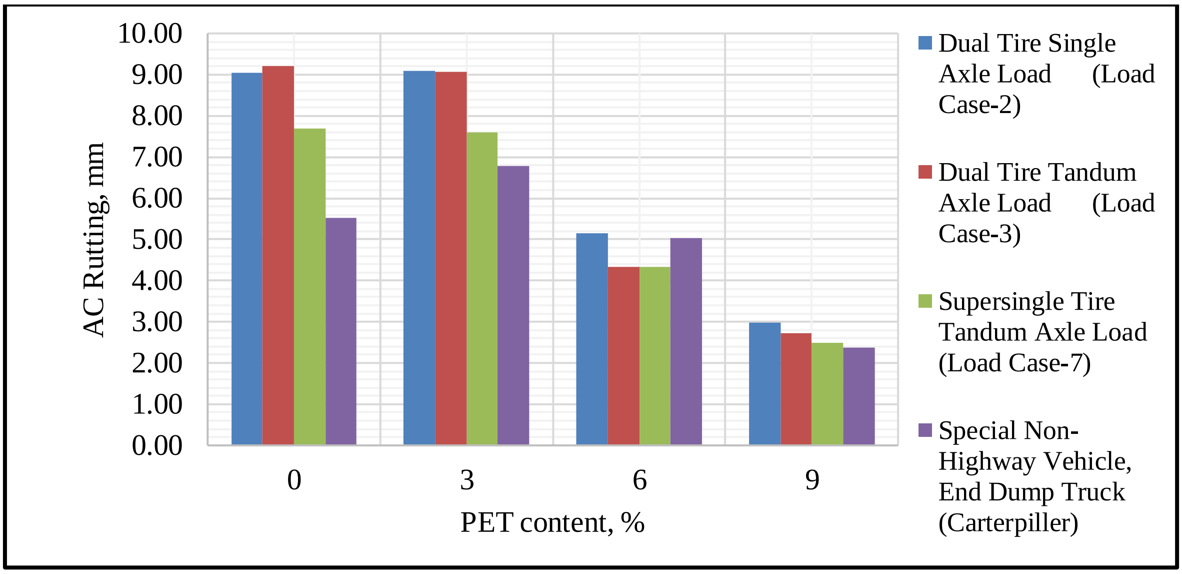


Figure 4.35 Effect of PET content on rutting of asphalt concrete layer

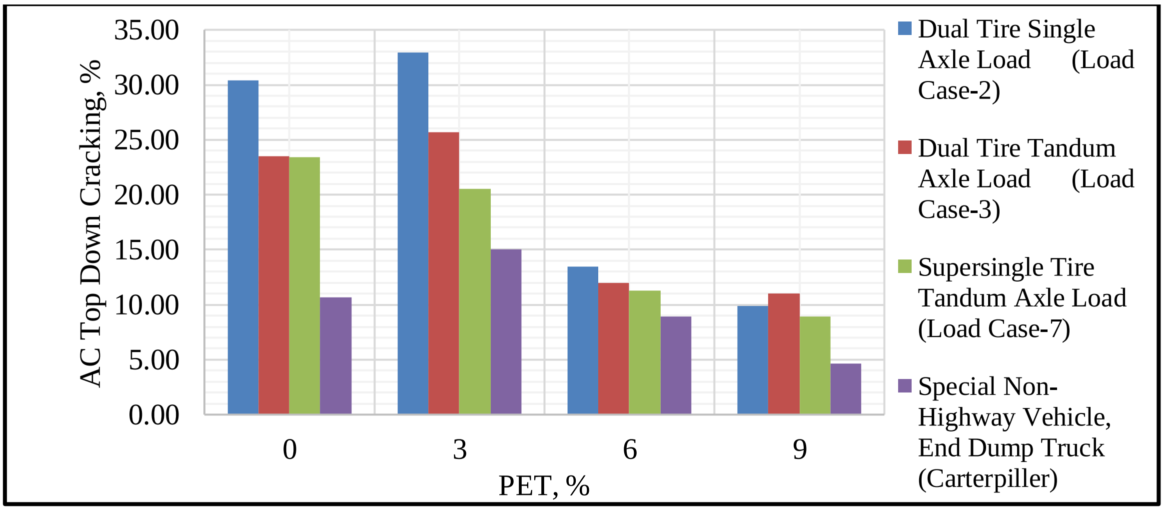


Figure 4.36 Effect of PET content on the top-down cracking of asphalt concrete layer

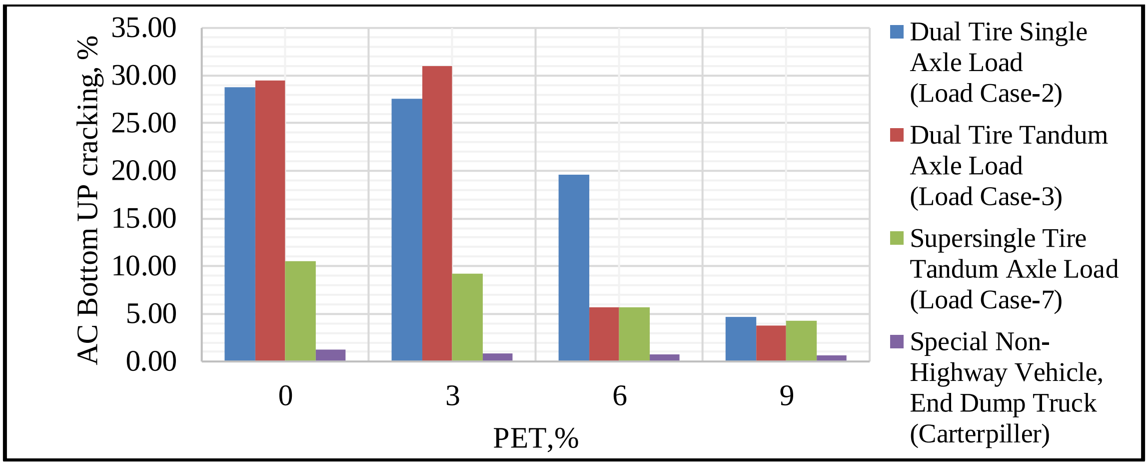


Figure 4.37 Effect of PET content on the bottom-up cracking of asphalt concrete layer

4.6. Summary

Test results obtained from the Dynamic shear rheometer, conventional and Martial tests were organized and analyzed under this chapter. What is observed is summarized as follows.

- As observed from the amplitude sweep test linear viscoelastic range increases as temperature increases despite the decrease in shear modulus with an increase in the PET content.
- PG determination indicates that 3%PET plastic modification does not change, but modifying with 6 and 9% has improved the PG 58 original bitumen to PG 64 and PG 70 modified binder.
- Observed from the frequency sweep test, the modulus master curves show that at low frequency and high temperature the shear stiffens increases as the percent modifier increases. And the phase angle master curve depicts the decrease in phase angle at low frequency and high temperature due to the increase in the modifier.
- The MSCR also showed that there is an improvement in non-recoverable creep compliance as the content of PET increases.
- From the conventional asphalt test, the penetration and ductility value decreased with an increased softening point as the PET plastic content increased.
- The HMA containing a combination of 6.8% PET plastic and 3% coffee husk ash filler shows better mechanical performance on all the Marshall stability and the volumetric properties compared to the conventional HMA.
- From 3D-Move Analysis shows that the PET content increase with a decrease in the rutting damage, top-down, and bottom-up crack.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

This research tries to study the rheological and conventional properties of a binder modified by PET plastic and the mechanical behaviour of HMA made from the combination of PET and CHA filler. In doing so, test parameters were evaluated. Based on the results obtained from this study, the following conclusions can be made:

The conclusion from Amplitude Sweep Test (AST)

- Linear viscoelastic range increases as temperature increases even though the complex modulus of pure and modified binders have decreased.
- And also increase G* values with the increase in the percentage of PET plastic. This because PET plastic changes the rheological properties of the binder by increase the total resistance of the binder to deformation when frequently sheared.
- As observed from the result a long LVE region has scored at a binder modified at 6% PET plastic modifier. This means that at 6% PET plastic content was a better performance than the other modifiers according to the LVE region.

The conclusion from the Performance Grade Determination Test (PG)

- PG grade upgrading from PG 58 to PG 64 and PG 70 was observed when the neat bitumen is modified with 6 and 9% PET plastic respectively.
- As the values of complex shear modulus (G*) increased and the values of the phase angle (δ) decreased, the value of the rutting parameter, $G^*/\sin\delta \geq 1.00$ kPa for unaged binder and $G^*/\sin\delta \geq 2.2$ kPa for aged binder increased. The higher this value is, the greater the resistance of the modified binder to rutting at high temperatures.
- Based on this it is possible to improve the rutting performance due to the modifier.

The conclusion from the Frequency sweep Test (FST)

- FST shows the relation between complex modulus Frequency and phase angle and all binders are almost similar pattern shear stiffness decreases as temperature increases.
- At high temperature and low frequency, the smaller value of shear modulus occurs but at a lower frequency and higher temperature, the modulus increases as the modifier increases.
- At high temperature and lower frequency, the phase angle decrease as a PET plastic content increase compared with a virgin binder.

- It is possible to say, the PET plastic modifier improves the complex shear modulus and phase angle of the virgin binder and improves the rutting problem at high temperatures due to slow-moving traffic.

The conclusion from the Multiple Stress Creep Recovery Test

- The virgin bitumen and the 3% PET plastic modified binders have similar Jnr results at 58^{0}C and have not a significant effect.
- At 6% PET plastic content has a lower Jnr value and it approaches to use for heavy traffic compared with the original binder at a test temperature of 64^{o}C.
- A lower Jnr value observed at 9% PET modified binder with test temperatures of 70^{o}C can be used for heavy traffic and approaches to very heavy traffic compare with 0, 3, and 6% PET plastic.
- In general, the PET plastic modifier improves the rheological properties of a binder with a decrease in the non-recoverable (permanent) shear strain of the binder and increasing the recoverable shear strain as a PET content increase. Therefore, at 9% PET plastic content bitumen has a significant improvement for rutting resistance at a test temperature of 70^{o}C.

The conclusion from the Conventional bitumen Test

- The addition of PET plastic changes the bitumen properties by decrease the penetration and ductility value with an increased softening point as the PET plastic content increased.
- At 3% PET modified bitumen is not a significant change from the original bitumen grade range. However, when 6% and 9% PET by weight of bitumen mixed, the penetration result become 49.3mm and 45.5mm respectively, which is closer to bitumen grade 40/50.
- At 3% PET plastic add to the bitumen has no significant effect on the ductility value of the original bitumen. However, gradually decrease with the increase in the addition of PET plastic.
- After ageing, penetration, ductility, and softening point value decreased because of both air oxidation and loss of more volatile components. Which makes it stiffer for all the mixes and decrease the value.

The conclusion from the Marshal Test

- The addition of CHA filler reduced the OBC from 5.7 to 5.4% compared to the control mix (stone dust filler) and the OBC of SD filler is higher by 0.3% than CHA filler.
- Marshall stability value obtained for stone dust filler (19.32 KN) is much higher than coffee husk ash filler (14.41 KN) from the minimum ERA specification. (Min. 9KN). the replacement of SD filler by CHA filler in HMA has satisfied all the standard specification of ERA.
- Due to this, stone dust filler is highly finer than the coffee husk ash filler and extremely fill the void between the aggregate skeleton in the mixture and increase the stability value.
- The optimum content of PET plastic from the Marshal test is 6.8% by weight of OBC.
- The partial replacement of asphalt binder with 6.8% PET plastic at optimum binder content of 5.04%, increase the stability and density value while decrease flow, VMA and VFA percentages compared without plastic.
- In general, it can be stated that the HMA containing a combination of 6.8% PET plastic and 3% coffee husk ash filler shows better mechanical performance on all the Marshall stability and the volumetric properties compared to the conventional HMA.

