Performance Evaluation of Waste Polymers Modified HMA using Superpave Mix Design

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Certified that the contents and form of thesis titled "**Performance Evaluation of Waste Polymers Modified HMA using Superpave Mix Design**" submitted by **Muhammad Sohail Saleh** have been found satisfactory for the requirement of **Master of Science degree**.

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DEDICATED

I dedicate this thesis to my loving Parents,
Dearest Sisters, Lovely Wife, Heartiest
Grandmother (late), Thesis Supervisor
and the people who have always been
there to support, congratulate, motivate
and show me the best path to follow.
To My Parents, I will never finish Thank You
for everything you do every day for me.

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In the end, I express my solemn gratitude with earnest sense of reverence to My Family for their encouragement, heartfelt prayers and kind wishes for successful completion of my studies along with this research work. Specially, I want to thank my Father from all my heart for being with me always and for motivating me at every point of my life. He is the Best Father in the world and without him nothing would have been possible.

Engr. Muhammad Sohail Saleh

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LIST OF ABBREVIATIONS

AASHTO – American Association of State Highway & Transportation Official

AC – Asphalt Concrete

ANOVA – Analysis of Variance

ESAL – Equivalent Single-Axle Loads

G_{mb} – Mix Bulk Specific Gravity

G_{mm} – Mix Theoretical Maximum Specific Gravity

HMA – Hot-Mix Asphalt

IDT – Indirect Tension Test

ITS – Indirect Tensile Strength

JMF – Job Mix Formula

LVDTs – Linear Variable Differential Transducers

MDL – Maximum Density Line

NCHRP – National Cooperative Highway Research Program

N_{des} – Number of Gyrations at Design Level

N_{ini} – Number of Gyrations at Initial Compaction

NMAS – Nominal Maximum Aggregate Size

$P_{0.075}$ – Percent Passing the 0.075 mm Sieve

P_{be} – Percent of Effective Binder

RZ – Restricted Zone

TMD – Theoretical Maximum Density

TSR – Tensile Strength Ratio

UTM – Universal Testing Machine

ARL – Attock Refinery Limited

ASTM – American Society for Testing and Materials

MEPDG – Mechanistic Empirical Pavement Design Guide

OBC – Optimum Bitumen Content

OPC – Optimum Polymer Content

SGC – Superpave Gyratory Compactor

SHRP – Strategic Highway Research Program

UTM – Universal Testing Machine

PAV –Percent Air Voids

VFA – Voids Filled with Asphalt

VMA – Voids in Mineral Aggregate

SPT – Simple Performance Testing

AMPT – Asphalt Mix Performance Tester

ABSTRACT

Transportation infrastructure plays a substantial role in the everyday life of social beings. The preservation of this vast infrastructure needs appropriate and cost-effective material and design technique. Several distresses are associated with pavement structure but more severe include Rutting, fatigue cracking and stripping Etc. Due to these severe kinds of distresses; pavement fails before completing its service life. In 1987, strategic highway research program (SHRP) put substantial effort to introduce new mix design procedure and in 1993 SHRP introduced SUPERPAVE system that is purely based on performance based specifications. Much work has been done on Superpave around the world but it is yet to be implemented in Pakistan.

In this research HMA is characterized by two testing protocols as Asphalt Mix Performance Tester (AMPT) and Universal Testing Machine (UTM-25P). The three candidate tests for AMPT include dynamic modulus |E*|, flow number (FN) and flow time (FT) tests and Three candidate test for UTM-25P includes Indirect tensile strength (ITS), Resilient Modulus (Mr) and Tensile Strength Ratio (TSR). All these tests are conducted on two Polymers (LDPE and HDPE) and conventional mixes. Bitumen binder used is of ARL 60/70 grade and aggregate source is of Margalla Quarry. Optimum binder content was determined by means of Superpave Mix design method and based on the 4.36% OBC optimum polymer contents were determined incorporating stability and flow test. Samples for performance testing were prepared and then cored and trimmed to the specified dimensions. Dynamic modulus (E*) test was directed on 4 different temperatures i.e. 4.4°C, 21.1°C, 37.7°C and 54.4°C and 6 different frequencies i.e. 25Hz, 10Hz, 4Hz, 1Hz, 0.5Hz and 0.1 Hz. The |E*| test results were subjected to non-linear optimization technique to develop stress-dependent master curves which revealed that Polymers significantly influence the stiffness of mixtures

2-level factorial design of experiment technique was utilized to find the simultaneous effect of independent variables and their interaction on the response. Three factors were found to have a significant effect on the values of dynamic modulus i.e. temperature, frequency and Modifiers. Mixture prepared using LDPE showed better stiffness. Fatigue parameter was calculated using the dynamic modulus and phase angle values and the results revealed that fatigue parameter value is low for LDPE mixtures at 4.4°C and at higher frequencies of 25HZ at 21.1°C which means that LDPE mixtures are less fatigue susceptible.

Flow Number and Flow Time tests were conducted at temperature of 54.4ºC and a stress level of 300 Kpa. Flow number and flow time results were also analyzed to determine the rutting susceptibility of mixes. The mixtures prepared by LDPE accumulated less strains as compared to HDPE and conventional mixtures making it less rut susceptible.

ITS test was carried out at 25ºC temperature both in dry and wet condition to determined tensile strength ratio which is a measure of moisture damage all mixes possess more than 0.9 TSR value .out of those LDPE showed 5.6% better resistant against moisture damage. Resilient Modulus (Mr) test was also performed at 25ºC temperature and 20% of Peak force obtained in indirect tensile strength test. Resilient modulus results also confirm the trend observed by dynamic modulus test that LDPE has high stiffens value following by HDPE and conventional mixtures.

Cost effectiveness analysis was also carried out keeping all the factors constant and calculating the cost of bitumen replaced by polymers. which shows that LDPE is 4.63% and HDPE is 0.53% cost effective than Conventional mixtures because both the waste polymers have less cost as compared to the bitumen.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Transportation theaters a leading role in the development and the socio-economic development of any country no matter whether it is developed or developing. If the transportation facilities of a country are enhanced they will lead to rapid movement of goods and people resulting in increased economic growth rate and development of a country. Building new airports, roads and railways improves the existing transportation system and provides massive employment opportunities. On the other hand, lack of transportation facilities may lead to delays and can become a barrier in the development and socio-economic growth of a country. Transportation modes include road transport, rail transport, space transport and pipeline transport etc.

Road transport is the major component of a transportation system all over the world specially in Pakistan. Asphaltic concrete pavements also known as flexible pavements are most commonly used form of the roads all over the world. Much importance is being given to constructing asphalt pavements which have extended life span and can provide the desired level of comfort and ease thus to serve the purpose for which they are intended. Cost-effective and acceptable design also plays significant role in hot mix asphalt (HMA) pavement structures. Main ingredients used in asphaltic concrete pavements are aggregates and bitumen as a binder upon which the response of pavements is mainly dependent. To achieve the anticipated performance, it is vital to build up a relationship between the ingredients of HMA and its performance. Distresses associated with pavement structure includes rutting, fatigue cracking, stripping, ageing and raveling etc. Factors contributing to these distresses include severe loading, temperature, moisture, design deficiencies, poor construction practices and material specifications. Due to which pavements fail before finishing their service life and requires maintenance and rehabilitation which in turn causes enormous burden on nation’s wealth. This premature failure of pavements is a global problem. So, there is a need for the development and improvement of mix.

The major step towards the mechanistically designed pavements was taken when AASHTO in collaboration with Federal Highway Authority and NCHRP started a project designated as NCHRP Project 1-37A to develop mechanistic empirical pavement design technique. A basic requirement for MEPDG is characterization of materials so that material input can be provided for the design process in addition to loading, traffic and environmental conditions. To fulfill this requirement FHWA and NCHRP started the development and funding of research projects and recommended Simple Performance Test Protocols for characterization of HMA mixtures (Bhasin 2004).

Dynamic modulus is a major performance test for characterizing asphaltic concrete and can be performed over different temperatures in the array of (-10 to 60°C) and different loading frequencies (25 Hz to 0.01 Hz). It is an essential input parameter used in MPEDG software for the characterization of materials and aids pavement structural design process and can be used to develop models for the prediction of pavement response. Along with dynamic modulus flow number and Flow time are also used in the direction of fully recognize the visco-elastic nature of asphalt mixtures (Witzak2002).

The flow number is carried out at a single temperature and a single effective stress level. It is used to evaluate pavement rutting performance. While performing this test, repeated load is applied axially on the sample with 0.1 sec of loading period after 0.9 sec of break period or dwell period which allow recovery of the elastic strains. And the load cycle at which tertiary flow just begins is designated as flow number. The difference among flow time and flow number is changed loading pattern. In flow time (Ft) test static load is applied axially on the sample and the strains are measured for a definite period or until failure. This test is also used to predict rutting performance and visco-elastic behavior of asphaltic concrete mixtures under static stresses.

1.2 PROBLEM STATEMENT

Pakistan National highways are facing serious concerns against premature distresses. To overcome this issue serval processes could be adopted. The factors which can contribute to improve overall HMA properties are Mix design, Binder properties and aggregate properties. A variety of mix design methods are being practiced all over the world. In Pakistan, the Marshall Mix design technique (ASTM D 1559) is being practiced for the design of HMA. Researches have revealed that the super pave gyratory compactor delivers specimens with lower overall changeability than specimen compacted by means of the Marshall hammer. This lower changeability should product in a more reliable design.

Waste polymer is a serious concern for environmental agencies because it is a non-degradable material in nature. The studies have shown that waste polymer of HDPE and LDPE has significant effect on stiffness of binder. A stiff binder is more resistant to permanent deformation. A comprehensive research program has been developed to investigate the effect of mix design with different type of polymer as LDPE and HDPE to improve the HMA characteristics for local condition of Pakistan.

1.3 RESEARCH OBJECTIVES

- The objectives of this research work are:
- To synthesis the research finding on use of waste polymers in HMA (wearing course)
- To determine the optimum waste polymer content
- To compare the rutting potential of conventional and waste polymer modified HMA (Wearing course) by flow number test(Fn)
- To compare the moisture susceptibility of conventional and waste polymer modified Samples by tensile strength ratio(TSR)
- To carry out a comparative analysis of mechanical properties of waste polymer modified HMA (wearing course) which includes Indirect tensile strength (ITS), Resilient Modulus (Mr) and simple performance testing (SPT)
- To carry out cost effectiveness analysis of use of waste polymers in HMA (wearing course).

1.4 SCOPE OF THE THESIS

To achieve above cited objectives a research methodology was developed and planned. A detail study will be carried out research on simple performance tester and UTM-25P. The level of research already carried out in Pakistan was done to get familiar with the simple performance tester and UTM-25P. In this study 3 simple performance tests (SPT) i.e. dynamic modulus, flow number (Fn), flow time (Ft) and on UTM-25P i.e Indirect tensile strength (ITS) and Resilient modulus (Mr) test will be carried out on conventional samples and modified samples with LDPE and HDPE.ARL 60/70 binder and a single sourced aggregate i.e. Margalla aggregate which is primarily lime stone was used. Superpave Mix design method was employed for the determination of optimum binder contents(OBC). Using these OBC, Optimum polymer content (OPC) were determined. After that laboratory specimens were prepared for the performance testing according to the specification and then trimmed and cored to meet the desired dimensions.

The indirect Tensile strength(ITS) was performed in wet and dry condition at temperature of 25°C to determine tensile strength ratio. Tensile strength ratio (TSR) shows the moisture vulnerability of the asphaltic samples. The resilient modulus (Mr) test was completed at 25°C. The dynamic modulus test was performed at 4 temperatures and 6 frequencies whereas the flow number and flow time tests were directed at a single stress level of 300kpa and a temperature of 54.4°C. Dynamic Modulus test results helps to develop master curves using non-linear optimization technique in excel with the help of solver add on. Fatigue parameter was developed for the mixes using dynamic modulus and phase angle and results were compared to determine which mixture is more susceptible to fatigue cracking.

Two level factorial design was also conducted using Minitab 15. Test matrix adopted for this research is shown in the Table 1.1.

Table 1-1: Test Matrix

Test	TSR		Resilient Modulus	Flow Time	Flow Number	Dynamic Modulus					
Temperature	25^{0}c		25^{0}c	54.4^{0}c	54.4^{0}c	4.4^{0}c, 21.1^{0}c, 37.8^{0}c & 54.4^{0}c					
Condition/ Loading	Wet	Dry	100 ms	Stress 300 Kpa	Stress 300 Kpa	0.1 HZ	0.5 HZ	1 HZ	5 HZ	10 HZ	25 HZ
Control	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
LDPE	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
HDPE	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Total samples	81 (3 replicate for each testing)										

1.5 ORGANIZATION OF REPORT

This research thesis consists of five chapters and an appendix portion.

Chapter one consists of introduction to simple performance testing, introduction to various mix design methodologies problem statement and research objectives.

Second chapter consists of literature review of already carried out research on simple performance testing and how to develop the dynamic modulus master curves and dynamic modulus prediction models. This chapter also covered a detailed literature on flow number and flow time, their mechanism and work done by various researchers.

Chapter three explains the detailed methodology of the research i.e. selection of materials and mix design process, preparation of specimens for the performance testing and testing procedures and equipment’s in details.

Chapter four consists of results and analysis of the test data. Master curves, factorial deign of experiment, comparison plots etc. are a part of this chapter.

The last chapter summarizes the report along with conclusions and recommendations

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Due to increased traffic volumes, severe loading of vehicles, adverse environmental conditions, poor construction practices and out dated mix design methodologies, flexible pavement failure soon after the construction, before completion of their design life has become a major problem for road stack holders which are mainly road users, road agencies and government. To solve this global problem of premature pavement failure pavement researchers and engineers by AASHTO and NCHRP started a project aiming to shift mix design methodology from empirical to mechanistic phase and the research project was designated as NCHRP 1-37A.

2.2 HOT MIX ASPHALT

Hot mix asphalt (HMA) is mostly used for the flexible pavements. If the composition of the hot mix asphalt is considered, then it is made up of two main materials one is the binder and the other are the aggregates. The other names of the hot mix asphalt are asphaltic concrete; plant mix and the bituminous mix. Primarily the aggregates depend upon the coarse and fine aggregates. The pavement made up of these materials is known as the flexible pavements. As the flexible pavements are composed of wearing surface, sub base, base and subgrade.

2.2.1 Materials Used in Hot Mix Asphalt

The maximum volume or a weight of the hot mix asphalt (HMA) is dependable on the various categories of the aggregates as per gradation curve criteria. These aggregates include the different sieve sizes. By their size the aggregates are characterized as coarse, fine aggregates and mineral filler. The high percentage of overall volume is involved in the hot mix asphalt therefore it’s necessary to have a careful selection and the addition of the different sizes aggregates in the hot mix asphalt.

Asphalt binder is a thick and a heavy material that is obtain after the process of the refining the crude oil. If the chemical composition of asphalt binder is taken then it consists of different amounts of carbon, sulfur and the other hydrocarbons. Most of its chemical composition is of carbon and oxygen. When the asphalt is placed at a room temperature it behaves like a soft rubber and its consistency is like a soft rubber. When the temperature is high the asphalt becomes liquid. But on the sub-zero temperature the property of the bitumen resembles to a brittle material. When the asphalt is mixed up with the polymers and its physical properties are improved then it is known to be the modified hot mix asphalt. Their consistency and changes in temperature is highly depended upon the types of the polymers to be added in the mix.

2.3 POLYMERS AS A MODIFIER IN ASPHALT MIX

The typical meaning of the polymer is having “more than one part”. Polymers are made up of the large molecules that are generally obtained by combining the small molecules. These small molecules known as the monomers. When these monomers combine with the large molecules they form long chains. Asphalt mix modification can be done by many ways; these includes the modification by adding several additives but the addition of the polymers to improve the hot mix asphalt (HMA) have many merits.

The use of polymers in hot mix asphalt has become a common practice. They can also be used for coating the aggregates. Polymers improves the moisture susceptibility, temperature susceptibility and if the typical value of polymers is used then it also increases the stiffness of HMA. Polymers are of many types but the considerable polymers in this study are high density polyethylene (HDPE) and low density polyethylene (LDPE). By the addition of the polymers the superior engineering properties can be achieved.

Kanlatar et al (2009), concluded the merits of the polymers used in the hot mix asphalt (HMA). These merits include fatigue resistance, moisture susceptibility, thermal resistance and cracking. To improve the hot mix asphalt (HMA) its properties depends upon the mixing and the compatibility of the polymers. Polymers can be classified into different groups and categories but the focus of this study was HDPE and LDPE. These polymers are widely used as a modifier and found to have the different merits for the process of the modification of the hot mix asphalt (HMA).

Yildirim (2007), investigated that the physical properties of polymers are highly depended upon the setting sequence, molecular weight and its chemical composition. One must observe the mixing composition as it effects the properties of a bitumen.

2.3.1 Polyethylene Waste

Plastic waste management is one of the serious environmental problems in the world and it has many challenges for the recycling process. The recycling and disposing of some waste polyethylene nylon types have many challenges. These are attributed to the difficulty of making separation, type of polyethylene used, incineration problems, environmental regulations, and increase in the cost of recycling. Use of such waste materials in civil and construction engineering has become an attractive alternative to disposal, to reduce both the cost of disposal and outdoor waste quantities.

Awwad and Shbeeb (2007), concluded that using polyethylene in a mixture commands over the internal properties of a bitumen as the aggregate and bitumen has the better adhesion. It increases the fatigue resistance and the overall reduces the pavement deformation. About the air voids the statement was not to be more than 4%. If the percentage varies then it may be possible to have a rutting, bleeding and skid. It is very necessary to add the optimum amount of polyethylene in hot mix asphalt(HMA), Because the increased amount of the polymers has the adverse effects on the life cycle of a road pavement. Along this the size of the polyethylene is also considered. Because the grinded particles are found to have the sound covering and shows the better outcomes against the moisture vulnerability of hot mix asphalt. For this purpose, various categories of polymers were used as HDPE and LDPE.

2.3.2 High Density Polyethylene (HDPE)

The use of high density polyethylene becomes popular day by day as it proved to be the good modifier in hot mix asphalt (HMA). The effect of high density polyethylene (HDPE) on the physical properties of bitumen like softening point penetration and ductility to be noted. By concerning the different research papers the high-density polyethylene has shown its practical behavior in modifying the hot mix asphalt (HMA). HDPE density is more than 0.941 g/cm3.

Figure 2-1: High Density Polyethylene

Hinislioğlu, Aras et al. (2005), stated that by preparing the high density polyethylene (HDPE) as per Marshall mix design criteria increases the Marshall quotient up to 57%.The creep behavior is increased up to 52% .According to their point of view the stability value increase up to the range from 3-21% while their results concluded the overall decrease in flow varies from 17 to 25 %.The overall summary of research shows that by using the high density polyethylene (HDPE) improves the properties of flexible pavement .

2.3.3 Low Density Polyethylene (LDPE)

Low density polyethylene can be obtained from the shopping bags. Chemically it involves a monomer ethylene in its composition. Low density polyethylene has a vital use in laboratory as well as in manufacturing of the different items. Its physical properties may be unpredictable under a high temperature. In melting point of view, it has a lower viscosity in consideration of strain. While going through the testing phase the low-density polyethylene plays a tremendous role by dry mixing. Low density polyethylene is less permeable as compared to the high-density polyethylene because of more viscosity and low shear sensitivity. In controlling the moisture sensitivity of modified hot mix asphalt (HMA) the low-density polyethylene is quite reasonable. But during the mixing process the melting point of (LDPE) should be considered. The addition of these polymers can advance the moisture vulnerability of HMA, and improves its internal properties to resist against the other damages. LDPE density range is 0.910–0.940 g/cm3

Figure 2-2: Low Density Polyethylene

Using LDPE, the environmental pollution can be controlled up to some extent. Commonly the shopping bags made up of the low-density polyethylene causes the environmental pollution so if these products are used as a modifier that can help to show the resistance against the moisture and reduce the pollution. These polymers can improve the voids and shows it significance in Marshall Flow and stability test. While mixing these types, the dry process is found to be useful as by the dry process of mixing the homogenous mixture can be obtained. The mixing percentage of the low-density polyethylene during the experimentation is found to be (4%, 6%, 8%, 10%, 12% and14%). The weight of the plastic is very light and they are not decade over the years as it shows a great chemical resistance and they are not attackable by the acids. One of its main advantages is that they are easily available and helps in the modification of bitumen at economical cost. Its properties like toughness and flexibility at low temperature leads it for a vital use. It can easily be transported from one place to another due to its light weight. It can be simply stated that low density polyethylene is found to be the alternate solution towards the environmental pollution and the problems caused by this pollution

Punith and Veeraragavan (2007), have concluded that while using the LDPE as modifier in the HMA. The higher stability and void mineral aggregate percentage was obtained between 4 to 12%. The percentage of the optimum polymer content (OPC) was to be tabulated as 10% by the overall weight of the aggregate particles. The value of the void filler aggregate (VFA) is increased. This value was increased when the aggregate of the bigger size was used then the overall value of low density polyethylene (LDPE) was increased form the amount of the 10% by the total weight of the aggregate. some of the properties of the HMA had shown the reasonable improvement using low density polyethylene (LDPE). These properties included fatigue resistance, pavement permanent deformation along with the achievement of the adhesion that occurred between asphaltic content and the aggregate which has been added in the mix.

By the economical point of view, they analyzed that it was useful to use the LDPE in the hot mix asphalt for the modification purpose. Using the LDPE, the amount of the bitumen can also be decreased and it may be helpful in accordance with the economic analysis. From the above research, it is to be concluded that (LDPE) was found to be useful in modifying and improving the internal properties of the HMA.

2.4 PAST RESEARCH ON LDPE AND HDPE:

Al-Hadidy and Yi-qiu (2009) Considered that modified HMA presented higher softening point, the ductility values of (100_{+} cm), and a decrease in percentage loss of weight due to heat and air. Use of LDPE in Stone mastic asphaltic mixes can entertain the performance constraint of high and low temperature.

Jassim, et al (2014) studied about the optimum use of plastic waste to improve the moisture resistance of hot mix asphalt. Resistance to the plastic flow and moisture damage are some of the effects of plastic waste thickness and plastic content. Marshall test and the retain strength index has used to determine the plastic waste properties such as thickness and percentage content.

Ahmed and Ismail (2009) stated that Superpave mix design produces Hot mix asphalt (HMA) with lower air void content and better moisture susceptibility than Marshall mix design. To enhance moisture susceptibility of HMA, Putman and Amir kanian (2006) reported that three types of liquid anti stripping agent and hydrated lime improve the moisture susceptibility. Hydrated lime was the most effective in raising the tensile strength ratio (TSR) value

Musa and Haron (2014) investigate the change in marshal stability, flow and voids due to addition of 4-18% waste LDPE with an increment of 2% of the asphalt. The results were improved especially when we use 10% of waste LDPE. Use of LDPE improved the stability, Flow and decrease the environmental pollution by using in asphalt mixture.

Khan and Kamal (2012) concluded that Superpave mix disclosed low permanent deformation strains, higher resilient modulus (Mr) and higher dynamic modulus as equated to Marshall and SMA mixtures. During Indirect Tensile Strength(ITS), Higher values of Resilient Modulus(Mr) were experiential in case of Superpave mixtures. Even at extreme temperature of 55^{o}C, Superpave mixture shows improvement than the other two mixes.

Khurshid,et al. (2013) concluded that dry Mixing of High Density Polyethylene has performed better than wet Mixing. Wet mixing of waste polymer has issue of non-homogeneous mixing. Dry mixing of 2-14 % HDPE was tested with an increment of 2% for each trial. HMA modified with HDPE has shown better results. Waste polymer modified bitumen has significant properties against permanent deformation as rutting. It also confirms that use of 8% Polymer would lead a significant decrease in construction cost of Rs. 141,200 per lane per KM.

Khan and Kamal (2008) concluded that the practice of the Marshall Mix design technique for asphaltic concrete is one of the contributing causes to the early distresses developed in Pakistani pavements. The drawbacks of the Marshall Mix design method include that it does not take into consideration variation in temperature, loads, or material properties. The compaction procedure of the Marshall Mix method does not replicate the actual compaction which occurs under moving traffic. Researches have revealed that the super pave gyratory compactor offers specimens with lower overall changeability than specimens compacted using the Marshall hammer. This lower changeability had better result in a more reliable design.

Gupta and Veeraragavan (2009) compacted the mixes by by means of both Marshall and Superpave Gyratory Compactor (SGC) and evaluation of these two is recognized in terms of the resilient modulus(Mr) and fatigue life. Involved Marshall Stability and indirect tensile strength ratio(TSR) tests were directed. The enhancement in fatigue life of polymer-modified mixes over the conventional mixtures is stated.

Aloysius and Napitupulu (2013) Carried out indirect tensile strength (ITS), resilient modulus (Mr) test. It was exposed to haversine loading shape. The test has 3 different loading time (i.e. 100, 250, and 400 ms) with break period of 900 ms, and 3 different temperatures. The values of resilient modulus fewer than 2000 MPa that attained from indirect tensile strength (ITS) test are 0.7 to 1.1 of the projected modulus from the formula. For modulus, more than 2000 MPa, are 1.19 to 1.6 of the projected value.