The conclusion from the 3D-Move Analysis Software

- The results showed that asphalt binders with higher PET plastic content have the best performance to resist the rutting damage, top-down and bottom crack.
- At 0 and 3%, plastic content has no significant variation while at 6 and 9% PET content the rutting failer decrease with an increase in the PET content.
- Top-down cracking at 3%PET plastic has slightly higher than the conventional binder. However, at 6 and 9%, PET plastic content have a lower top-down value.
- Super single tire tandem axle load has a lower value than others while Special non-highway vehicle end dump truck has a less significant effect on the bottom-up cracking effect.

5.2. Recommendations

Based on the study results the following recommendations are made.

- The study showed it is possible to use PET plastic as a modifier of bitumen to modify the rheological properties of bitumen.
- It also possible to improve the performance grade of a binder with the addition of PET plastic in the range of 6-9% PET, thus it is recommendable to use this modified binder in Ethiopia around wide areas when the pavement design temperature less or equal to 64°C use PG64 at 6% PET while above 64^0C used PG 70 at 70^0C.
- It is recommended to more studies are needed to study the effect of chemical composition and interaction between virgin bitumen with PET plastic and Coffee husk ash filler.
- And also advisable to evaluate the Mechanical performance of HMA containing a combination of PET plastic and coffee husk ash filler using an additional mechanical test.
- And also advisable to evaluate the economic benefit of using coffee husk ash filler and PET plastic in HMA and compare them with the conventional mixture.
- It is recommended for the government to implements and promotes this technology by helping the investor or contractor to construct the asphalt pavement depending on the results of this research and other researches, and to encourage using recycled materials in construction fields.
- It also recommended that to consider other types of the axle and non-highway vehicle load in the analysis of 3D-Move software.

5.3. Future Study

If I have an opportunity to study in the area of asphalt binders and mix characterization, I would like to work on these topics, which may include:

- To correlate the relationship between the rheological properties of asphalt binder and performance of asphalt mix using Multivariate analysis (MVA) techniques.
- Further studies, to characterize the chemistry of binder composed of PET plastic and coffee husk ash and its interaction.
- Life cycle cost analysis of roads constructed using PET plastic and CHA in comparison to conventional asphalt binder HDM-4 Software.

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Appendix A – Materials Quality Test Results

Table A-1 Proportion of aggregate gradations result

Sieve size (mm)	% of Passing for each fraction			JMF	ERA Specification		Midpoint
	9.5-19mm	**4.75-9.5mm**	**0-4.75mm**		**LL**	**UL**	
25	23.0	27.0	50.0	100.0	100.0	100.0	100.0
19	15.1	27.0	50.0	92.1	90.0	100.0	95.0
9.5	1.5	22.1	49.7	73.4	56.0	80.0	68.0
4.75	0.3	2.9	48.9	52.1	35.0	65.0	50.0
2.36	0.2	0.5	37.2	37.9	23.0	49.0	36.0
0.3	0.2	0.3	15.3	15.8	5.0	19.0	12.0
0.075	0.1	0.3	2.7	3.1	2.0	8.0	5.0
% of Comp.	23	27	50	100			

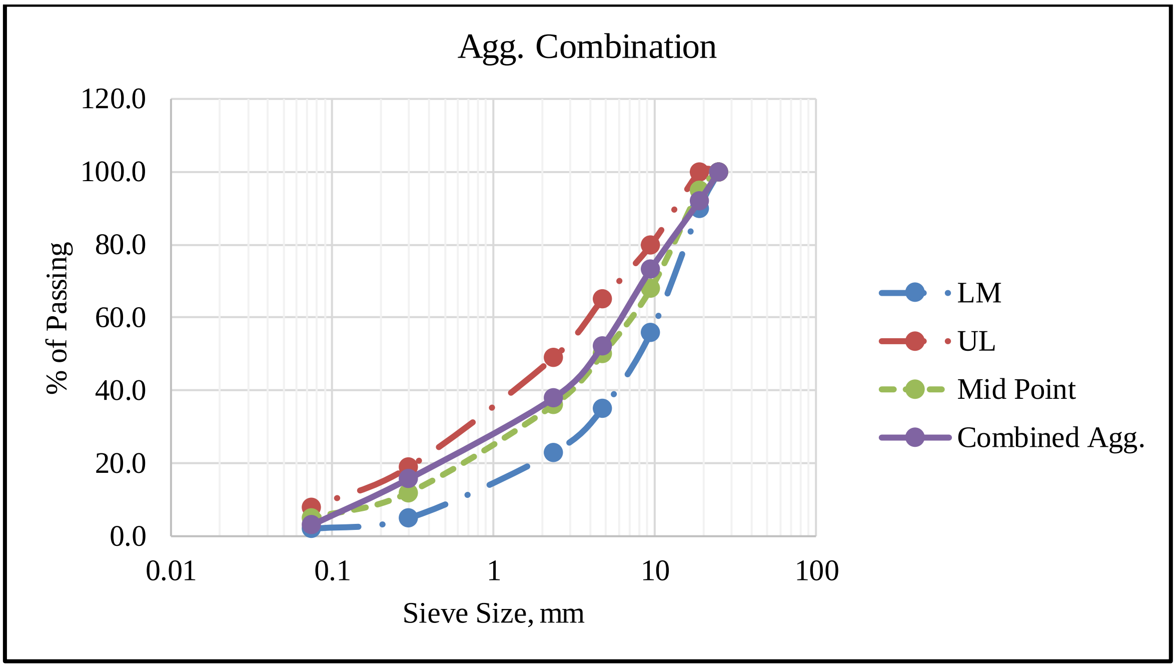


Figure A-1 Graphical representation of aggregate combination

Table A-2 Aggregate quality test results

Type of tests	Result	ERA Specification
Flakiness Index (%)	24.3	<35
Elongation Index (%)	23.9	N/A
ACV (%)	19.6	<25
AIV (%)	18.7	<25
10 % Fine aggregate (dry), KN	276.3	>160
LAA (%)	19.3	<30

Coarse Aggregate (9.5mm-19mm)		
Apparent Specific gravity	2.68	N/A
Bulk Specific gravity (SSD)	2.64	N/A
Bulk Specific gravity (oven dry)	2.62	N/A
Water Absorption (%)	0.85	<2
Intermediate Aggregate (4.74mm-9.5mm)		
Apparent Specific gravity	2.6	N/A
Bulk Specific gravity (SSD)	2.56	N/A
Bulk Specific gravity (oven dry)	2.53	N/A
Water Absorption (%)	1.11	<2
Fine Aggregate (0-4.74mm)		
Apparent Specific gravity	2.6	N/A
Bulk Specific gravity (SSD)	2.53	N/A
Bulk Specific gravity (oven dry)	2.49	N/A
Water Absorption (%)	1.73	<2

Table A-3 Bitumen quality test results

Type of tests	Result	ERA Specification
Penetration at $25^{0}C$	66.5	60-70
Ductility at $25^{0}C$ (cm)	100+	Min 50
Flash Point Cleveland Open Cup (^{0}C)	312	Min 232
Softening Point (^{0}C)	48.9	46-56
Specific Gravity at $25^{0}C$ (g/cm3)	1.024	0.97-1.06

Table A-4 Bitumen quality test results with a different percent of shredded plastic before and After Ageing

Before Ageing using RTFO Test				
Test Type/ % PET	0	3	6	9
Penetration Test, mm	66.5	59.7	49.3	45.5
Ductility Test, cm	142	134	97	85
Softening Point Test, ^{0}C	48.9	50.4	57.0	62.0
After Ageing using RTFO Test				
Test Type/ % PET	0	3	6	9
Penetration Test, mm	61.6	54.9	45.3	43.4
Ductility Test, cm	134	128	93	82
Softening Point Test, ^{0}C	51.5	53.9	59.8	63.3

Appendix B- Superpave Asphalt Physical Test Results.