Bhasin et al. (2004) states that the flow time test has been known as one of the tests to measure the important properties of HMA associated to rutting by Witczak et al. (2002) in NCHRP Project 9-19. The test objects at calculating the visco-elastic behavior of an HMA sample under a static stress level. This test can be carried out in confined or unconfined environments. The total compliance at any given point in time, D(t), is considered as the ratio of the measured strain ε_t to the applied stress σ_0.

Awwad and Shbeeb (2007) have shown that density of polythene effects the stiffness of asphalt concrete. Comparison of LDPE and HDPE confirms that high density of polyethylene has shown better results as compared to Low density. Dry mixing of polymer can utilize higher percentage of polymer as compared to wet mixing so 5.4% optimum binder content with 12% optimum polymer content was used to prepare the sample.

Yu and Shen (2012) investigate numerous testing approaches anticipated by Witczak et al. for assessing the rutting resistance including the dynamic modulus (E*) test, flow number (Fn) test, and flow time (Ft) test. The flow number test was originated to be able to compare with field rut depth as confirmed by field projects. The connection between the reduced flow number and field rut depth at a detailed traffic level were considered.

Witzcak (2002) examine flow number(Fn) test to calculate the rut depth of asphalt concrete mixes. It is a difference of the repeated load, permanent deformation test that has been used by investigators since the 1970s. Flow number is defined as the number of load pulses when the lowest rate of change in permanent strain happens through the repeated load test and is determined by variation of the permanent strain versus number of load cycle curve

El-Saikaly (2013), found that addition of the polymers in the hot mix asphalt is founded to be very effective, as they improve the external as well as the internal properties of a bitumen and proved them to be a best modifier. By the previous research and the studies, the compatibility of the modifiers in the hot mix asphalt is reasonable.

2.5 SUPERPAVE VOLUMETRIC PARAMETERS:

Superpave volumetric mix includes air voids, voids filled with asphalt and voids in mineral aggregate. Performance of Superpave gyratory compacted specimen depends upon certain mix characteristics. Typically, HMA weight-volume relationship helps in mix design and construction purpose. Figure 2-3, Figure 2-4 describes weight volume relationship as shown below and Table 2-1 shows Superpave volumetric requirements for HMA.

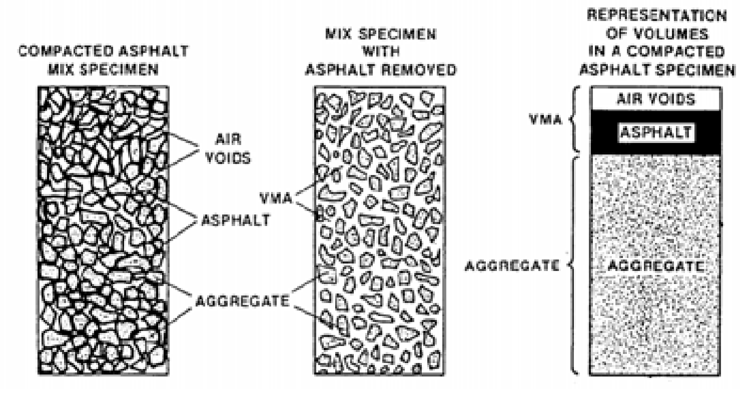


Figure 2-3: Compacted Specimen

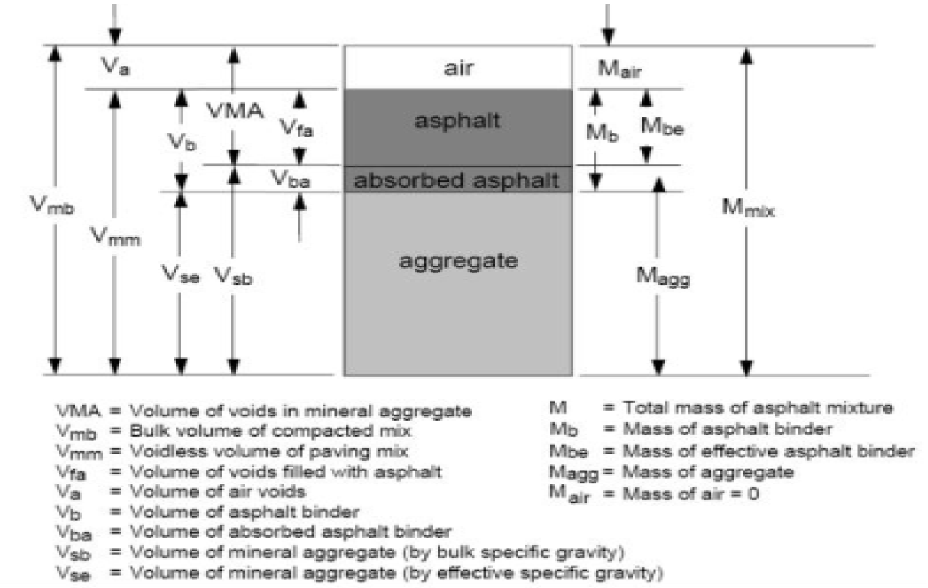


Figure 2-4: Superpave Volumetric

Table 2-1: Superpave Volumetric Requirements

Design ESALs (Million)	Required Density (% of Gmm)			Voids in mineral aggregate (VMA)minimum					Voids filled with Asphalt (VFA)	Dust to Binder Ratio
				NMAS (mm)						
	Nini	Ndes	Nmax	37.5	25	19	12.5	9.5		
<0.3	≤91.5								70-80	0.6-1.2
0.3 to <3	≤90.5	96	≤98.0	11	12	13	14	15	65-78	
3 to <10	≤89.0								65-75	
10 to < 30										
≥30										

2.5.1 Bulk Specific Gravity/Density

Bulk specific gravity is computed as per standard ASTM D 2726 – 05a as determined by this test technique, density of the substance to the density of the water is found to be as the specific gravity or it is equal to the volume of the water. It is an important factor and for the better life of pavement it should be calculated. The weight of the specific volume of the asphaltic mixture is commonly to be termed as the bulk specific gravity. Bulk specific gravity can be transformed into density by multiplying it with 62.14 lb./ft^3.It is calculated by Equation 2.1.

$$Bulk\ Specific\ gravity\ (Gmb) = \frac{A}{B \quad c} \qquad Eq. 2.1$$

Where,

A = weight of sample in air, g

B = saturated surface dry weight (SSD) , g

C = weight of a sample in water, g

2.5.2 Maximum Specific Gravity/Density

Theoretical specific density of mixtures computed according to ASTM D2041.The method described is used to determine the theoretical maximum specific gravity of un-compacted asphalt-aggregate mixes using a vacuum saturation technique and calculated by below Equation 2.2

$$Maximum\ Specific\ gravity\ (Gmm) = \frac{A}{A + (B \quad c)} \qquad Eq. 2.2$$

Where,

A = oven dry weight of a sample in air, g

B = weight of container with water, g

C = weight of sample and container with water, g

2.5.3 Percentage G_{mm} @ $N_{des.}$ and Correctional Factor

SGC data is analyzed to calculate bulk specific gravity (G_{mb}), corrected bulk specific gravity (G_{mb}) and corrected percentage of Theoretical Maximum Density (TMD) for each desired gyration. During compaction, the computer protocol attached with SGC continuously record the height during each gyration. Bulk specific gravity of compacted gyratory sample and TMD of mixture is measured and estimate of Gmb at any gyrations is measured by the Equation 2.3 as shown below

$$Gmb(estimated) = \frac{Wm/_{Vmx}}{\gamma w} \qquad Eq. 2.3$$

Where,

$G_{mb\ (estimated)}$ = estimated bulk specific gravity of sample during compaction

W_m = mass of sample, g

V_{mx} = Volume of compaction mould, cm_3

The above calculation undertakes that SGC sample is smooth sided which is not actually the case because the surface loopholes causes the volume of the specimen to some extent less than volume of level sided cylinder. Thus, correction factor is introduced that is the ratio of measured over the estimated G_{mb} to counter the effect of surface irregularities. The below correction factor is shown in Equation 2.3 is

$$Correction\ Factor(C) = \frac{Gmb(corrected)}{Gmb(estimated)} \qquad Eq. 2.3$$

Where,

C = correction factor for Gmb

$G_{mb\ (measured)}$ = measured bulk specific gravity of a smaple after $N_{des.}$

$G_{mb\ (estimated)}$ = estimated bulk specific gravity of a sample at $N_{des.}$

As discussed earlier mixture specific gravities (G_{mb} and G_{mm}) formulas and the way to compute the gravities is mentioned earlier. Now, G_{mb} is divided by G_{mm} to determine the value of % G_{mm} @ N_{des} and is given in below Equation 2.4.

$$\%Gmm\ @\ Ndes = \frac{Gmb}{Gmm} \qquad Eq. 2.4$$

% G_{mm} at any number of gyrations (N_x) is computed by multiplying %G_{mm} @ $N_{des.}$ by ratio of height a N_{des} and N_x. and is given in below Equation 2.5

$$\%Gmm \;=\% \; gmm \; @ \; Ndes \quad \frac{Ndes}{Nx} \qquad Eq. 2.5$$

2.5.4 Percent Air Voids

Compacted paving mix consists of the low air voids between the coated aggregate particles which are the air voids. The percentage of air voids in a compacted mix can be calculated by the following Equation 2.6:

$$Percent\ Air\ Voids\ \ (PAV) = 100(\frac{Gmm \quad Gmb}{Gmm}) \qquad Eq. 2.6$$

Where,

G_{mb} = Bulk specific gravity of the compacted sample.

G_{mm}= Maximum theoretical specific gravity of the sample.

PAV =Percent Air voids in compacted sample

Air voids are also defined as air pockets that exist between gyratory compacted specimens as given in below Equation 2.7. Purpose of air voids in all field or laboratory compacted specimen is to secure pavement infrastructure from rutting, shoving and flushing. Air. voids in asphaltic pavement can be reduced by bitumen or by fines passing through sieve No.200 and by changing gradation. Air voids content is directly related with density/packing of asphaltic concrete. Higher the packing/density means lower the air voids content. Air voids of compacted specimen is 4.0% but according to Superpave volumetric requirement of Superpave gives flexibility of change in air voids content from to 3% to 5% (Christensen, 2006).

$$Percent\ Air\ voids\ (PAV) = 100 \quad \%Gmm\ @\ Ndes \qquad Eq. 2.7$$

Where,

PAV = Percent air voids @ N_{des}, percentage of total volume

%G_{mm} @ $N_{des.}$ = Percentage maximum theoretical specific gravity @ N_{des}.

2.5.5 Voids in Mineral Aggregate

The voids in mineral aggregate (VMA) is explained as the inter-granular void spaces among the aggregate particles in a compacted mix that include the air voids and the effective binder content. This is quantified as a percent of total volume. The VMA is calculated based on the bulk specific gravity of the aggregate and is also articulated as percentage of the bulk volume of compacted mix So, VMA can be calculated by deducting the volume of aggregate determined by it bulk specific gravity from the bulk volume of the compacted mix. The process for calculation is demonstrated as per the Equation 2.8:

$$Voids\ in\ Mineral\ Aggregate\ (VMA) = 100(\frac{Gmb \quad Ps}{Gsb}) \qquad Eq. 2.8$$

Where,

VMA = Voids in mineral aggregate (%)

P_s = Percent of total aggregates in the mix

G_{sb} = Combined specific gravity of aggregates

VMA considerably influence the behavior of mixes because if its value is low, the mixtures might experience durability complications, and if its value is high, then the mixes will exhibit stability issues and thus it becomes uneconomical to produce. The volume of bitumen in the mix including the aggregate controls the depth of the bitumen film around aggregate particle. Without acceptable film depth, the bitumen oxidizes faster, the films are more easily penetrated by moisture, and tensile strength of mix is badly affected. For this reason, the VMA should be necessarily high to make sure that there is space for the bitumen plus the air voids. VMA is defined as voids spaces that occur among the aggregates in compacted mixes including space that filled with binder. VMA also represent the space that accommodates effective volume of binder and volume of air voids necessary in paving mixture. VMA can be calculated by below Equation 2.9:

$$\%\,VMA = 100 \quad \frac{\%\,Gmm\,@\,Ndes \quad Gmm \quad Ps}{Gsb} \qquad Eq.2.9$$

Where,

G_{mm}=Theoretical maximum specific gravity.

G_{sb} = Bulk specific gravity of aggregate.

P_s = aggregate content, cm3/cm3, by total weight of mix.