B-1 Amplitude Sweep Test Results

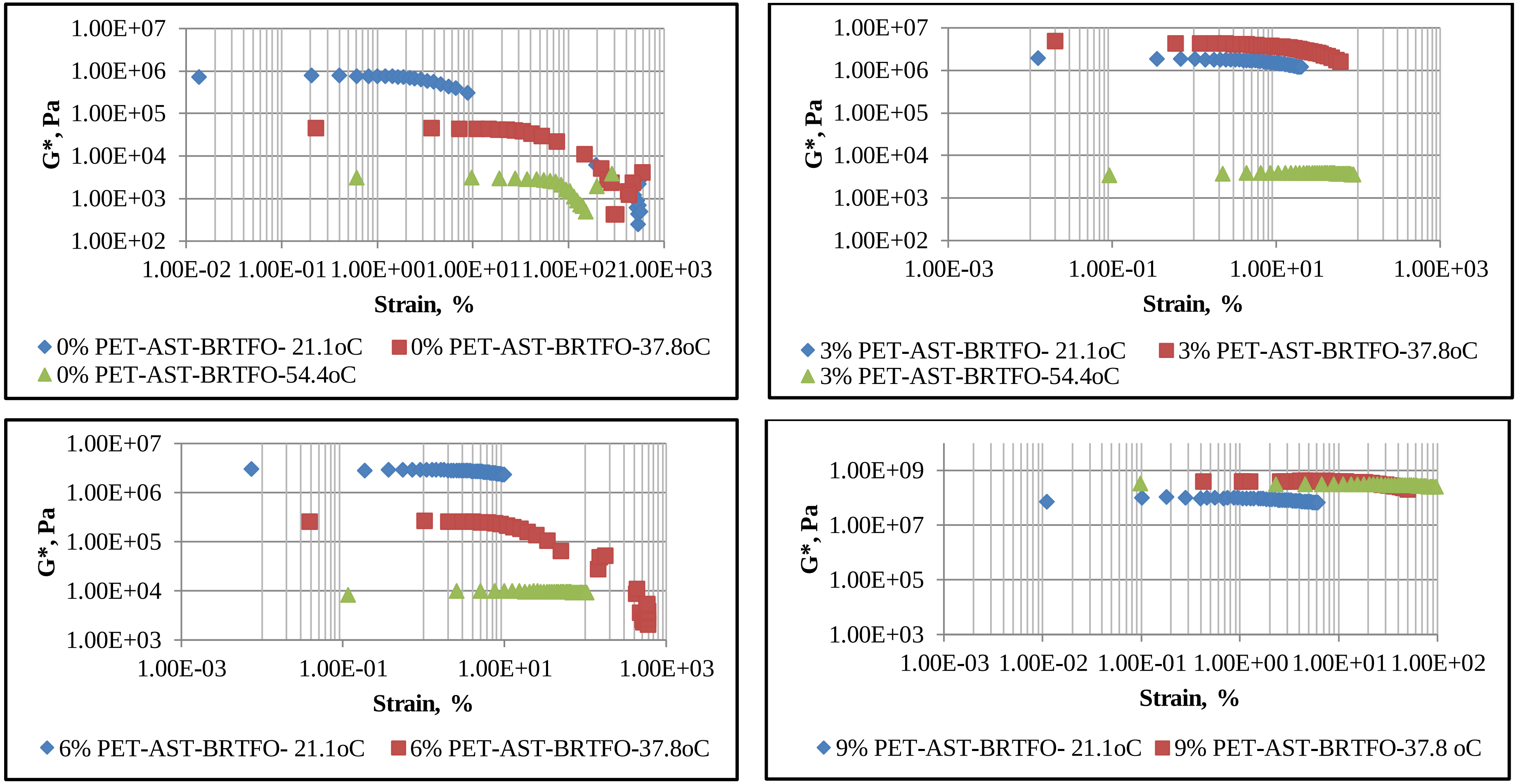


Figure B-1 AST Result for Before RTFO (Unaged Binder) with different Temperature

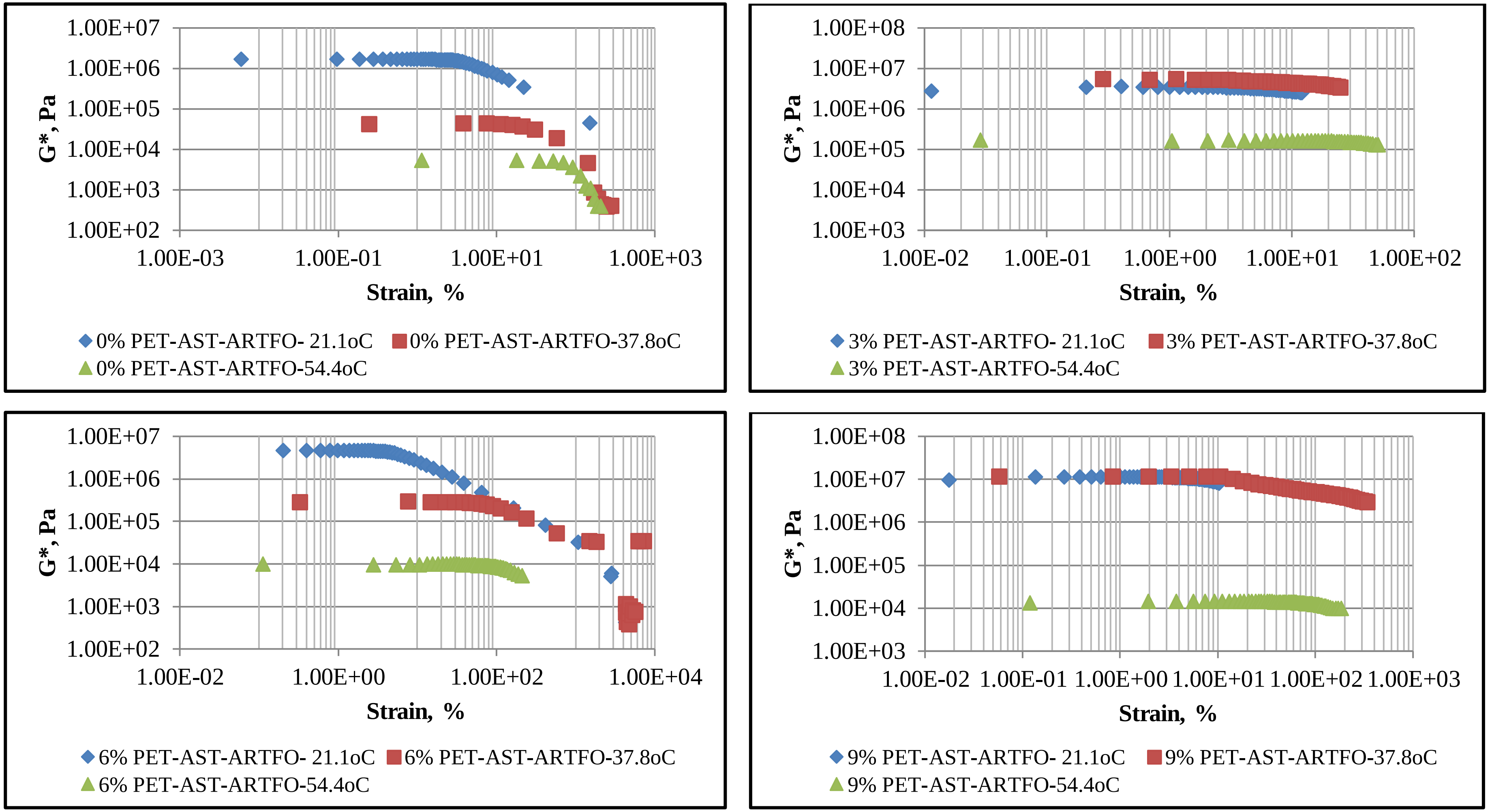


Figure B-2 AST Result for After RTFO (Aged Binder) with different Temperature

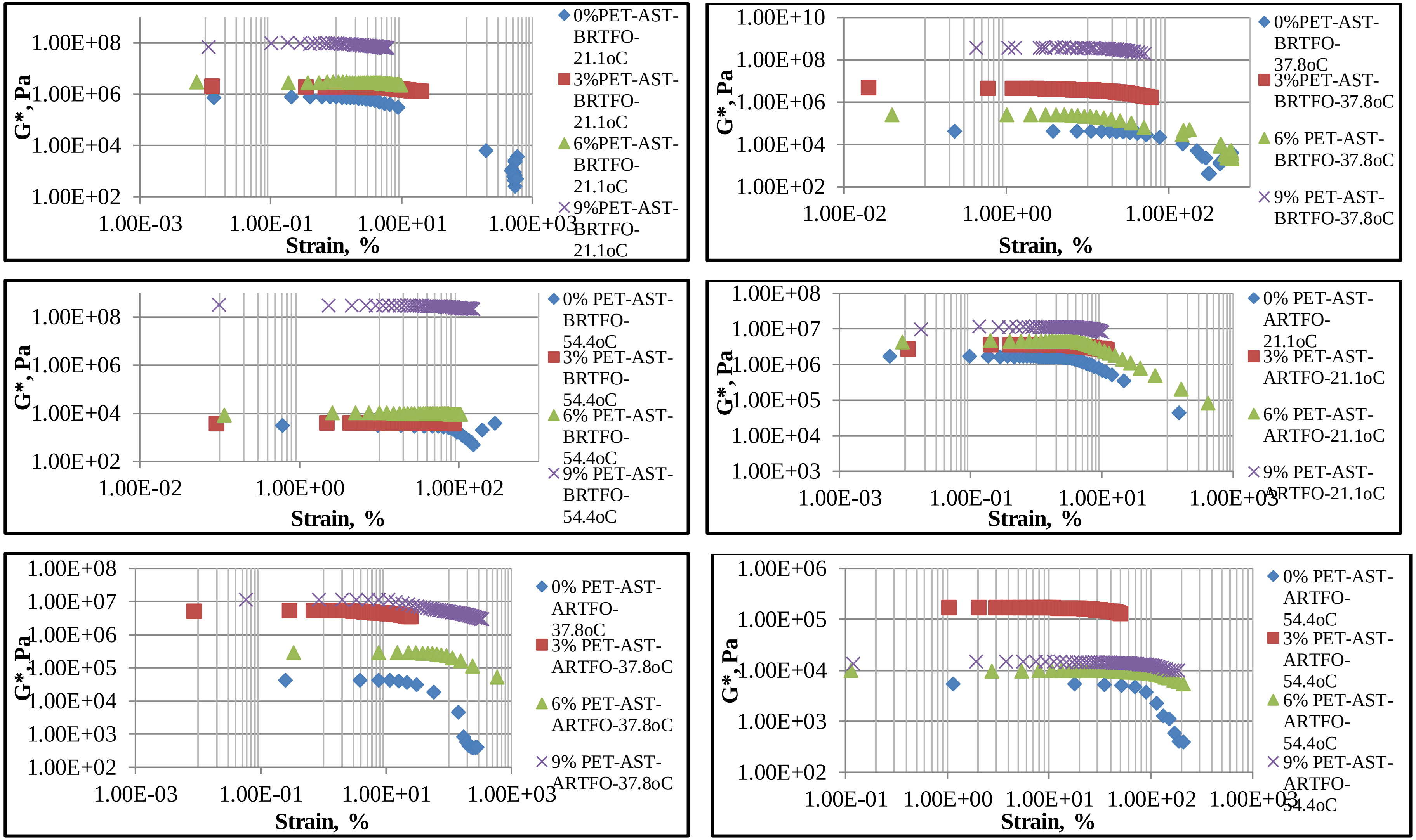