If optimum percentage air voids criteria is satisfied then volumetric criteria is compared and analysis of blend is completed. The volumetric (VMA and VFA) at N_{des} and mix density at N_{ini} is estimated at this bitumen content is calculated by using Equation 2.10: For VMA

$$\%\,VMA(estimated) = \%\,VMA\;ini\; + C \quad (4 \quad PAV) \qquad Eq.2.10$$

Where,

C = constant = 0.1 if PAV is less than 4.0 percent

C= 0.2 if PAV is greater than 4.0 percent

2.5.6 Voids Filled with Asphalt

Voids filled with asphalt (VFA) are illustrated as the percentage of the inter-granular void spaces between the aggregate particles (VMA) that is occupied with asphalt. VFA, not with the absorbed asphalt, is determined using following Equation 2.11:

$$Voids\;Filled\;wit \quad Asp \quad alt\;(VFA) = 100(\frac{VMA \quad PAV}{VMA}) \qquad Eq.2.11$$

Where,

VFA = Voids filled with asphalt content (%).

VMA = Void in mineral aggregates (%).

PAV = Percent air voids in the compacted mix.

VFA is also defined as “the voids that exist between the aggregate particles occupied by bitumen and is stated as percentage of voids in mineral aggregate that contains binder” and can be shown through the below Equation 2.12:

$$\%\,VFA(estimated) = 100(\frac{\%VMA(estimated)\quad 4.0}{\%VMA(estimated}) \qquad Eq. 2.12$$

VFA restricts less durable mix resulting from minimum asphalt film thickness over aggregate in the case of light traffic condition as well as avoids those mixtures that are rut susceptible under heavy traffic condition.

2.5.7 Dust to Binder Ratio

The quantity of percentage of aggregate finer than the sieve # 200 by weight to the projected effective binder content articulated as a percentage of total mixture. The dust quantity is calculated as shown in Equation 2.13.

$$Dust\ to\ Binder\ ratio\ = \frac{P0.075}{Pbe} \qquad Eq. 2.13$$

Where,

P_{200} = Passing 0.075mm sieve.

P_{be} = Effective binder content (%).

2.5.8 Absorbed Asphalt

Absorption of asphalt is defined as percentage of mass of aggregate rather than percentage of total mix. Absorbed asphalt is express by the following Equation 2.14.

$$Volume\ of\ absorbed\ asp\ \ alt\ (Vba) = \left(\frac{Gse\quad Gsb}{Gsb\quad Gse}\right)\ Gb\quad 100 \qquad Eq. 2.14$$

where,

G_{se} = Effective specific gravity of aggregate.

G_{sb} = Bulk specific gravity of aggregate.

G_b = specific gravity of binder.

2.5.9 Effective Asphalt

It is the content which is absorbed by aggregate out of total asphalt content and is calculated by using Equation 2.15

$$\boldsymbol{Effective\ binder\ contnet\ (P\text{be}) = Pb \quad (\frac{Vba}{100}\ \ Ps) \qquad Eq.2.15}$$

Where,

P_b = Binder content, (%).

Ps = Aggregate content, (%).

2.6 STABILITY AND FLOW TEST

The Marshall Stability and Flow is important parameter in JMF besides density, Voids in Mineral Aggregates (VMA), Percent air Voids (PAV) and Voids Filled with Asphalt (VFA). These are used for the evaluation of bituminous mixture and mix design (ASTM D6927). In addition, Marshall Stability and flow can help observe the plant process of producing the asphaltic mixture. Marshall Stability and flow can comparatively assess the different bituminous mixtures and the effects of habituation. When the bulk specific gravity of the specimens has been calculated, the stability and flow tests are carried out by means of Marshall testing machine.

The stability of the mixture is stated as the measure of the extreme load that is supported by test sample at the constant loading rate of about 2-inch/minute. Fundamentally, the load is amplified until it reaches the extreme. The loading is stopped when load decreases and the extreme load is recorded. Through the loading, attached dial gauge to the machine, measures the sample's flow as an outcome of the loading. The flow value is documented in 0.01 inch additions at the same time the load is documented.

2.7 SIMPLE PERFORMANCE TESTING

Papzian was the first who in 1962 developed an advanced method for defining the response of linear viscoelastic materials. He investigated the viscoelastic performance of asphalt mixture by means of the complex modulus testing method. For this purpose, he performed testing on several test samples under controlled temperature and frequency and viscoelastic behavior of hot mix asphalt was studied by means of algebraic coefficients linking stress to strain which are complex functions of frequency, and equations were formed which state viscoelastic stress-strain laws in the frequency domain, as well as in the time domain (Papzian 1962).

Kallas in 1970 observed that considerable variations turn into more obvious with the increase in temperature chiefly associated to phase angle (Kallas, 1970).

Bonnaure et al. (1977) performed testing in which dynamic load were applied to trapezoidal specimens and the modulus of these specimen was determined from a graph of load applied and resulting deformations. He concluded that stiffness of hot mix asphalt is significantly affected by loading time and temperature. It had a negative relation i.e. E* value reduces with an increase in the loading time or temperature. In addition, they also revealed that E* curves from various temperatures and loading times could be superimposed. This has now turned into an incredibly helpful tool in the shape of master curves. By making use of the bi-modular study method, together with the modulus determined both in tension and compression the researchers could bring more accuracy in the prediction of hot mix asphalt properties. (Witzak and Root, 1974; Khanal and Mamlouk, 1995).

Lekarp et al. (2001) conducted triaxial test on three unbound granular materials to find the effect of grading materials with different nominal maximum aggregate sizes. From the results, he concluded that nominal maximum aggregate size plays important role in structural response of the unbound materials. And if nominal maximum aggregate size is decreased it will directly affect the permanent deformation properties and resilient strain properties of the HMA mixes.

However, he was unable to find the nature of these impacts due to complexity of the different materials and inconsistency of the results.

Nega et al. (2015) tested seven asphalt mixtures produced in laboratory with different polymer modified binders. Dynamic modulus test was used as a performance indicator and the temperature susceptibility of the dynamic modulus result was found. Effect of confining pressure was also evaluated. Master curves were generated using the laboratory testing results and a very good correlation was found for each polymer modifier and between binder viscosity and temperature.

Uzan et al. (2003) characterized HMA mixes based on their rutting performance using a mechanistic empirical procedure. Dynamic modulus and repeated load tests were carried out at numerous temperatures and confining pressures to check material sensitivity against testing conditions. He found that material behavior was very sensitive to testing temperature and confining pressure. He also developed a master curve based on Fillers–Moonan–Tschoegl (FMT) equation.

Mu-yu L& Shao-yi (2003) took two factors rutting and cracking and developed an optimization model, using genetic algorithms for solution of that model. It was a new idea for HMA structural optimization.

Amit et al. (2003) carried out a study in which nine HMA mixes were acquired from state Departmental of Transportation and have changing levels of field performance. The research also included testing of three lab prepared specimens. Three simple performance test protocols were used for mixture characterization. From this research, they arrived at the conclusion that gradient of flow time (Ft) (creep load) and flow number (Fn) (repeated load) values demonstrated a strong correlation with the APA's rut depth. And if we compare them the correlation of APA's rut depth with the flow time (Ft) slope was stronger than that of with flow number (Fn).

Bonaquist et al. (2004) carried out a research study to develop an instrument for conducting three SPT tests. The study was extremely successful and at the end of the project they develop full specifications of the instruments for both the manufacturers and the users. Two instruments were developed for SPT testing one was Interlaken SPT system and the other was SPT system. An evaluation study was carried out to check the suitability of these two instruments for SPTs. The findings suggested that both the units were user friendly and meeting the requirements with certain common deficiencies. In case of dynamic modulus results, there was some variability in the results that was found to be due to variability of deformation measuring devices that were glued to the test specimen placed in the environmental conditioning system. The level of variability for flow number results by the two instruments was not significant. Based on the findings of the evaluation study SPT system was found to be more suitable for SPT testing. The study resulted in the development of AMPT that is Asphalt Mix Performance Tester.

Bahia et al. (2005) used two aggregate gradations commonly used in Wisconsin. Aggregate was acquired from four different sources. The output achieved from the uniaxial repeated creep test was the flow number, and from the results it was clear that asphalt mixtures showed tertiary creep failure. The results also showed that a strong relation exists between resistance to permanent deformation and traffic force index.

Mohammad et al. (2005) performed four tests including dynamic modulus and flow number on six plant-prepared hot mix asphalt mixtures. From the results, they arrived upon the conclusion that results from flow number tests were consistent with the In-situ performance of those mixes selected for the study. Additionally, they could find a strong correlation between flow number (Fn). a and b-values determined from secondary portion of accumulated strain vs cycles curve. These parameters were used for the investigation of a flow number test, particularly when the tertiary flow zone was not attained. It was also found that rut depths found by using Hamburg Wheel Tracking test and by flow number test had a strong correlation.

Romanoschi et al. (2005) picked up four Superpave mixtures commonly used in base course of pavements in Kansas with the objective to characterize and evaluate the mixtures dynamic modulus, their bending stiffness and resistance to fatigue cracking. After analyzing the results dynamic modulus was found to be a poor indicator of the mixture’s fatigue resistance. Mixtures with lower amount of air voids shown more resistance to fatigue cracking as compared to those with higher amount of air voids because low air voids content leads to higher values of dynamic modulus. The predictive ability of the Witzak model was also evaluated in this study and it was found that it strongly under estimated the dynamic modulus for all the four mixtures under study. In most cases measured E* was twice the Predicted E*. Keeping the temperature and frequency constant a comparison of E* and bending stiffness revealed that E* is twice the bending stiffness of the HMA mixtures.

Muhammad et al. (2006) characterized asphalt mixes based on their rutting performance using dynamic modulus, flow number (Fn) and Hamburg wheel tracking test. Sensitivity analysis was carried out using MEPDG software to check dynamic modulus role in rutting performance prediction of pavements. They also tested these hot mix asphalt mixtures by conducting repeated load test (FN) and static creep test (FT) HWTT and |E*| test. From the tests results they arrived at the conclusion that the rut resistance parameters from the FT, FN and |E*|test can differentiate among mixtures based on their design traffic.

Cross et al. (2007) conducted dynamic modulus test on various mixtures prepared using different types of aggregate and bitumen binders. Testing was directed at 5 different temperatures and 6 different loading frequencies were selected for application of sinusoidal loading. The testing of dynamic modulus at lower temperature of -10 °C was observed to be difficult and intensive process due to development of frost on the test frame including test specimens and LVDTs. After analyzing E* testing results he reported that loading frequency and testing temperature were the major factors having a significant effect on the values of E*.

Garcia & Thompson (2007) Conducted a study that comprised of three different stages for the evaluation of E* prediction equations. The mixtures used for the study were taken from Illinois DOT. Objective of the study was to develop modulus-temperature generic equations that can be used for design of roads. Currently used E* prediction models were also evaluated and Hirsh model was originated to be best forecasting the dynamic modulus values with high precision and low error. To eliminate or minimize these errors a database of correction factors was developed and the amount of error was significantly reduced when these correction factors were applied to the Hirsch model.

Flintsch et al. (2007) conducted HMA testing to characterize HMA mixes for implementation of MEPDG in Virginia. Dynamic modulus test was conducted along with creep compliance test and indirect tensile test for evaluation of thermal cracking in surface, intermediate and base layers. Resilient modulus test was also conducted to correlate dynamic modulus with resilient modulus. Based on the test results he found that dynamic modulus test can best characterize the HMA mixes at different temperatures and frequencies. Also, dynamic modulus results were affected by mix characteristics such as NMAS, binder content, aggregate type etc. He also found that dynamic modulus was reasonably predicted by level 2 prediction equations with some differences. But the results produced by indirect tensile strength (ITS) test and creep compliance test were non-repeatable due to some reasons. Phase angle values generally increased with increased temperature, but at high temperature and low frequency reduction was observed in phase angle values due to aggregate interlock dominant behavior.

Wu et al. (2007) directed dynamic modulus test to assess and characterizes the HMA mixes modified with different fibers such as cellulose, polyester and mineral fibers. Dynamic modulus and phase angle values were found at numerous temperatures and frequencies for modified and control HMA mixes and it was observed that HMA mixes modified with different additives produced higher dynamic modulus values as related to control mixtures. Using dynamic modulus results he developed dynamic modulus master curves by means of time temperature superposition principle based on nonlinear regression. He also calculated fatigue and rutting parameters and comparison with control mix revealed that these properties were improved by using fiber additives.

Sugandh et al. (2007) evaluated the ability of flow number test to detect the existence of modifier in HMA mixtures and to assess the changes in the performance of HMA mixes which occurred due do the addition of modifier. Four modifiers were used for designing HMA specimens for performance testing. Flow number test (Fn) was led at a static stress level of 210 kpa at a temperature of 54.4ºC. After analyzing the results, it was concluded that flow number test could detect the existence of modifier in case of rutting susceptibility of the mixtures but it failed in case of fatigue.

Abdo et al. (2009) conducted a research study involving testing of 17 HMA mixtures using dynamic modulus test for developing a prediction model that uses HMA mix parameters for estimating dynamic modulus. He reported dynamic modulus results were significantly affected by the variations of bitumen grade and its percentage and by the aggregate gradation. Further, regression analysis was used for the development of the model which resulted in R2 value of 0.94.

Ceylan et al. (2009) employed advanced neural network methodology instead of regression modelling for prediction of dynamic modulus. The new ANN methodology is used to solve complex problems. The ANN predictive model was developed using latest E* data base. The predictive capability of the new ANN models was found to be higher as compared to the existing predictive models based on regression equations. He suggested that this technique due to its higher predictive accuracy will result in better material characterization and may reduce chances of premature pavement failures.

Bonaquist tested twelve (12) laboratory prepared HMA mixes commonly used in Wisconsin prepared using aggregate from different sources and binders of different grades for E* and permanent deformation. Specimens were tested at three different temperatures that are 4ºC, 20ºC and 35ºC and three frequencies were selected for application of sinusoidal loading that were 0.1 Hz, 1 Hz and 10 Hz. He determined the sensitivity of AMPT results when mix design factors were changed and concluded that E* and flow number results are significantly affected by change in binder or gradation. His results were used to develop a data base of E* and master curves for use in MEDG related efforts (Bonaquist 2010).

Kaloush et al. (2010) conducted tests on 94 hot mix asphalt mixes and obtained a large number of flow number test results. These results were used to build up a flow number prediction model. Their model showed good accuracy with an R2 of .62. Researchers stated that their model is applicable to broad array of temperature, stress condition and mixture types. Furthermore, it is also essential to point out here that the inconsistency inside replicates used in their research was reasonably high.

Wassage et al. (2010) conducted two tests on HMA mixes. One was repeated creep test, a new test method to check the elastic response of modified bitumen binders and behavior of HMA mixes was modelled using linear and nonlinear rheological modeling. The second test was repeated load permanent deformation test recommended by NCHRP as a candidate member of SPTs. The accumulated strains in the material due to cyclic loading were evaluated and viscoelastic theory was used to describe HMA behavior under haversine pulse loading.

Ahmed et al. (2011) conducted a study using densely graded HMA mixtures produced by Superpave and marshal mix design method and using granite aggregate of Klang, Malaysia to assess the rutting resistance by using dynamic modulus test. Test results were obtained for different temperatures and frequencies so that data at Various temperatures can be shifted w.r.t loading frequency to develop a master curve that was used as a comparison of stiffness between the mixtures and it was revealed that mixtures designed using Superpave mix design had higher stiffness as compared to that of marshal mix design method. Rutting resistance of mixtures was also evaluated using HWTT and an effort was made to develop a relationship between the results of dynamic modulus at higher temperatures and rut depths obtained by HWTT. Strength of the correlation was higher at a frequency of 5Hz and test temperature ranging between 40 C and 50ºC and it was concluded that rutting resistance of HMA mixes can be evaluated by means of dynamic modulus test.

John & Dallas (2011) evaluated the capability of flow number (Fn) and flow time test (Ft) to assess the rutting susceptibility of airport pavements that are designed to take higher tire pressures as compared to normal road pavements and the results of the study were compared to rut depths obtained by conducting tests on APA. Aggregate used for design of specimens were of lime stone, granite and chert gravel. A total of twenty-six specimens were formed by means of a single binder that is PG 64-22 and fine and coarse gradations. After analyzing the results, they concluded that mixture containing a rich amount of natural sands as higher as 30% sand is more prone to rutting as compared to mixtures with lower amount of sand. Tertiary flow state was achieved in less than 10 seconds in case of flow time test whereas in flow number test the time taken for starting of tertiary flow stage was 60 cycles. A correlation was found between the secondary flow part of the flow time test and results achieved through APA having a higher value of R2 representing higher strength of the correlation. Similar correlation was produced between flow number and APA test results but having comparatively low R2. After ranking the HMA mixtures based on their rutting susceptibility it was found that the order of the mixtures is same for both flow time and flow number test.

Miljković and Radenberg (2011) reported that excessive rutting can cause troubles in terms of safety, ease, and overall pavement life-cycle cost. HMA rutting vulnerability is relying on constituent materials and their content. In addition to Superpave mix volumetric design Simple Performance Tests were suggested to get a better idea of the HMA properties.

Apeagyei et al. (2012) developed a technique to develop master curves without conducting dynamic modulus testing at highest and lowest temperatures as required by AASHTO TP-62. This technique was named as abbreviated testing temperature (ABBREV) resulted in time saving due to reduced testing temperatures. Dynamic modulus results at highest and lowest temperatures were predicted using regression models. Master curves were developed using combination of predicted and measured dynamic modulus values.

Hefeez et al. (2012) conducted a research study for prediction of performance of HMA mixes from the characteristics of bitumen binder. For this purpose, four different type of HMA mixtures were used prepared from two aggregate gradations and two bitumen binders. Aggregate gradations were Class A and Class B of National Highways Authority Specifications. Testing was performed by applying a sinusoidal uniaxial stress at 6 different frequencies and 3 temperatures and 2 parameters were measured which are dynamic modulus and phase angle. From the outcomes, it was noted that HMA behavior was significantly related to temperature and frequency. Frequency was directly related to dynamic modulus at a constant temperature whereas temperature was inversely related to dynamic modulus. Master curves were developed at a reference temperature of 25 C by means of TTS. It was found that HMA mixtures with coarse particles and polymer modified binder exhibited higher dynamic modulus values for all frequencies and using master curves one can easily predict HMA behavior from bitumen binder.

Shen et al. (2012) carried out dynamic modulus performance testing using specimens representing the seven asphalt plants of Washington State. After analyzing the results, he suggested the use of Hirsch Model and a modified flow number (Fn) prediction model for conventional dense graded asphalt mixes of Washington State. Moreover, he also reported that air voids significantly affect both the dynamic modulus and the flow number (Fn). Increasing percentage of air voids will also increase the dynamic modulus and flow number values. They also managed to locally calibrate a model for predicting flow number values for Washington State. The model could predict flow number values using volumetric parameters, temperature and type of bitumen binder. They could develop a model which predicted reasonably well for conventional mixes but was not applicable in case of highly polymer modified mix.

Yu and Shen (2012) carried out dynamic modulus testing on HMA mixes containing granite aggregate because of its abundance in Korea. Four HMA mixes were evaluated containing aggregate gradations with two different NMAS and two different. Asphalt binders at a temperature range of -10 to 55. They also compared laboratory results with the values found by using dynamic modulus predictive equations and found that predicted values were lower than the actual values at high testing temperatures and vice versa. He reported that softest binder resulted in HMA mixes with lowest dynamic modulus and as the stiffness of the binder increases dynamic modulus values increases.

Seo et al. (2103) carried out a dynamic modulus study using experimental results and numeric simulations to relate loading frequency with vehicular speeds by using pulse duration along the depth results due to vertical compressive stress pulse. He found that dynamic modulus can predict HMA pavement performance with varying speeds. He used a falling weight deflecto-meter to estimate in-situ dynamic modulus of undamaged pavements and developed a factor for their conversion and found that it agreed with the trends generally found by field measurements.

Ameri et al. (2014) evaluated and compared several methods to find flow number parameter which is an indicator of rutting performance of HMA pavements and onset of tertiary flow. Permanent deformation data from twelve (12) different mixtures was obtained and after the comparison based on variability in flow number values he recommended franken model as best method to find flow number with limited variability.

Khosravifar et al. (2015) used time temperature superposition principle to construct master curves for repeated load permanent deformation test on three temperatures low, medium and high as recommended by NCHRP. He conducted dynamic modulus and repeated load axial deformation tests to find temperature shift factors using the results of dynamic modulus and apply these shift factors to the results of permanent deformation repeated load test to achieve a smooth master curve and avoid time consuming material characterization. This confirmed to that time temperature superposition was also valid for results of permanent deformation repeated load test. The master curve was constructed on a plot of cumulative strain vs reduced loading cycles.

Yu et al. (2015) conducted repeated load triaxle test which was modified to check high temperature pavement performance of asphalt pavements. It could simulate confinement and temperature gradient as in actual pavements. A three-layer test specimen was prepared and was tested to evaluate the effect of different elements such as binder, temperature and mix type. This test can be performed at RLT test apparatus with little modifications to test the high temperature performance of three-layer asphalt mixes using flow number test. Main disadvantage of this test was that some extra effort was needed to create specimens for the testing.

2.8 INDIRECT TENSILE STRENGTH TEST

Tensile strength of hot mix asphalt (HMA) is very important since it is a decent pointer to confirm the HMA mixture probable of cracking. The mixture which exhibits high tensile stain demonstrates that HMA is extra probable to struggle against cracking and permit higher strains. Indirect tensile strength (ITS) of HMA mixture is carried out by loading the compacted cylindrical sample diagonally its vertical diameter plane at a standard proportion of distortion (50 mm/min) and at 25º C temperature as per ASTM standard (D6931).

The significance of ITS test is determining the potential of bituminous mixes against rutting and cracking. Specimen split when even tensile stress is along perpendicular diametrical plane and vertical to functional load (Yoder, 1975). The loading procedure offers an even tensile stress along the perpendicular diametral plane and perpendicular to the functional load. Splitting of the HMA sample is the outcome of ITS test. According to ASSHTO TP9-96, the indirect tensile strength (ITS) is determined by applying a constant rate of ram movement to failure. Tensile strength is intended as follows by means of Equation 2.23:

$$S = \frac{2P}{\pi t d} \quad Eq. 2.23$$

Where,

d = Diameter of the sample (mm),

t = Thickness of the sample (mm),

P = Peak load (KN).

The stress distribution on the vertical diametral plane ITS is shown in Figure 2-5.

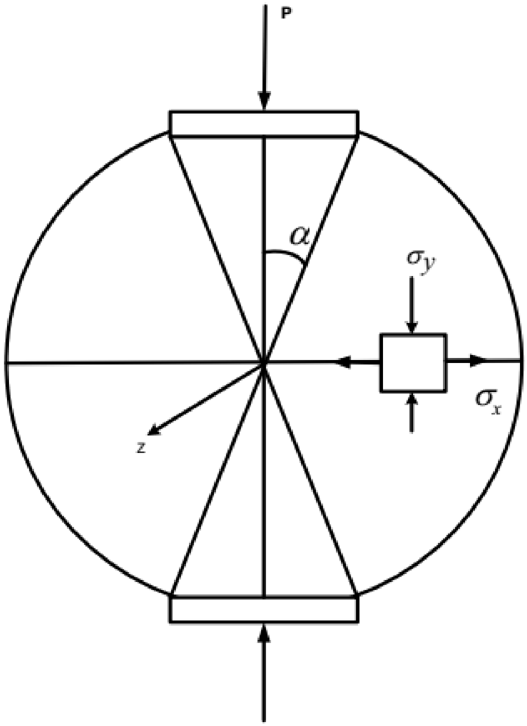


Figure 2-5: Indirect Tension Test Schematic

2.8.1 Factors Affecting the Indirect Tensile Strength Test

In indirect tensile strength test (ITS) a micro-crack is commenced at the center of the sample and spreads in the direction of the loading strips along the perpendicular direction because of the tensile stresses. Hence, the behavior of the material at the middle of the testing sample is of consideration. Meanwhile the stress and strain dispersal in the indirect tensile strength (ITS) test sample is not even, the strain at the middle of the sample is not identical to the average strain which is found by dividing measured movement by gauge length used (Kennedy, 1977).

Hot mix asphalt (HMA) samples compacted by the Superpave Gyratory Compactor (SGC) are anisotropic. The course of tensile stress in an ITS test is vertical to that in a uniaxial direct tension test. According to research it is extremely probable that HMA is not an isotropic material. Hence, it gives the impression that the anisotropicity of HMA may be a basis for difference of outcomes from the ITS test and the uniaxial direct tension creep test.