Figure B-3 AST Result for before and after RTFO with different PET plastic

B-2. Frequency Sweep Test (FST) result

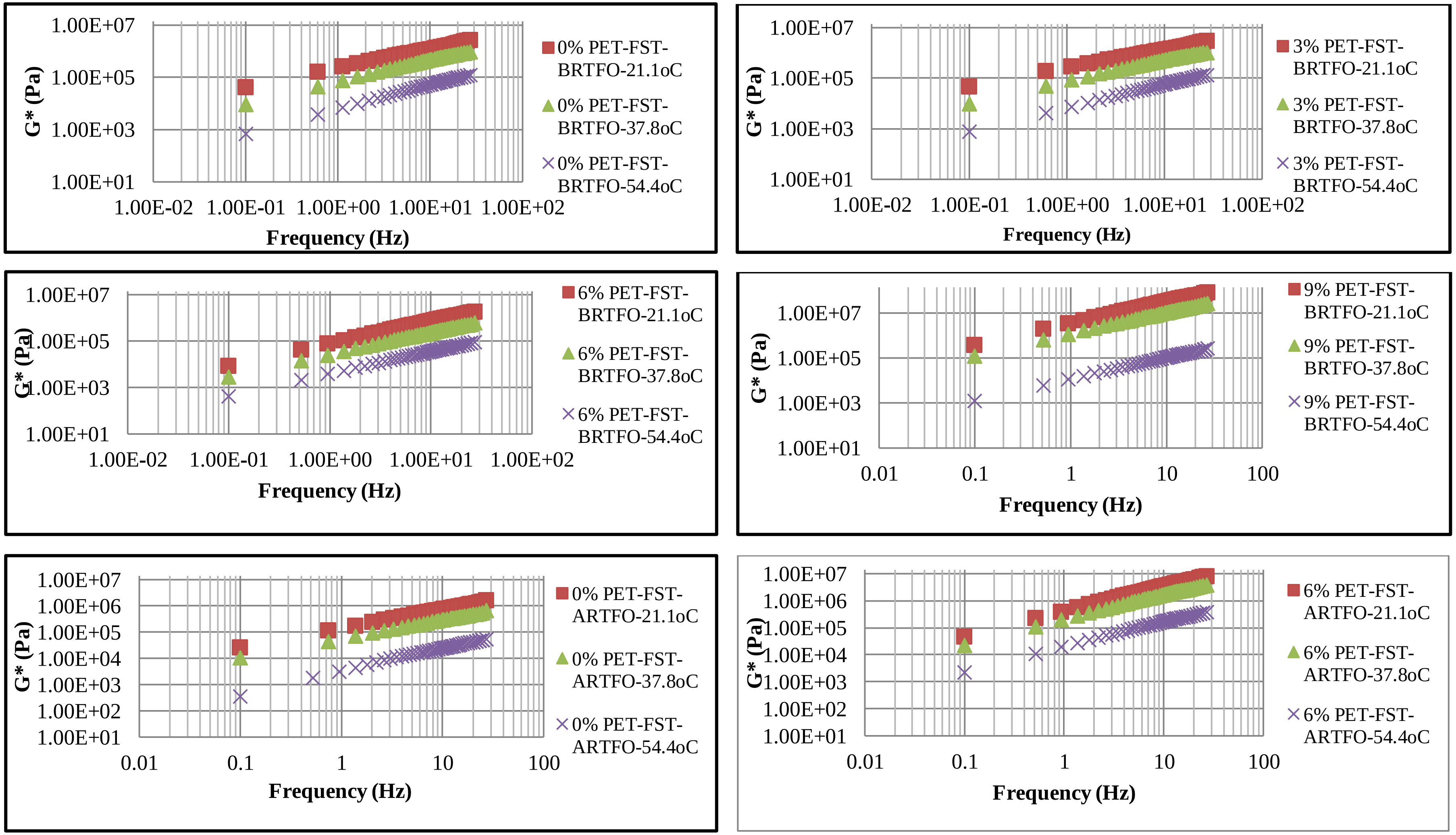


Figure B-4 Complex modulus versus frequency for unaged and aged binder with PET

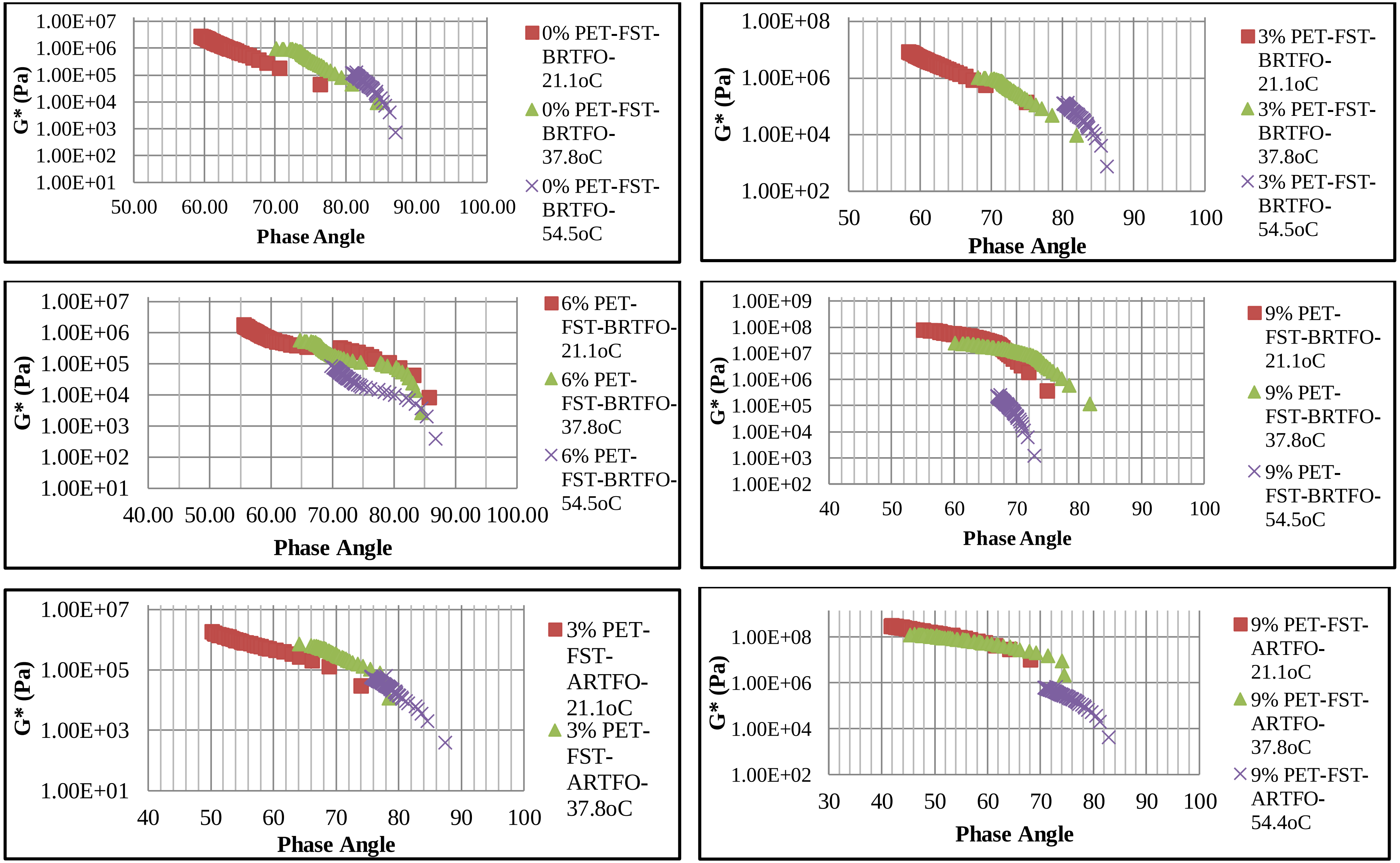


Figure B-5 Complex modulus versus Phase Angle for unaged and aged binder with PET