2.9 RESILIENT MODULUS

Resilient modulus (Mr) is defined as the rate of total deviator stress to the total recoverable strain under repetitive loading .it is also called the elastic modulus.

$$\text{Resilient Modulus(Mr)} = \frac{\sigma d}{\epsilon r} \qquad Eq. 2.14$$

Where,

MR = Resilient Modulus (MPA)

σd = Deviator Stress

ε_r = Recoverable Strain

The resilient modulus (Mr) is the basic parameter which help in the layered elastic theory (LEA) flexible pavament material which does not show elastic response mostly undergo permeant deformation. however, if the repetitive loading is not significant as compared to the material strength then it can recover its strain produced under repetitive loading confirming its elastic response. initially the permanent strain occurs when repetitive load is applied with the increases

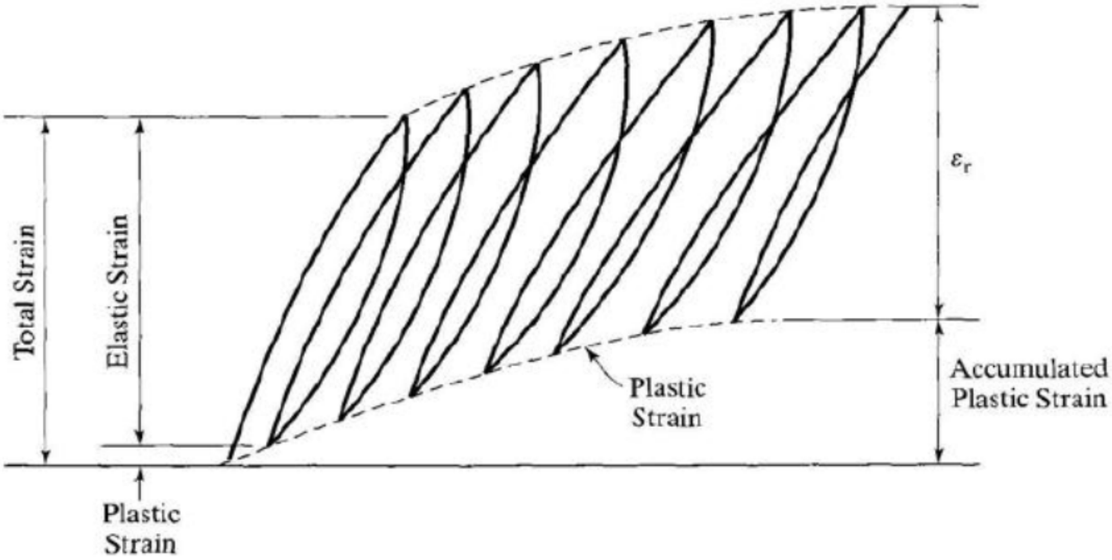


Figure 2-6: **Recoverable Strain Under Cyclic Load**

in repetitive loading plastic strain reduced under each repetition of load. After 100 load repetition the entire strain will be recovered .

2.9.1 Loading Waveform

To determine the repetitive indirect tensile strength test setup the most important factor is the loading period and type of loading pulse which must be realistic with the real field circumstances. The point of application of load is also important if the axle load is at greater distance than the point of consideration than there will be zero stress and if the axle load is right above the point of consideration then that portion will be prone to extreme stress level. The speed of vehicle dictate the use of loading duration and the thickness of point of consideration below the surface. so, normally haversine shape is used for the loading pulse (Huang, 2007).

Theoretically, the quantity of load variations w.r.t the haversine function is shown in the figure 2-7. At *t=0*, the load function can be articulated as:

$$L(t) = Qsine^2\left(\frac{\pi}{2} + \frac{\pi t}{d}\right) \qquad \text{Eq.2.15}$$

Where,

d = load period (sec)

q = Intensity of load (N)

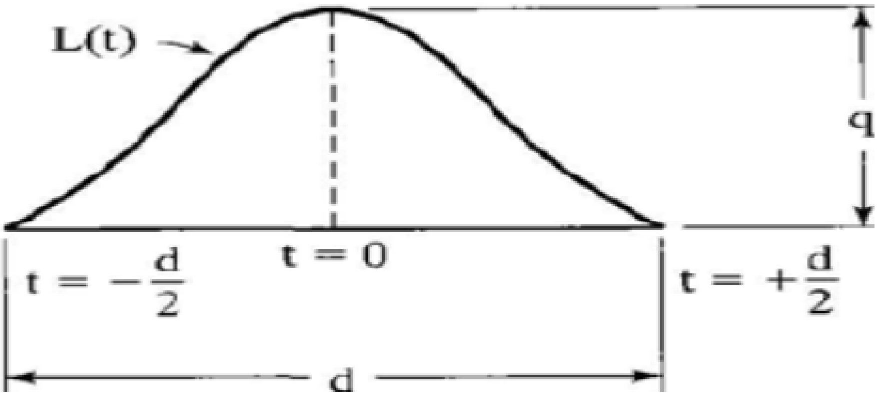


Figure 2-7: Moving Load as a Function of Time

Once the load will be at a significant distance from a stated point where t = ±d/2, the load above that point will be zero which can be articulated as L(t) = 0. Extreme load intensity (q), when the load is directly above the specified point, will be used to compute compressive stress. The loading period depends upon the speed of vehicle s and the tire interaction radius a. It is supposed that the load has almost no effect when it is at distance of 6a from the stated point or,

$$d = \frac{12a}{S} \quad eq. 2.16$$

Where,

d = period of load (sec)

s = speed of vehicle (m/s)

a = Tire interaction radius (meter)

as the vehicle speed and material depth is a serious concern during design phase, which is difficult to measure during design so it is recommended to use 0.1 sec loading period followed by 0.9 sec dwell or rest period for the resilient modulus (Mr) test in laboratory. Loading period of resilient modulus in granular material has very minimal effect whereas fine grained soil which are susceptible to moisture conditions has substantial effect of the properties of HMA. The rest period in resilient modulus test is still under discussion that either it has significant effect on HMA or not.

2.9.2 Resilient Modulus Significance

Flexible pavement design includes resilient modulus (Mr) test as a basic parameter to characterize the pavement material response. Past several years it has its prime importance in designing a new pavement structure. It indicates the elastic behavior of material used for hot mix asphalt pavament, generally this method used to calculate pavement response under the cyclic load, for pavement structure. It helps in evaluation of Layered Elastic Analysis. Asphaltic pavement layers are designed based on material elastic response and its poison ratio. However, pavament material does not exhibits purely elastic response

2.9.3 Determination of Resilient Modulus

When constant specified conditioning completed, we use first 5 cycles to calculate the average resilient modulus value. Resilient modulus is calculated from the resulting equation 2.16.

$$M_R = \frac{\{p(v + 0.2374)\}}{\delta t} \quad Eq. 2.16$$

Where,

M_R = Resilient modulus of HMA (MPa)

P = Extent of the dynamic load (N)

υ = Poisson’s ratio (assumed 0.4)

δ = Total recoverable deformation

t = Sample thickness (mm)

2.9.4 Factors Affecting the Resilient Modulus

Numerous factor affect the resilient modulus (Mr) of HMA mixes. Such as loading period, test temperature, asphalt binder grade, loading waveform, thickness and diameter of sample Nominal maximum aggregate size (NMAS) used for research work affect the properties of resilient modulus test outcomes. Plenty of investigation are carried out to analyze the effect of these parameters on the resilient modulus of hot mix asphalt combinations.

Basset et al. (1990) conducted a laboratory investigation on the effect of varying the Nominal maximum aggregate size (NMAS) and analyzed its effect rutting potential and other HMA properties. They evaluated 5 reformed HMA blends with different aggregate gradations. Nominal maximum aggregate size (NMAS) of gradations were 3/8, 1/2, 3/4, 1, and 1 1/2 in. Superpave gyratory compactor (SGC) was used to compact the samples and attain 4% air voids. 5 different blend were prepared for performance testing counting indirect tensile strength (ITS), static creep test. /Flow time (Ft), and resilient modulus to calculate stiffness. 2 different sample size was evaluated having 4-inch and 6-inch diameter. 4-inch sample was used to evaluate blend properties and mix design, while 6-ich samples were tested to analyzed to determine optimum bitumen content (OBC) for ITS test and the static creep test. The outcomes of 4-inch and 6-inch samples for different blends were examined. Outcomes of the test exhibited that blends having big aggregate were relatively strong than blends having small aggregate having comparable air voids identical to 4%.

Almudaiheem et al. (1991) determined that Indirect tensile strength percentage used as a cyclic loading in resilient modulus test affect the its properties. Researcher carried out different test varying the cyclic loading percentage ranging from 10 % to 30 % on the hot mix asphalt samples prepared under identical conditions.

Lim et al. (1995) investigated the laboratory prepared samples having 3 different sample size including 4 inch, 5 inch and 6-inch diameter. They inspected the effect of sample size on the properties of HMA mixes by resilient modulus and indirect tensile strength. They also investigated the effect of nominal maximum aggregate size (NMAS) on the mention above tests. For analyzing the effect of aggregate size, they selected 4 different aggregate blends ranging from 15.8 mm to 31.5 mm NMAS. Overall there was reduction tensile strength and resilient by the decrease in NMAS and small diameter of sample used.so It was suggested that for high stiffness parameter sample having 5 inch and 6-inch diameter with high NMAS should be used.

Loulizi et al. (2002) conducted a comprehensive research program to evaluate the vertical compressive stress pulse produced by moving heavy vehicles.so they selected 12 different station point in different areas where they will collect the data. The collect the pavament response presser cell was placed beneath the pavament surface and the specified instruments were installed on all the stations while the road was under construction. Four different speed level including 8 km / h, 24 Km / h ,40 Km / h, and 72 km / h and 5 different level pavament depth including 40 mm, 190 mm, 267 mm, 419 mm, and 597 mm were selected. A haversine function was examined to represent the real field condition by moving truck loads. Laboratory dynamic testing on HMA specimens are done by means of haversine loading pulse with 0.1 sec loading duration and 0.9 dwell period. viscoelastic material influenced by loading time. After comparison, of field and laboratory testing results it is recommended to choose 0.3 sec loading time to simulate filed conditions.

Pan et al. (2005) inspected the elastic behavior of HMA by resilient modulus test by means of indirect tensile strength. Coarse aggregate effect the resilient modulus values. They detected that by coarse aggregate having rough surface improved the resilient modulus test outcomes performed at 25°C temperature on different HMA blends.

Saleh et al. (2006) examined the different features inducing the resilient modulus. Statistical technique was used for the 2 level (High and Low) factorial design. 5 dominant input variables were considered. The input variables were: the compaction procedure, diameter and depth of sample, loading period, loading waveform and the nominal maximum aggregate size (NMAS) of blend. Outcomes of full factorial design showed that NMAS was more significant input variable which effect the resilient modulus values followed by loading period and sample geometrical parameters as depth and Diameter.

Loulizi et al. (2006) setup a detailed research program to evaluate the HMA properties. They use two testing protocol as simple performance tester (SPT) and universal testing machine UTM-25P). They considered five testing temperature ranging from 4.4 ^{0}C to 54.4 ^{0}C and six loading frequencies ranging from 0.1 Hz to 25 Hz for dynamic modulus test. for resilient modulus test one loading pulse was selected. Sample were prepared of 100 mm and 150 mm diameter to examine the testing protocol mentioned above. After the analysis, it was observed that diameter of sample has significant effect on the properties of HMA .100 mm diameter sample proved better than 150 mm diameter sample in resilient modulus test. Correlation was also found that at 5 Hz loading frequency in dynamic modulus test show resemblance in resilient modulus test results.

TJAN et al. (2013) studied the resilient modulus test in laboratory using asphalt institute method (AI) in Indonesia at single asphalt blend.to carry out resilient modulus test we need indirect tensile strength value to calculate he cycling loading which is kept 20% of the peak tensile strength value. After testing it was observed that resilient modulus values coming below 2000 MPA in lab test are 0.7-1.1 times greater than projected. And the values coming above 2000 MPA in lab are 1.19-1.6 times more than projected values.so it was summarized that eccentricity of assessed and projected values are within a satisfactory domain.so we can use these values in filed practically.

2.10 MOISTURE SUSCEPTIBLITY

Indirect Tensile Strength (ITS) test depicts properties that are useful in exemplifying moisture susceptibility if Hot Mix Asphalt (HMA). One very important property is the Tensile Strength Ratio (TSR). The TSR value shows how susceptible the HMA is to strip off or a decrease in strength under a wet habituation technique. For moisture susceptibility, samples are grouped into two (02) subgroups; tensile dry strength and tensile wet strength. The wet tensile strength samples are equated to the dry tensile strength specimens to calculate the tensile strength ratio (TSR). A high TSR, above 85% classically showed that minimal stripping is expected while a low TSR, lower than 85% classically designates that poor performance against stripping is expected. The TSR is also used to help calculate the cracking potential in HMA. Moisture damaged is common issue and it is studied and researched on the in international basis.

Hicks, Santucci et al. (2003), stated that the term moisture damage can be well-defined as the total damage in hot mix asphalt mixture of its strength along its durability Moisture related problems depends upon many factors but the two main factors are the adhesive failure and the cohesive failure. If the adhesive failure is considered, it is the because of moisture damage of the bituminous film from the aggregate surface. While if there is a loss of mixture stiffness then the failure will be termed as cohesion failure. The above two failures are highly interlinked with the aggregate, bitumen and their mutual interaction.

Coplantz and Newcomb (1988), have a researched on the moisture exposure of hot mix asphalt (HMA) concluded the following conditions which includes the saturation of a sample then again saturating and going through the single freeze thaw cycle. After the evaluation of a single freeze thaw they went through the multiple freeze thaw cycle. Indirect tensile strength ratio (TSR) and resilience modulus (MR) were also included in the test. Moisture related problems can also be caused by mix design along with the construction issues. To overcome the problem different researches and treatments are carried out, in which the addition of different polymers while preparing a mix design is also considerable.

Parker Jr and Gharaybeh (1988) , Showed the comparison and the testing procedure for the determination of the stripping potential. They concluded the tensile strength ratios (TSR) values limited from 0.7 to 0.8, but later stated that the tensile strength ratio cannot differentiate between the stripping and non-stripping aggregate combination. This statement may not be valid for all types of hot mix asphaltic combinations. The moisture damage has a great effect on the mechanical as well as the physical properties of HMA. For improving the moisture susceptibility, properly mixed designed of HMA is very necessary. By means of the polyethylene by the optimum amount of percentage in controlling the moisture damaged has been studied. It is to be notable that polymers modifiers are very useful for the mitigation of this problem. If the chemistry of the bitumen is considered, then it acts as a glue type material and has totally different behavior then the water. But if the chemical properties of polymers are considered then it can have the similarity with the bitumen. Moisture damaged caused so many problems. Some of the moisture related distresses are discussed as under.

2.10.1 Factors Affecting the Moisture Susceptibility

The aspects affecting the values of moisture susceptibility will be addressed here. Moisture damage or better to call moisture exposure is extremely interrelated by aggregate source including additional variables that can considerably rise or decline the risk of moisture exposure for HMA Mixtures.

Harvey et al, (2002) studied that thickness of bituminous film on the aggregates and the absorptivity of the HMA mix design can be determined by several factors counting bitumen content, gradation and dust to binder ratio. It may include binder selection from the source, which governs the toughness of the bitumen and the vulnerability to infiltrate the water in bituminous film. Modification of binder using modifiers can decrease the overall vulnerability of the HMA mixture. Construction inconsistency, counting segregation, can produce areas or pockets with high air void content and low bitumen content. This phenomenon allows water to penetrate and thus the mix becomes more vulnerable to stripping. Similarly, modification from the job mix formula (JMF) can produce the same vulnerable areas .

Application of test site test outcomes will be less problematic in field, danger of removing practical mixtures and allowing vulnerable mixtures will be compact if the field adjustment is probable in terms of reflection of these factors. This must be attained in the light of a common test adjusted to native circumstances suitable to the local environment and loading conditions .

Philips and Marek, (1986) highlighted the requirement for a stripping test via a common technique. With respect to measuring performance, documentation of moisture damage can be difficult in deficient coring when it happens beneath the surface, and moisture damage that does not continue to stripping is problematic to recognize short of evidence about HMA mix presentation in the absence of moisture

For example, some researchers recognized roadways as being good, high maintenance, comprehensive rehabilitation. Aschenbrener et al (1995) placed pavement structure in the groups considering the data including pavament design life to reach its termination period and recommend coring technique to measure moisture damage to the pavament structure. Further investigators use diverse measures for good pavements. Stripping is occasionally correctly recognized and its data is rarely arrived into a database form the field projects, when they reach the failure point. Stripping itself is visually recognized. This is the case for laboratory investigation that depend on visual recognitions .

The magnitude to which openings in the aggregate engage bitumen disturbs the volume of air voids in the hot mix asphaltic mixture. If HMA air voids surpass about 8 % by volume, they possibly will develop organized and permit water to simply enter in the HMA mix. This in-turn causes moisture damage due to pore pressure. HMA mix design changes bitumen content and aggregate gradation curve in such a technique to produce design air voids of around 4 %. Same effect was used in this research study

Different techniques are applied to control over the moisture damage. When the water is entered to a bituminous specimen then it affects the internal properties of bitumen. For this reason, this problem can be overcome by applying the different methodologies. Beside these methods anti stripping additives, lime additives and the addition of the polymers is also an effective process to control the moisture and to modify the bitumen.

Ismail stated that viscoelastic property of a bitumen of hot mix asphalt (HMA) can be improved by the addition of polymers. So according to his point of view the bitumen can be modified by the addition of different kinds of the polymers. Polymers modified binders don’t have the similarity with that of unmodified so they cannot be mix up with each other.

CHAPTER 3

METHODOLOGY

3.1 GENERAL

This chapter describes the methodology implemented for this research work in detail. aggregate and bitumen binder selection, mix design method, specimen fabrication, conditioning time for the specimens, testing method and testing equipment's are discussed.

Superpave wearing course gradation was tested using simple performance testing protocols and universal testing system. The aggregate was acquired from a single source that is Margalla quarry situated in Islamabad Pakistan and ARL 60/70 binder was used. Optimum binder content was calculated using Superpave mix design technique. Superpave Gyratory Compactor (SGC) was used for the fabrication of specimens for the performance testing, fabricated specimens were then cored and trimmed to the dimensions specified for performance testing using core cutter and saw cutter. Performance testing was performed by means of Asphalt Mix Performance Tester (AMPT) and universal testing machine UTM-25P.

1st step was the mix design process and determination of OBCs. After The determination of OBCs samples were prepared for determination of optimum polymer content (OPC). After determining OPC's samples were prepared for performance testing using SGC. After preparation, the specimens were cored and sawn according to the specifications. After conditioning the samples for required time performance tests were performed and the results were obtained in the form of excel sheets that were used for further analysis.

3.2 RESEARCH METHODOLOGY

Firstly, the process of sieving is done by following the gradation curve. For the characterization of the aggregates the Abrasion value test, crushing strength of the aggregate test, Impact value of the aggregate, Flakiness test, Elongation test and the specific gravity test was performed.

Determining the properties of a bitumen used in the hot mix asphalt, the penetration test, softening point, ductility test along with the flash point and fire point tests has been carried out.

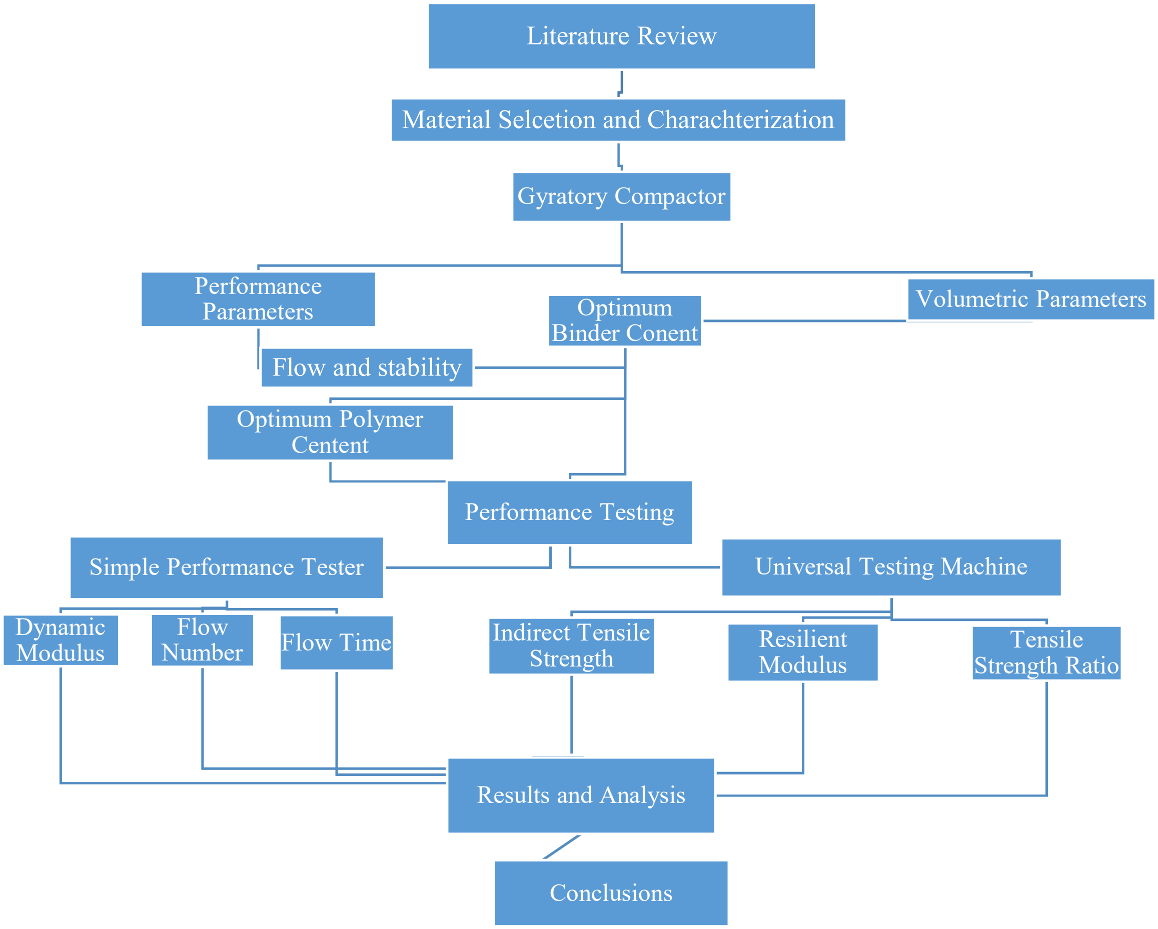


Figure 3-1: Flow Chart

The optimum binder content (OBC) was calculated by the Superpave mix design procedure. For the aggregate characterization, the Superpave gradation curve was followed. For the determination of the OBC 15 samples have been prepared of the various percentages of the bitumen (3.5 %, 4%, 4.5%, 5%, and 5. 5%).The optimum polymer content (OPC) was determined by using Superpave gyratory compactor. The addition of the polymer content for LDPE and HDPE was 0%, 4%, 6%, 8%, 10%, 12%, and 14% respectively. The research methodology is better explained with the help of flow chart shown in Figure 3.1.

3.3 MATERIAL CHARACTERIZATION

Asphaltic concrete is composed of aggregate and bitumen. Margalla crush and Attock Oil Refinery 60/70 penetration grade binder is widely used for the construction of pavement in Pakistan and is used for this research. The aggregate used for the preparation of samples was lime stone and was acquired from a single source i.e. Margalla quarry situated near Islamabad. The bitumen was ARL 60/70.

3.3.1 Aggregate Characterization

Superpave specification divides the aggregates into two properties one is consensus property and other one is source property. Consensus properties are critical for well performing of asphalt mixtures whereas source property is related with the asphalt mixture performance.

3.3.1.1 Shape Test

Flat and elongated particle is the percentage by mass of coarse aggregate particle larger than 4.75 mm sieve that has minimum to maximum dimension ratio greater than five. This criterion was developed to avoid the particles that break during construction and under traffic. ASTM standard D4791 procedure is followed for F&E particles. Result is shown in below Table 3.1.

3.3.1.2 LOS Angles Abrasion Value

This test is to determine the hardness of road aggregate. Aggregate used in road construction should be strong enough to resist the wear due the heavy traffic load. If the aggregate has high abrasion value, then stability of road pavement is to be expected to be harmfully affected. Aggregates with distinctly different origins should be expected to perform differently in this test machine. Test was performed according to the following standard: ASTM C 535 & AASHTO T-96.

3.3.1.3 Aggregate Impact Value

It gives the comparative strength of aggregates against impact loading. Toughness is the property of material to counterattack impact due to traffic loads. The aggregates are exposed to the pounding action and there is opportunity of aggregate stone breaking into lesser fragments.

The aggregate must consequently be sufficiently tough to resist breakage under impact. Test was performed according to the following standard: BS: 812 & IS: 383.

3.3.1.4 Specific Gravity and Absorption Value

Specific Gravity is the proportion of the density of aggregate to the density of water at a temperature of 23°C. a material having specific gravity of 1.0 means that density of any substance is equal to the density of water. Test was performed according to the following standard: AASHTO T: 85-88 & ASTM C 127-88.

Table 3-1: Aggregate Characterization

Test	Designation	Average Value
Elongation index	ASTM D 7491	11.24%
Flakiness Index		12.7 %
LOS Angeles abrasion value	ASTM C 535 & AASHTO T-96	22.28%
Aggregate Impact Value	BS: 812 & IS: 383	21.39%
Absorption of aggregate	AASHTO T: 85-88 ASTM C 127-88	1.29%
Specific gravity of aggregate		2.64

3.3.2 Bitumen Characterization

3.3.2.1 Grade Penetration

This test is used to determine the penetration grade of bitumen. The behavior of bituminous materials varies significantly with change in temperature. It is therefore important to use the appropriate grade of bitumen that is best suitable for the climatic conditions of the project area. The penetration of bitumen is defined as the distance in tenths of millimeter that a standard needle vertically penetrates in a sample of bitumen under known conditions of loading, time and temperature. (A load of 100 grams applied for 5 seconds at 25 0C is standardized for the test) A small penetration value indicates that the bitumen is hard, while the high penetration value indicates that the bitumen is soft.