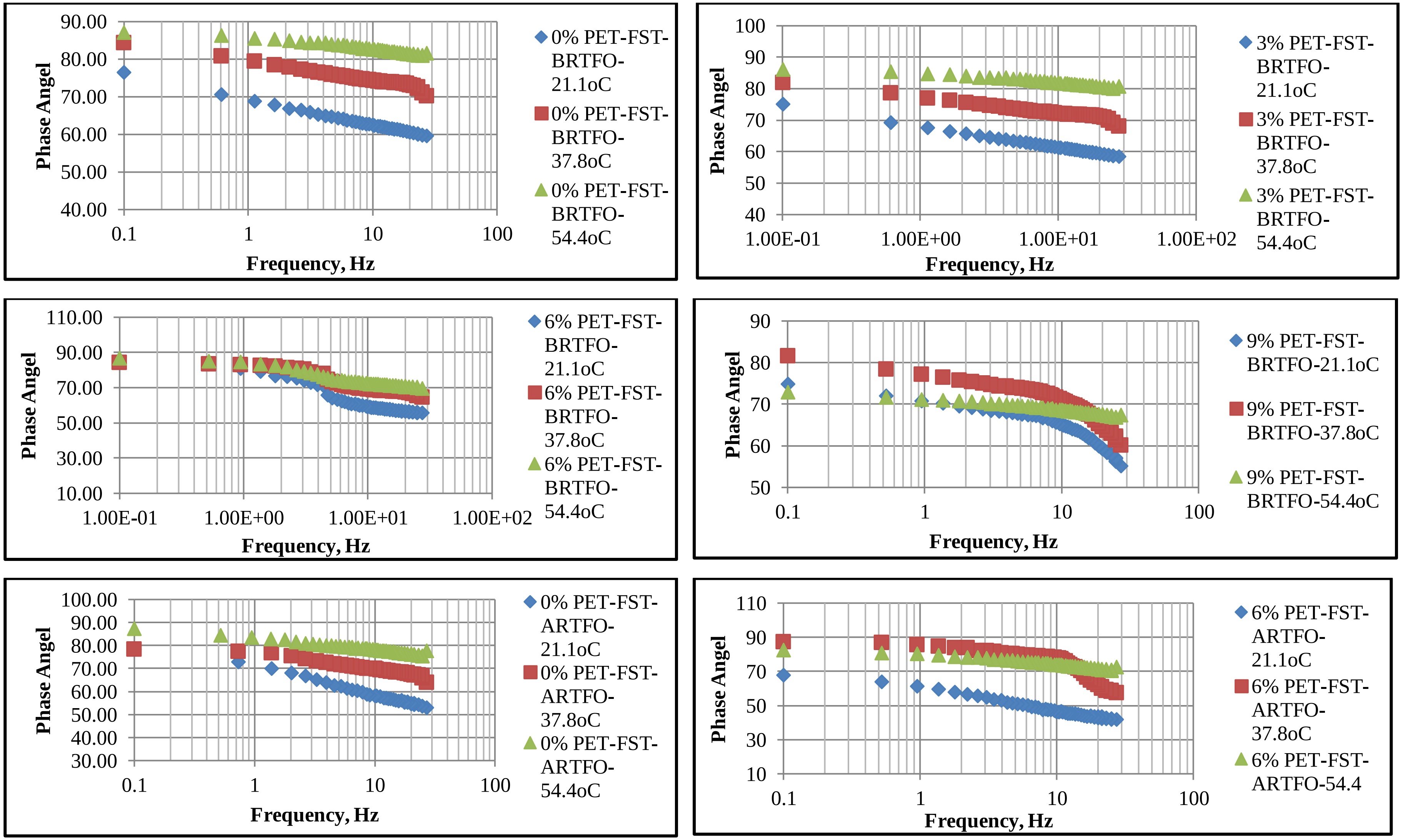


Figure B-6 Phase Angle versus Frequency for unaged and aged binder with PET

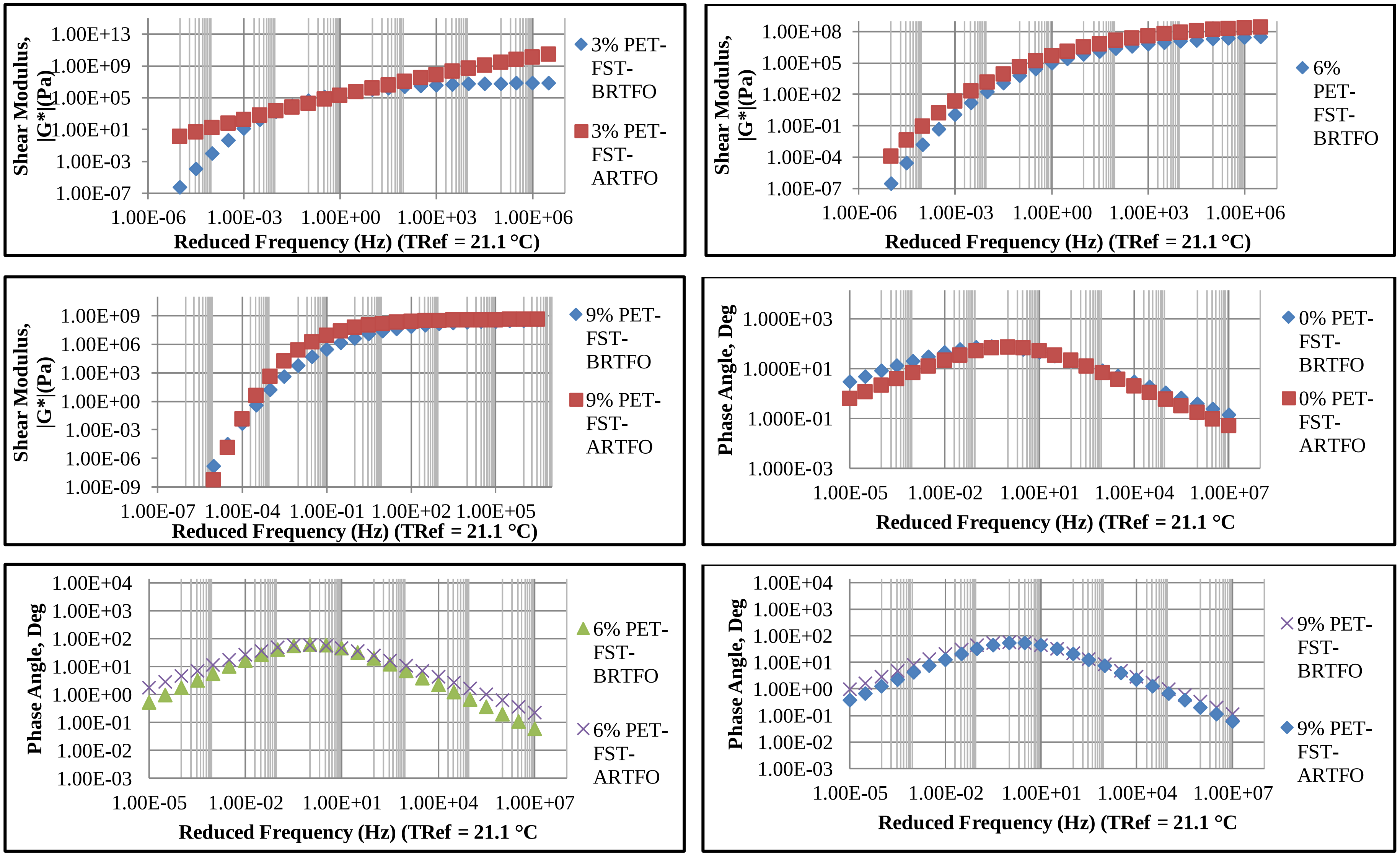
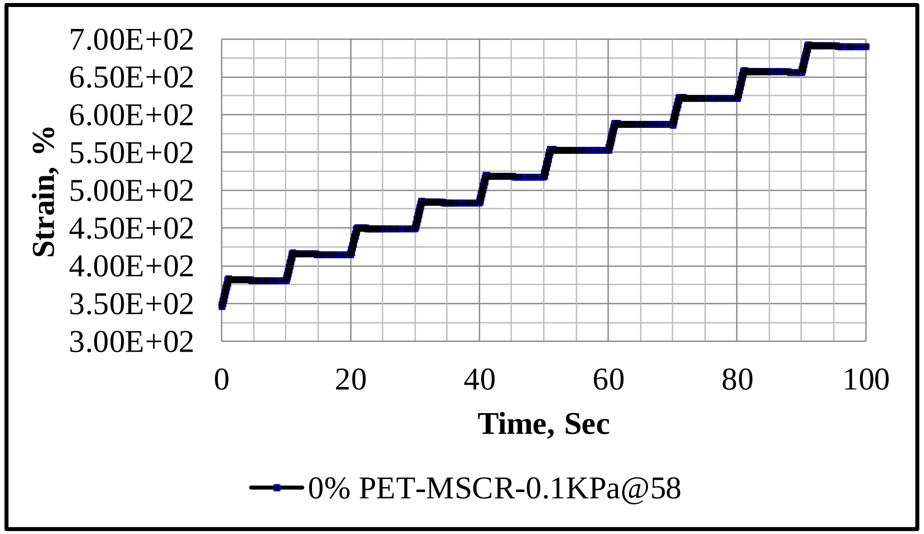
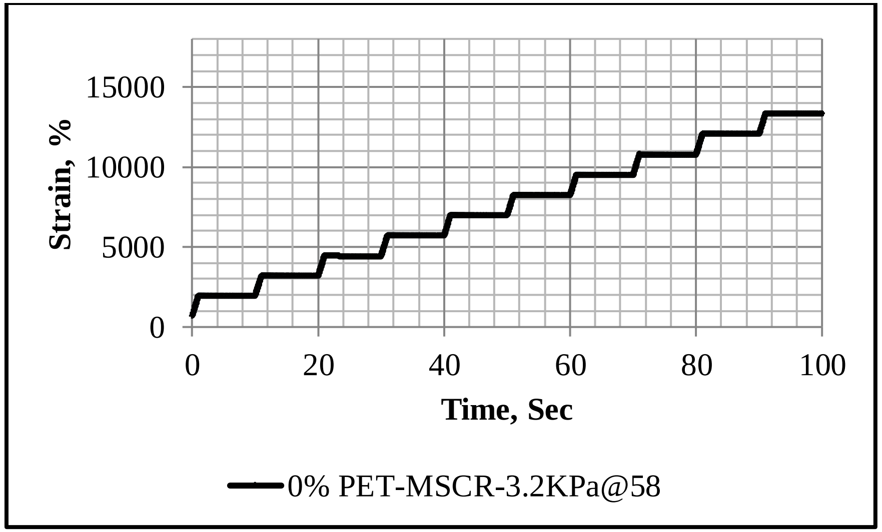
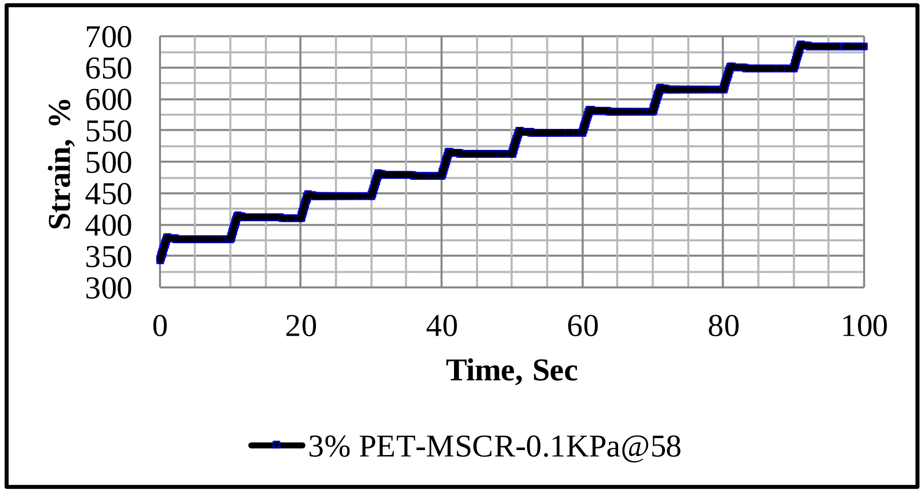


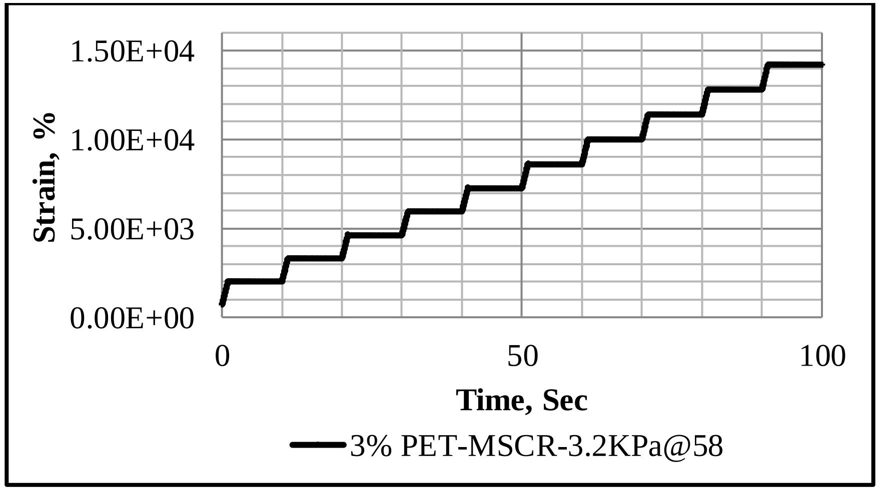
Figure B-7 Shear Modulus and Phase Angle Master Curve before and after RTFO with PET

B-3 Multiple Stress Creep Recovery (MCSR) Test









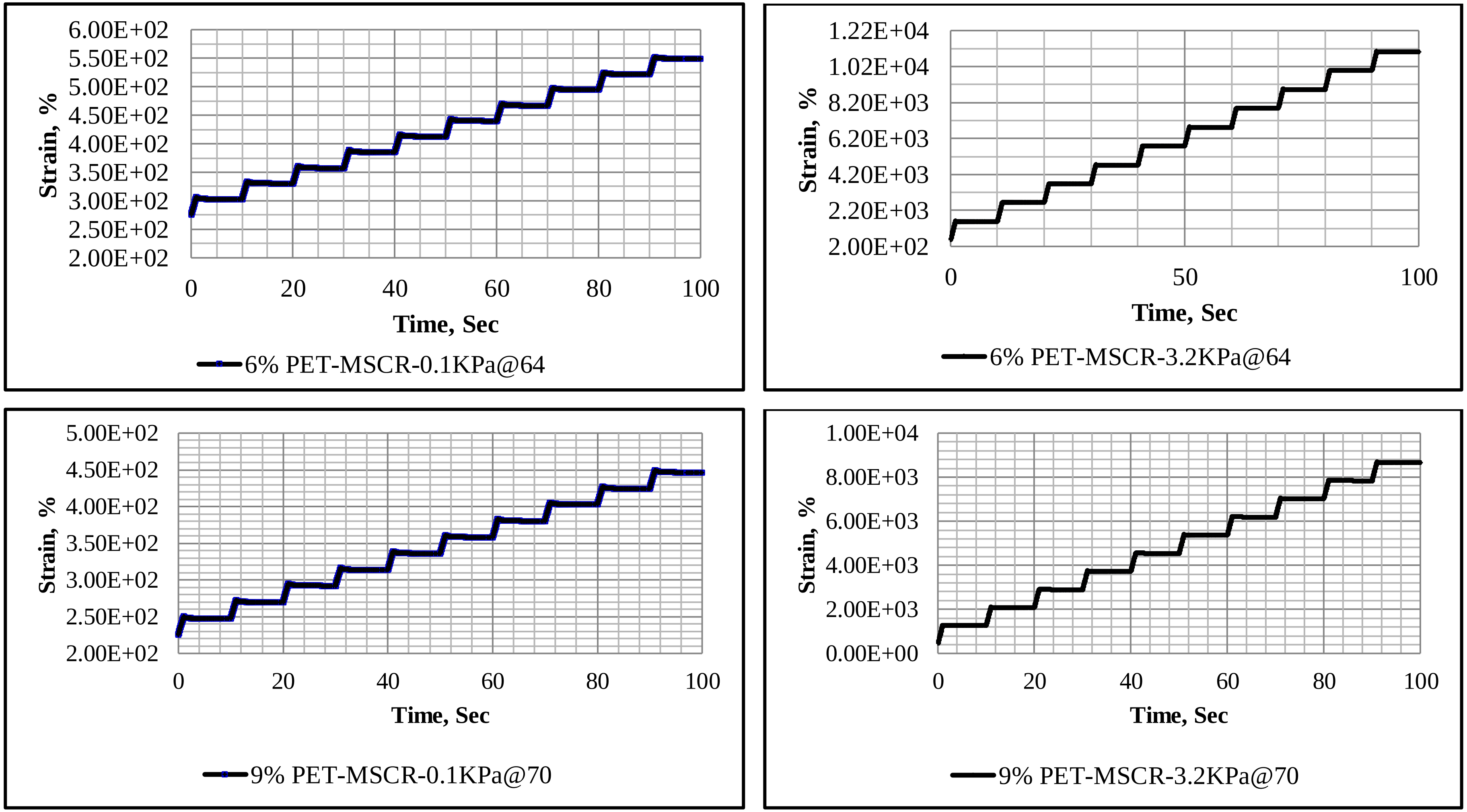


Figure B-8 MSCR test result for stress 0.1KPa and 3.2KPa at PG temperature with a different PET content.

Appendix C – Marshal Mix Test Results.

C-1 Design of aggregate gradation

Table C-1 Gradation of aggregate before blended

Sieve Size (mm)	% of Pass (Coarse Agg. 9.5-19mm)	% of Pass (Intermediate Agg. 4.75-9.5mm)	% of Pass (Fine Agg. 0-4.75mm)	Specification		Median
25	100.0	100.0	100.0	100	100	100.0
19	65.7	100.0	100.0	90	100	95.0
9.5	6.6	82.0	99.5	56	80	68.0
4.75	1.3	10.9	97.8	35	65	50.0
2.36	0.9	2.0	74.3	23	49	36.0
0.3	0.7	1.2	30.7	5	19	12.0
0.075	0.6	1.0	5.5	2	8	5.0

Table C-2 Gradation of aggregate after blended

Sieve size (mm)	**% of Passing for each fraction**			**JMF**	**ERA Specification**		**Midpoint**
	9.5-19mm	**4.75-9.5mm**	**0-4.75mm**		**LL**	**UL**	
25	23.0	27.0	50.0	100.0	100.0	100.0	100.0
19	15.1	27.0	50.0	92.1	90.0	100.0	95.0
9.5	1.5	22.1	49.7	73.4	56.0	80.0	68.0
4.75	0.3	2.9	48.9	52.1	35.0	65.0	50.0
2.36	0.2	0.5	37.2	37.9	23.0	49.0	36.0
0.3	0.2	0.3	15.3	15.8	5.0	19.0	12.0
0.075	0.1	0.3	2.7	3.1	2.0	8.0	5.0
% of Comp.	23	27	50	100			

Table C-3 Marshal mix preparation by weight of the total mixture

Preparation for Marshal Mix by Weight										
Sieve size (mm)	**% Passing of each Size of Aggregate.**			**% Retained of each Size of Aggregate.**			**Retained Weight on each fraction Size**			
	9.5-19mm	4.75-9.5mm	0-4.75mm	9.5-19mm	4.75-9.5mm	0-4.75mm	9.5-19mm	4.75-9.5mm	0-4.75mm	Total
25	100.0	100.0	100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	65.7	100.0	100.0	34.3	0.0	0.0	94.7	0.0	0.0	94.7
9.5	6.6	82.0	99.5	59.1	18.0	0.5	163.1	58.3	3.0	224.4
4.75	1.3	10.9	97.8	5.3	71.1	1.7	14.6	230.4	10.2	255.2
2.36	0.9	2.0	74.3	0.4	8.9	23.5	1.1	28.8	141.0	170.9
0.3	0.7	1.2	30.7	0.2	0.8	43.6	0.6	2.6	261.6	264.7
0.075	0.6	1.0	5.5	0.1	0.2	25.2	0.3	0.6	151.2	152.1
Pan				0.6	1.0	5.5	1.7	3.2	33.0	37.9
% of Comp.	23.0	27.0	50.0	100.0	100.0	100.0	276.0	324.0	600.0	1200.0
							1200.0			

C-2 Design bitumen content

Before conducted the different bitumen content first determined the design bitumen content for the selected blend of aggregates by testing specimens prepared at bitumen contents that span the expected design value. The expected design value is estimated from the following formula (MS-2 Asphalt Institute, 2014):

$$\mathbf{P = 0.035a + 0.045b + Kc + F} \text{..........Equation C-1}$$

Where:

DBC = approximate design bitumen content, percent by weight of the mix

a = percent of mineral aggregate retained on the 2.36mm sieve

b = percent of mineral aggregate passing the 2.36mm sieve and retained on the 0.075mm sieve

c = percent of mineral aggregate passing the 0.075mm sieve

K = 0.15 for 11-15% passing the 0.075mm sieve;
0.18 for 6-10% passing the 0.075mm sieve;
0.20 for 5% or less passing the 0.075mm sieve;

F = 0 - 2%. Based on absorption of bitumen; in the absence of other data, a value of 0.7 is suggested.

Table C-4 Calculation design bitumen contents

Sieve size	% of Passed	Parameters	
25	100.0	a =	54.2
19	92.1	b =	34.7
9.5	73.4	c =	3.2
4.75	52.1	K =	0.2
2.36	37.9	F =	0.7
0.3	15.8	DBC =	4.8
0.075	3.2	**DBC = 4.8%**	
Pan	0.0		

The aggregate samples are used to estimated optimum bitumen content (Design bitumen content, DBC) and at two increments of 0.5% above and below approximate design bitumen content (design bitumen content). Therefore, the design bitumen content was 4.8% and 0.5 percent above and below the DBC bitumen content for mix design was taken (i.e 3.8%, 4.3%, 4.8%, 5.3%, and 5.8%)

C.3 Marshal Mix design

Table C-5 Mix Proportion -Binder content as Percentage of Total Mix

Sieve size (mm)	Retained Weight on each fraction Size By Weight of Mixture																			
	3.8 % Bitumen Content				**4.3 % Bitumen Content**				**4.8 % Bitumen Content**				**5.3 % Bitumen Content**				**5.8 % Bitumen Content**			
	9.5-19	4.75-9.5	0-4.75	Total	9.5-19	4.75-9.5	0-4.75	Total	9.5-19	4.75-9.5	0-4.75	Total	9.5-19	4.75-9.5	0-4.75	Total	9.5-19	4.75-9.5	0-4.75	Total
25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19	91	0	0	91	91	0	0	91	90	0	0	90	90	0	0	90	89	0	0	89
9.5	157	56	3	216	156	56	3	215	155	56	3	214	154	55	3	212	154	55	3	211
4.75	14	222	10	245	14	220	10	244	14	219	10	243	14	218	10	242	14	217	10	240
2.36	1	28	136	164	1	28	135	164	1	27	134	163	1	27	133	162	1	27	133	161
0.3	1	2	252	255	1	2	250	253	1	2	249	252	1	2	248	251	1	2	246	249
0.075	0	1	145	146	0	1	145	146	0	1	144	145	0	1	143	144	0	1	142	143
Pan	2	3	32	36	2	3	32	36	2	3	31	36	2	3	31	36	2	3	31	36
Total	265	312	577	1154	264	310	574	1148	263	308	571	1142	261	307	568	1136	260	305	565	1130
Mix wt.	1200				1200				1200				1200				1200			

Table C-6 Retained Weight on each fraction size by Weight of Mixture

Retained Weight on each fraction size by Weight of Mixture						**Total wt. of Mixture =**		1200gm
Sieve Size (mm)	**Bitumen Content (%)**					**% of BC**	Wt. of BC	Wt. of Agg.
	3.8	**4.3**	**4.8**	**5.3**	**5.8**	**3.8**	46	1154
25	0	0	0	0	0	4	52	1148
19	91	91	90	90	89	5	58	1142
9.5	216	215	214	212	211	5	64	1136
4.75	245	244	243	242	240	6	70	1130
2.36	164	164	163	162	161	6	76	1124
0.3	255	253	252	251	249			
0.075	146	146	145	144	143			
Pan	36	36	36	36	36			
Wt. of Agg	1154	1148	1142	1136	1130			
Wt. of BC	46	52	58	64	70			
Total Wt. Mix	1200	1200	1200	1200	1200			

C.4 Marshal Test result

C.4.1. Marshal Test result with Stone Dust Filler

Table C-7 Asphalt Concrete Marshall Test Result used normal filler (stone dust filler)

Asphalt Concrete Marshall Test Result for Wearing Course (Stone dust filler)

Material	Asphalt Mixture	Asphalt Grade	60/70	Bulk S.G of Agg. Gsb (B)	2.53
Marshall Compaction	75 Blows each face	Specific Gravity of Binder, Gb (A)	1.024	Effective S.G of Agg. Gse	2.572
Specification	Dense Grade (20mm)	Load Factor (KN)	0.219175	Binder Absorption, Pba (%)	0.676

Sample No.	1	2	3	4	5	7	8	9	10	12	13	14	15	16	17	18
	AC (%), Pb	Spec. height (mm)	Specimen Weight (gm)			Density (g/cm3)				Void Properties			Stability (KN)			Flow (mm)
			In Air	In Water	SSD in Air	Apparent	SSD	Bulk S.G of Specimen (Gmb)	Theo. Max. S.G (Gmm)	VTM (%)	VMA (%)	VFA (%)	Max. Dial Reading	Corr. Factor	Adjusted (KN)	
						(3/3-4)	(5/5-4)	(3/5-4)		(1-9/10)*100	100-(9*Ps)/B	(13-12/13)			(15*16)	
A	3.8	66	1178	649	1181	2.23	2.22	2.21	2.433	8.69	15.50	43.93	93	0.94	19.16	2.1
B		66	1181	652	1183	2.23	2.23	2.22					95	0.94	19.57	2.9
C		66	1180	654	1184	2.24	2.23	2.23					92	0.94	18.95	3.6
Average		66	1180	652	1183	2.23	2.23	2.22	2.433	8.69	15.50	43.93	93.33	0.94	19.23	2.87

A		66	1182	658	1186	2.26	2.25	2.24					94	0.94	19.37	3.4
B	4.3	66	1185	657	1189	2.24	2.23	2.23	2.416	7.42	15.38	51.78	97	0.94	19.98	3.2
C		66	1182	658	1185	2.26	2.25	2.24					96	0.94	19.78	2.8
Average		66	1183	658	1187	2.25	2.24	2.24	2.42	7.42	15.38	51.78	95.67	0.94	19.71	3.13
A		66	1176	657	1179	2.27	2.26	2.25					101	0.94	20.81	3.4
B	4.8	64	1173	658	1178	2.28	2.27	2.26	2.398	5.91	15.07	60.79	102	0.99	22.13	3.2
C		65	1178	661	1182	2.28	2.27	2.26					100	0.96	21.04	2.9
Average		65	1176	659	1180	2.27	2.26	2.26	2.398	5.91	15.07	60.79	101.00	0.96	21.33	3.17
A		65	1165	655	1169	2.28	2.27	2.27					104	0.96	21.88	3.2
B	5.3	65	1163	656	1168	2.29	2.28	2.27	2.381	4.83	15.15	68.15	102	0.96	21.46	4
C		64	1164	654	1169	2.28	2.27	2.26					105	0.99	22.78	4.1
Average		64.7	1164	655	1169	2.29	2.28	2.27	2.381	4.83	15.15	68.15	103.67	0.97	22.04	3.77
A		63	1167	659	1172	2.30	2.28	2.27					85	1.01	18.82	4.8
B	5.8	64	1165	657	1169	2.29	2.28	2.28	2.364	3.64	15.16	75.98	84	0.99	18.23	5.2
C		64	1167	662	1173	2.31	2.30	2.28					87	0.99	18.88	2.4
Average		63.7	1166	659	1171	2.30	2.29	2.28	2.364	3.64	15.16	75.98	85.33	1.00	18.64	4.13

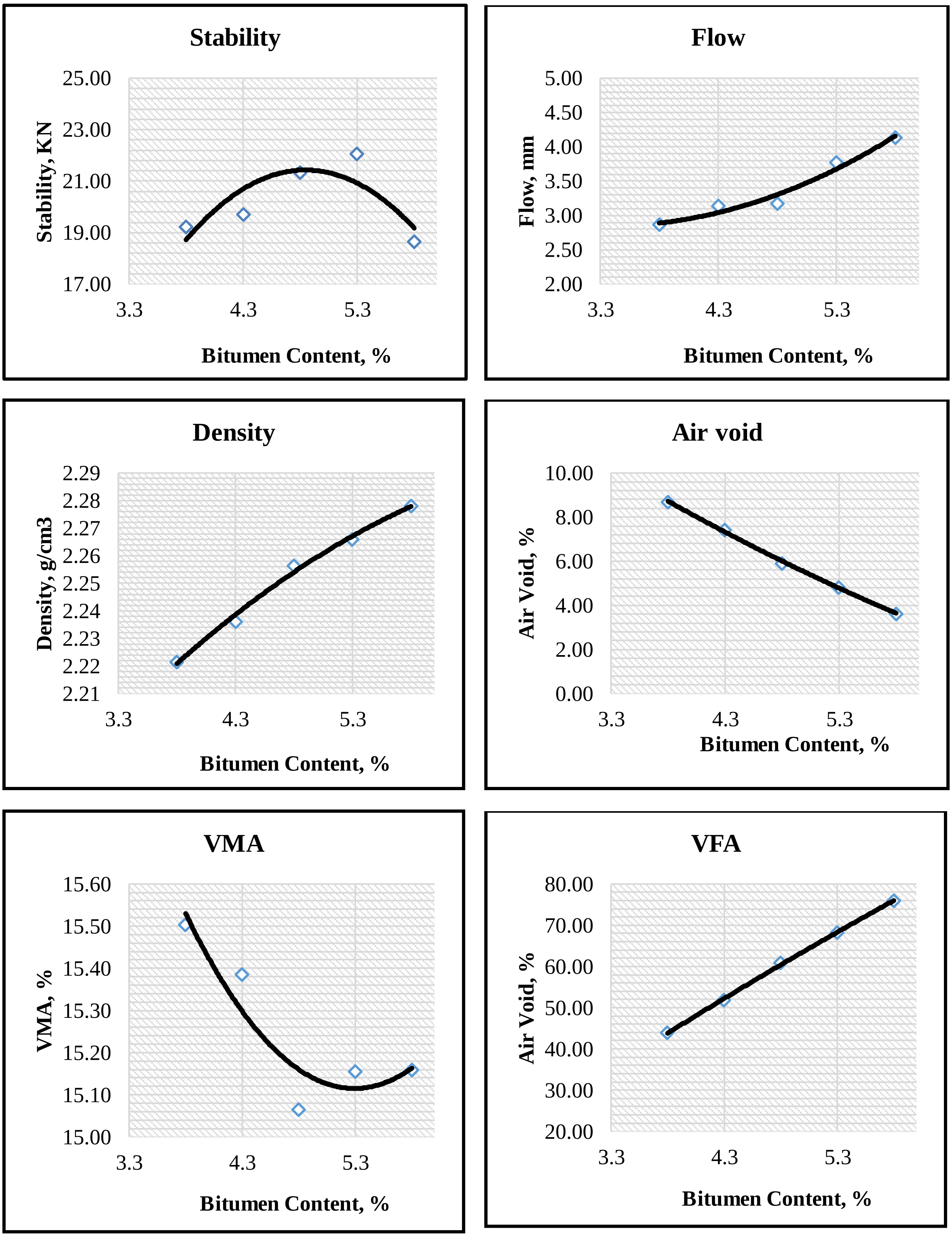


Figure C-8 Graphical representation of mix test properties SD Filler

C.4.2. Marshal Test result with Coffee Husk Ash Filler

Table C-8 Asphalt Concrete Marshall Test Result used Coffee Husk Ash Filler

Asphalt Concrete Marshall Test Result for Wearing Course (Coffee husk ash filler)

Material	Asphalt Mixture	Asphalt Grade	60/70	Bulk S.G of Agg. Gsb (B)	2.53
Marshall Compaction	75 Blows each face	Specific Gravity of Binder, Gb (A)	1.024	Effective S.G of Agg. Gse	2.571
Specification	Dense Grade (20mm)	Load Factor (KN)	0.219175	Binder Absorption, Pba (%)	0.661

Sample No.	1	2	3	4	5	7	8	9	10	12	13	14	15	16	17	18
	AC (%), Pb	Spec. height (mm)	Specimen Weight (gm)			Density (g/cm3)				Void Properties			Stability (KN)			Flow (mm)
			In Air	In Water	SSD in Air	Apparent	SSD	Bulk S.G of Specimen (Gmb)	Theo. Max. S.G (Gmm)	VTM (%)	VMA (%)	VFA (%)	Max. Dial Reading	Corr. Factor	Adjusted (KN)	
						(3/3-4)	(5/5-4)	(3/5-4)		(1-9/10)*100	100-(9*Ps)/B	(13-12/13)			(15*16)	
A	3.8	64	1098	641	1180	2.40	2.19	2.04	2.432	12.39	18.97	34.70	50	0.99	10.85	2.8
B		65	1185	640	1187	2.17	2.17	2.17					56	0.96	11.78	2.9
C		65	1177	643	1181	2.20	2.20	2.19					48	0.96	10.10	2.6
Average		65	1153	641	1183	2.26	2.18	2.13	2.432	12.39	18.97	34.70	51.33	0.97	10.91	2.77
A	4.3	63	1083	607	1084	2.28	2.27	2.27	2.414	7.79	15.76	50.59	50	1.01	11.07	3.3
B		64	1172	638	1175	2.19	2.19	2.18					56	0.99	12.15	2.9
C		65	1191	658	1193	2.23	2.23	2.23					60	0.96	12.62	2.8
Average		64	1149	634	1151	2.23	2.23	2.23	2.41	7.79	15.76	50.59	55.33	0.99	11.95	3.00

A	4.8	63	1064	592	1066	2.25	2.25	2.24	2.397	6.31	15.46	59.17	65	1.01	14.39	3.3
B		64	1177	653	1180	2.25	2.24	2.23					67	0.99	14.54	3.2
C		64	1182	662	1185	2.27	2.27	2.26					60	0.99	13.02	3
Average		64	1141	636	1144	2.26	2.25	2.25	2.397	6.31	15.46	59.17	64.00	1.00	13.98	3.17
A	5.3	63	1157	648	1160	2.27	2.27	2.26	2.381	4.44	14.83	70.04	65	1.01	14.39	3.6
B		62	1169	667	1170	2.33	2.33	2.32					70	1.04	15.96	3.2
C		63	1154	643	1158	2.26	2.25	2.24					60	1.01	13.28	3.3
Average		63	1160	653	1163	2.29	2.28	2.27	2.381	4.44	14.83	70.04	65.00	1.02	14.54	3.37
A	5.8	62	1163	659	1164	2.31	2.30	2.30	2.364	3.43	14.97	77.11	65	1.04	14.82	3.5
B		63	1156	650	1157	2.28	2.28	2.28					62	1.01	13.72	3.6
C		63	1158	648	1159	2.27	2.27	2.27					59	1.01	13.06	3.7
Average		63	1159	652	1160	2.29	2.29	2.28	2.364	3.43	14.97	77.11	62.00	1.02	13.87	3.60

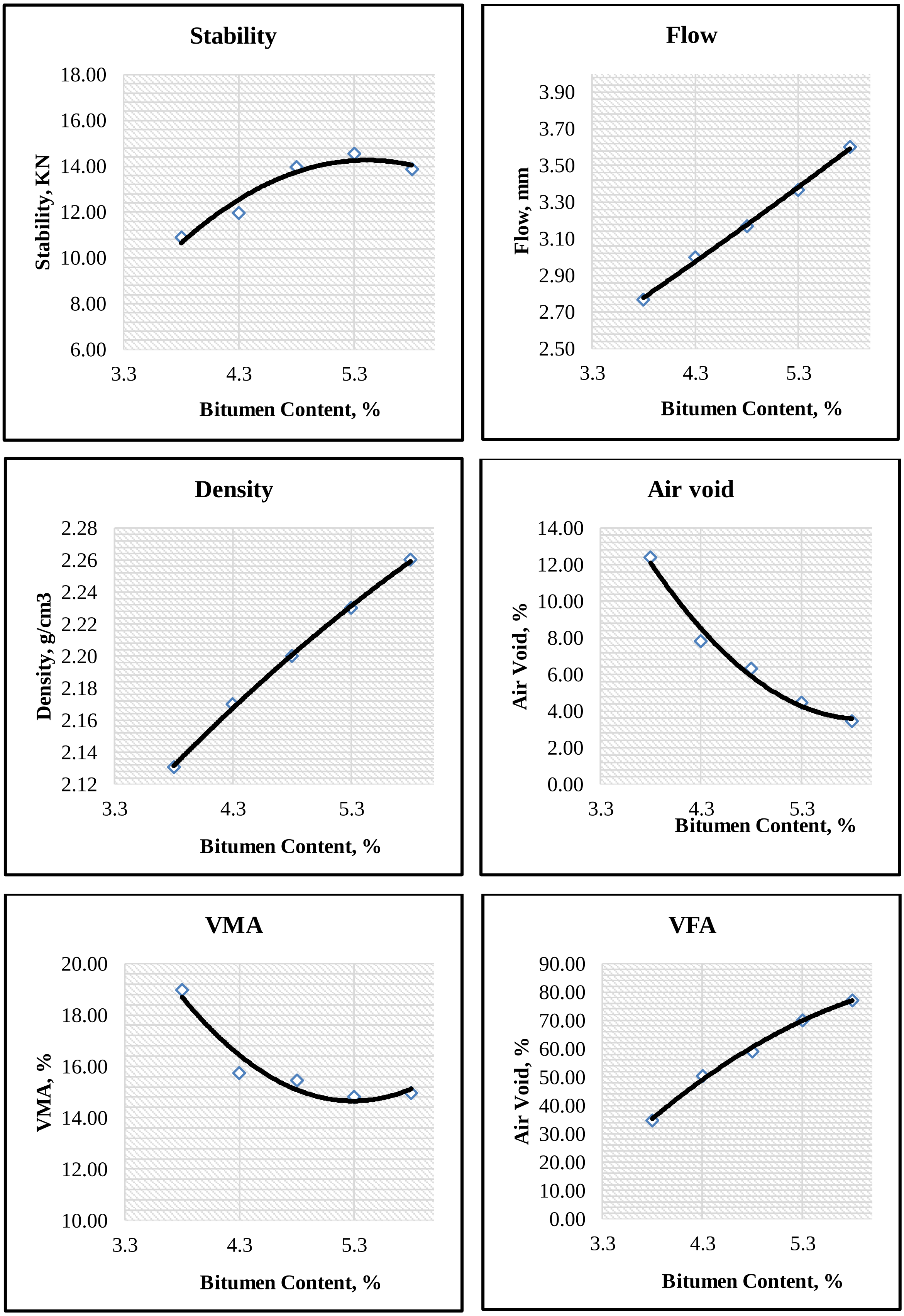


Figure C-9 Graphical representation of mix test properties with CHA Filler

C.4.3. Marshal Test result containing PET plastic with Coffee Husk Ash Filler

Table C-9 Asphalt Concrete Marshall Test Result Containing PET plastic with Coffee Husk Ash Filler

Asphalt Concrete Marshall Test Result for Wearing Course (Different % of PET plastic with coffee husk filler)

Material	Asphalt Mixture	Asphalt Grade	60/70	Bulk S.G of Agg. Gsb (B)	2.53
Marshall Compaction	75 Blows each face	Specific Gravity of Binder, Gb (A)	1.024	Effective S.G of Agg. Gse	2.557
Specification	Dense Grade (20mm)	Load Factor (KN)	0.219175	Binder Absorption, Pba (%)	0.438

OBC = 5.4% (66gm)

Sample No.	% of PET plastic by wt. of OBC	1	2	3	4	5	7	8	9	10	12	13	14	15	16	17	18
		OBC (%), Pb by wt of agg	Spec. height (mm)	Specimen Weight (gm)			Density (g/cm3)				Void Properties			Stability (KN)			Flow (mm)
				In Air	In Water	SSD in Air	Apparent	SSD	Bulk S.G of Specimen (Gmb)	Theo. Max. S.G (Gmm)	VTM (%)	VMA (%)	VFA (%)	Max. Dial Reading	Corr. Factor	Adjusted (KN)	
							(3/3-4)	(5/5-4)	(3/5-4)		(1-9/10)*100	100-(9*Ps)/B	(13-12/13)			(15*16)	
A	0	5.40	63	1145	657	1162	2.35	2.30	2.27	2.347	3.90	15.62	75.06	70	1.01	15.50	3
B			63	1142	653	1165	2.34	2.28	2.23					65	1.01	14.39	3.5
C			64	1135	650	1150	2.34	2.30	2.27					60	0.99	13.02	4
Average			63	1141	653	1159	2.34	2.29	2.256	2.347	3.90	15.62	75.06	65.00	1.00	14.30	3.50

A			65	1185	651	1178	2.22	2.24	2.25					59	0.96	12.41	3.7
B	3	5.25	64	1187	659	1188	2.25	2.25	2.24	2.342	3.87	15.66	75.29	75	0.99	16.27	2.9
C			64	1192	657	1184	2.23	2.25	2.26					65	0.99	14.10	3.5
Average			64	1188	656	1183	2.23	2.24	2.251	2.342	3.87	15.66	75.29	66.33	0.98	14.26	3.37
A			62	1190	647	1170	2.19	2.24	2.28					77	1.04	17.55	2.5
B	6	5.08	65	1180	640	1163	2.19	2.22	2.26	2.359	3.94	14.98	73.69	65	0.96	13.68	3.5
C			63	1187	642	1166	2.18	2.23	2.27					70	1.01	15.50	3.1
Average			63	1186	643	1166	2.18	2.23	2.266	2.359	3.94	14.98	73.69	70.67	1.00	15.57	3.03
A			65	1195	639	1163	2.15	2.22	2.28					80	0.96	16.83	2.8
B	9	4.92	63	1205	647	1175	2.16	2.23	2.28	2.375	4.17	14.42	71.12	85	1.01	18.82	2.5
C			65	1190	635	1160	2.14	2.21	2.27					70	0.96	14.73	3.0
Average			64	1197	640	1166	2.15	2.22	2.276	2.375	4.17	14.42	71.12	78.33	0.98	16.79	2.77
A			64	1187	625	1150	2.11	2.19	2.26					58	0.99	12.59	3.3
B	12	4.75	65	1185	635	1150	2.15	2.23	2.30	2.387	4.47	14.14	68.39	68	0.96	14.31	2.7
C			62	1180	630	1148	2.15	2.22	2.28					65	1.04	14.82	2.9
Average			64	1184	630	1149	2.14	2.21	2.280	2.387	4.47	14.14	68.39	63.67	1.00	13.90	2.97

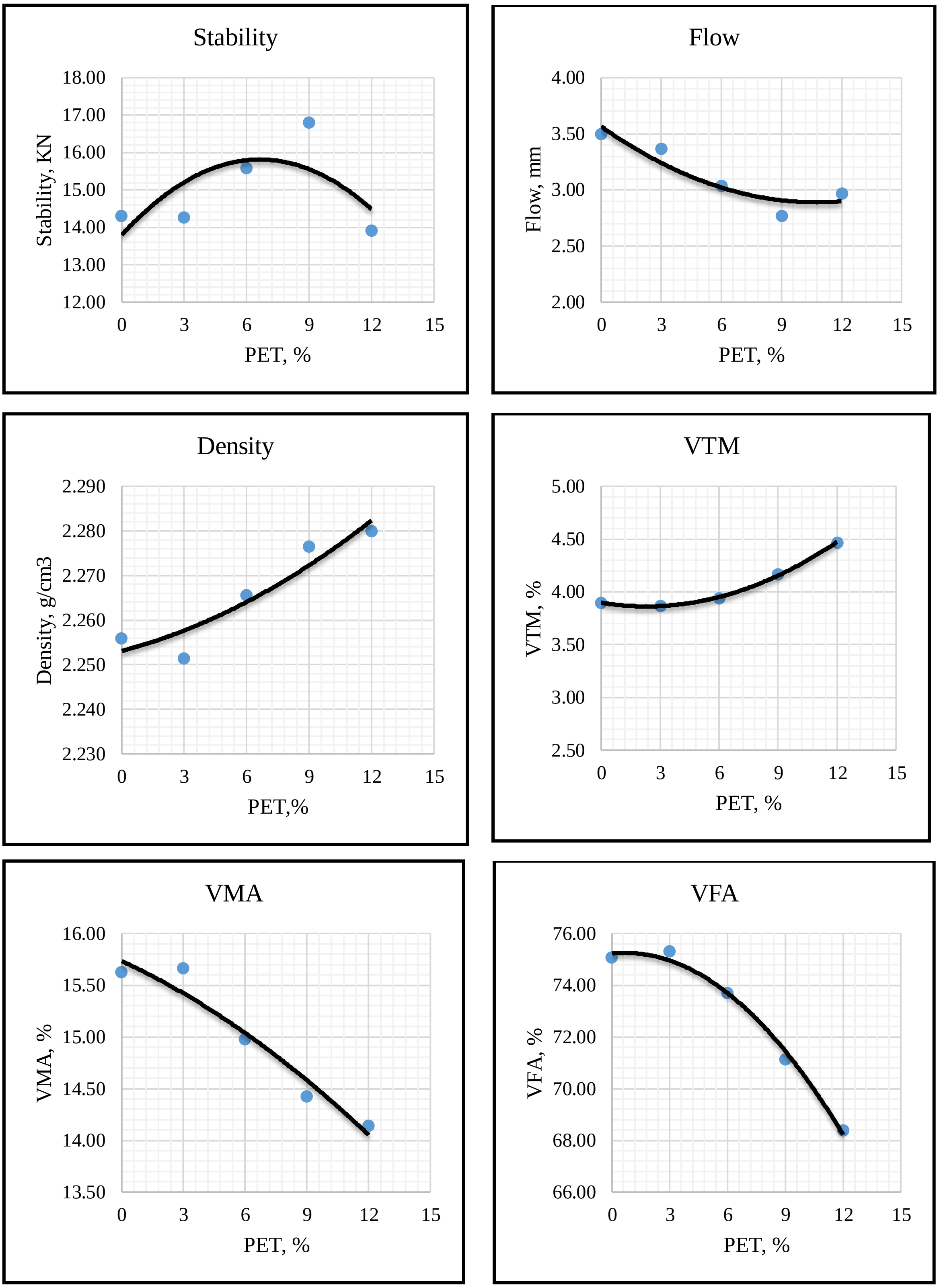


Figure C-10 Graphical representation of mix test properties containing PET plastic with CHA Filler

C.5 Calculation of volumetric composition

C.5.1 Bulk Specific Gravity of total aggregate (G_{sb})

$$G_{sb} = \frac{P_1 + P_2 + P_3}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \frac{P_3}{G_3}}$$

Where Gsb = Bulk Specific Gravity Combined Agg

G1, G2, G3 = Specific gravity value for fraction 1, 2, 3

P1, P2, P3, = Weight percentage of a fraction

$$G_{sb} = \frac{23+27+50}{\frac{23}{2.62}+\frac{27}{2.53}+\frac{50}{2.49}} = 2.53$$

C.5.2 Maximum specific gravity of mixtures with different bitumen contents

$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}}$$

Where: Gse = effective specific gravity of aggregate;

Gmm = maximum specific gravity of mixed material (no air voids);

Pb = percent of bitumen content by total weight of mixture;

Gb = specific gravity of bitumen

First Calculate the Gmm value from the laboratory using a pycnometer

$$G_{mm} = \frac{B}{B + A - C}$$

Where. A = Weight of bottle + water (gm)

B = Weight of sample (500gm)

C= Weight of bottle +water + sample

Table C-10 Maximum specific gravity of mixtures contents effective and effective specific gravity of aggregate of combined aggregate with different bitumen, Filler and PET plastic contents

Marshal Mix with Stone Dust Filler					
%BC, %	3.8	4.3	4.8	5.3	5.8
Maximum specific gravity of paved mix, Gmm	2.433	2.416	2.398	2.381	2.364
The effective specific gravity of combined aggregate, Gse	2.573	2.573	2.572	2.572	2.571
Marshal Mix with Coffee Husk Ash Filler					
%BC, %	3.8	4.3	4.8	5.3	5.8
Maximum specific gravity of paved mix, Gmm	2.432	2.414	2.397	2.381	2.364
The effective specific gravity of combined aggregate, Gse	2.571	2.571	2.571	2.571	2.571
Marshal Mix with PET plastic + Coffee Husk Ash Filler					
%BC, %	0	3	6	9	12
Maximum specific gravity of paved mix, Gmm	2.347	2.342	2.359	2.375	2.387
Effective specific gravity of combined agg, Gse	2.534	2.528	2.548	2.569	2.583

C.5.3. Bitumen absorption

Bitumen absorption is expressed as a percentage by weight of aggregate and is calculated using:

$$P_{ba} = \frac{100(G_{se} - G_{sb})G_b}{G_{se} * G_{sb}}$$

Where, Pba = absorbed bitumen, percent by weight of aggregate, Gse = effective specific gravity of aggregate, Gsb = bulk specific gravity of total aggregate, and Gb = specific gravity of bitumen

$$P_{ba} = \frac{100*(2.57-2.53)*1.024}{2.57*2.53} = 0.676\%$$

C.5.4. Percent voids in mineral aggregate (VMA)

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

Where: VMA = voids in mineral aggregate

Gmb = bulk specific gravity of compacted mix

Gsb = bulk specific gravity of total aggregate

Ps = aggregate content, percent by total weight of the mix

C.5.5. Percent air voids in a compacted mix (VIM)

$$VIM = 100 * \left[\frac{G_{mm} - G_{mb}}{G_{mm}}\right]$$

Where: VIM = air voids in the compacted mix, percent of the total volume

Gmm= maximum specific gravity of mix

Gmb = bulk specific gravity of compacted mix

C.5.6. Percent voids filled with bitumen (VFB) in a compacted mix

$$VFA = 100 * \left[\frac{VMA - VIM}{VMA}\right]$$

Where: VFB = voids filled with bitumen (percent of VMA)

VMA = voids in mineral aggregate, percent of bulk volume

VIM = air voids in the compacted mix, percent of the total volume