3.3.2.2 Softening Point

Softening point is an important factor describing the behavior of bitumen material. When softening point reaches bitumen cannot sustain its shape so it starts flowing under a weight of a 3.5 grams' steel ball in ring ball apparatus used to measure the softening point. This test was performed by standard specification of ASTM D 36. Two brass rings containing the bitumen sample are placed in ring ball apparatus. Then we heat the bitumen sample by means of distilled water and a steel ball is placed above the sample. when it starts softening both the balls will pass through the ring and touches the disk below at the same time. the temperature at which it touches the disk is called softening point.

3.3.2.3 Flash and Fire Point

Flash and fire point is an important test to insure the safety requirements within the range at job site. Flash point is the temperature at which it gives off vapor and show a spark at bitumen surface while fire point is the temperature at which the flame keep burning for 5 sec. at high temperature liquid is transformed into the vapors. bitumen is a byproduct of petroleum so when it reaches that temperature it also gives off vapor at its surface which are ignitable when it meets the fire flame. Cleveland open cup is used to determine the flash and fire point. cup is filled with bitumen and placed over heater and record the temperature at regular interval .is surface is exposed to fire flame at the same time when temperature is recorded. the moment a spark is observed is the indication of flash point and temperature at which this spark is observes is recorded as flash point temperature.as per NHA specification flash point should be 232 oC and fire point should be 242 oC. This test was performed by ASTM D 92 standard.

3.3.2.4 Ductility Test

Ductile material is preferable as compared to brittle material because ductile material gives warning before failure .so ductility is also important to measure for the bituminous material. ductility test of bitumen is performed as per ASTM standard specification of ASTM D 113. standard briquet mold is used having a shape like dog bone. Water bath maintaining a temperature of 25°C is used. Briquet mold is filled with bituminous sample and placed in the ductility apparatus. It's one end is fixed and other end moved at rate of 5cm/min. Machine pulled the free end up to the limit when the cross-sectional area of bituminous thread equals the zero or negligible is recorded as ductility value in cm. Table 3-2 displays the test results values performed on the bimanous samples.

Table 3-2: Bitumen Characterization

Test	Designation	Average Value
Flash and Fire point	ASTM D 92	243 and 289
Softening point	ASTM D 36	42.62
Grade penetration test	ASTM D 5	62.9
Ductility test	ASTM D 113	100

3.4 GRADATION

Accurate proportioning of different sizes of aggregate according to the gradation specification is vital. This study focuses on Superpave wearing course gradation. Aggregate gradations are shown in tabulated form in Table 3-3 as shown below.

Table 3-3: 19 mm NMAS Gradation Curve

Sieve Size		Control Points		Restricted Zone		%
(mm)	(U.S.)	Lower	Upper	Lower	Upper	Pass
25	1 inch	100				100
19	3/4 inch	90	100			95
12.5	1/2 inch		90			83
9.5	3/8 inch					65
4.75	No. 4					46
2.36	No. 8	23	49	34.6	34.6	30
1.18	No. 16			22.3	28.3	20
0.6	No. 30			16.7	20.7	13
0.3	No. 50			13.7	13.7	9
0.15	No. 100					6
0.075	No. 200	2	8			5

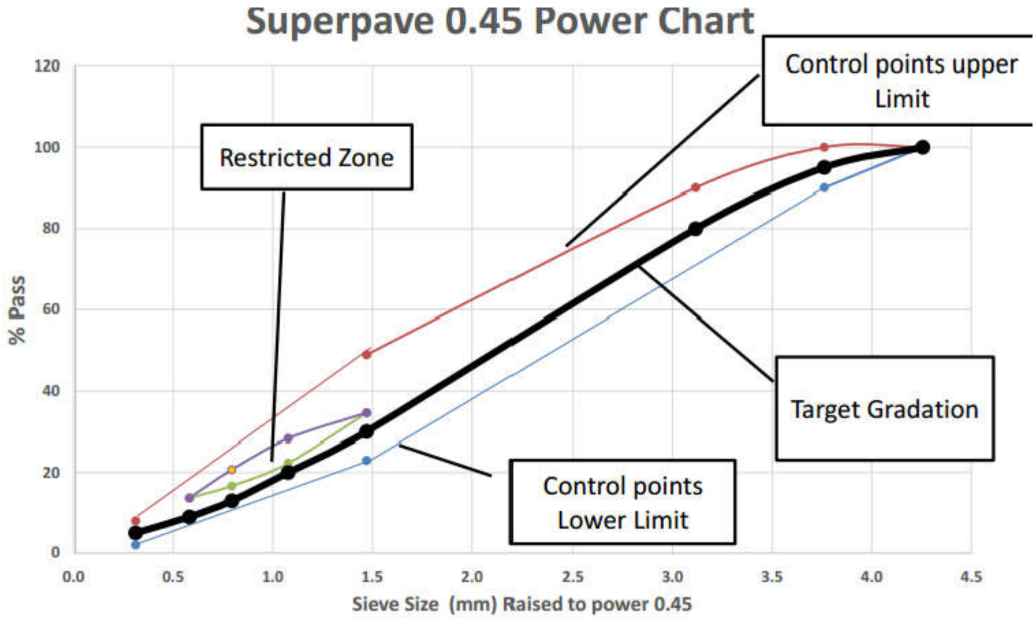


Figure 3-2: Superpave Gradation Chart

3.5 Sample Preparation

One of the initial steps in preparation of specimens for performance testing is determination of Optimum Binder Content (OBC) and determination of optimum polymer content (OPC). Superpave mix design procedure was used for determination of OBC and OPC's.

3.5.1 Optimum Binder Content

Specimens were fabricated according to the specification ASTM D6929, Standard Exercise. Preparation of Samples by means of Superpave Gyratory Compactor (SGC) 6-inch diameter specimens were used for the wearing course. Figure 3.3 represents specimen prepared for OBC for wearing course mixtures. Triplicate specimen was prepared for each binder percentage.

3.5.2 Optimum Polymer Content

After the determination of Optimum binder content (OBC) optimum polymer content (OPC) was determined by replacing the bitumen with HDPE and LDPE by 4%,6%,8% 10% ,12% and 14% by weight of bitumen content .sample were prepared in accordance with Superpave mix design procedure as used for OBC samples.

3.5.3 Performance Testing

Three simple performance tests (SPT) consisting of Dynamic Modulus |E*| test, Flow Number (Fn) test and Flow Time (Ft) test were carried out. These performance tests are discussed in detail in the following sections.

3.6 DYNAMIC MODULUS TEST

It is used to characterize hot mix asphalt by evaluating its visco-elastic behavior and stiffness properties. This test is performed by applying a haversine stress pattern and from the induced strains dynamic modulus is calculated. It is the absolute value of complex modulus mathematically,

$$|E \quad| = \frac{(\sigma_o)}{(\varepsilon_o)} \qquad Eq.3.1$$

Where,

|E*|=Dynamic Modulus

σ_o=max stress that is applied dynamically

ε_o= max strain that is produced axially

Complex modulus consists of a real and an imaginary part. The real part represents the elastic stiffness of hot mix asphalt and the imaginary part describes of HMA viscosity. These components are mathematically written as under,

$$E = E' + iE'' \qquad Eq.3.2$$

Where,

E=Complex Modulus

E' = Elastic Stiffness

E" = viscous modulus

For perfectly elastic materials, viscous modulus is zero i.e. E" =0, so the above equation becomes:

$$E = E' \qquad Eq.\ 3.3$$

From above equation, it is evident that when E” =0, the dynamic modulus is equal to elastic modulus as shown below:

$$E \ = \sqrt{(\frac{\sigma^o}{\varepsilon^o}\cos\phi)^2 + (\frac{\sigma^o}{\varepsilon^o}\sin\phi)^2} \qquad Eq. \quad 3.4$$

Where,

E* = dynamic modulus expressed in lb/seq in

σo = peak dynamic stress (psi)

εo= Peak Recoverable Axial Strain (με)

φ = phase angle (radians)

Phase angle shows the viscoelastic characteristics of the mixture. It is the angle by which the compressive dynamic stress is ahead of induced axial strains (Witczak 2002).

$$\theta = \frac{T_i}{T_p} \times 360 \qquad Eq. 3.5$$

Where,

Ɵ = Phase Angle

Ti = lag between a cycle of strain and stress (seconds)

Tp = stress cycle (seconds)

In a perfectly elastic material Ɵ= 0º and for perfectly viscous material Ɵ= 90°. Dynamic modulus is described by angular velocity ω and time t shows that the phase angle presents the time dependence of hot mix asphalt (HMA) as shown in Figure 3.3. Equation 3-6 shows the relation between angular frequency and the loading frequency.

$$\omega \ = 2\pi f \qquad Eq.\ \ 3.6$$

Where,

f = loading frequency (Hz)

ω= angular frequency (rad/sec)

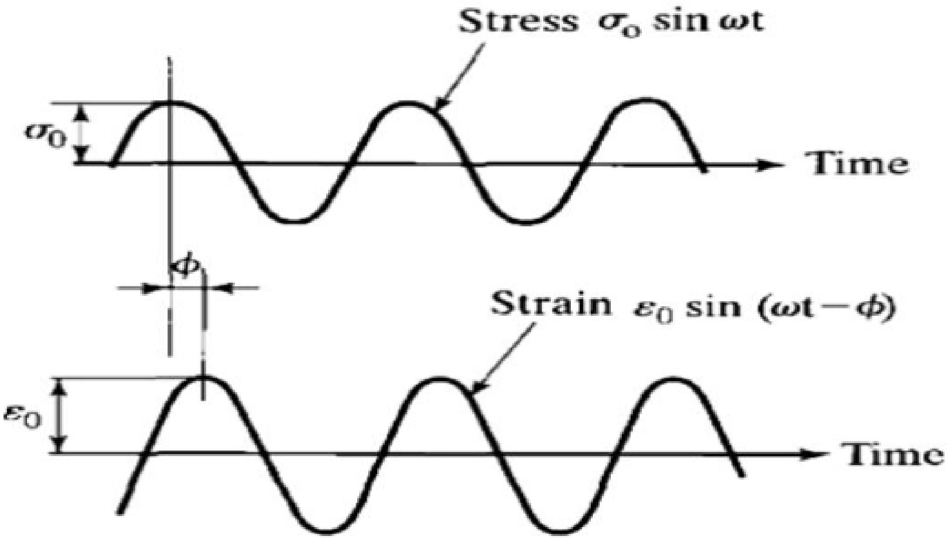
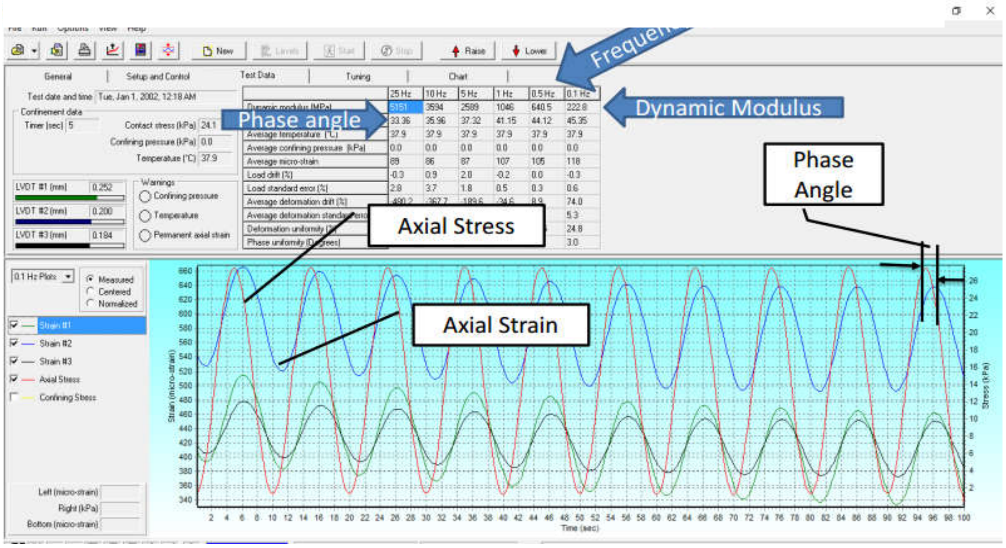


Figure 3-4: Dynamic Modulus Test Mechanism



Test date and time: Tue, Jan 1, 2002, 12:18 AM

Confinement data

Timer (sec): 5

Contact stress (kPa): 24.1

Confining pressure (kPa): 0.0

Temperature (°C): 37.9

LVDT #1 (mm): 0.252

LVDT #2 (mm): 0.200

LVDT #3 (mm): 0.184

Warnings

○ Confining pressure

○ Temperature

○ Permanent axial strain

	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
Dynamic modulus (MPa)	5151	3594	2589	1046	640.5	222.8
[illegible]	33.36	35.96	37.32	41.15	44.12	45.35
Average temperature (°C)	37.9	37.9	37.9	37.9	37.9	37.9
Average confining pressure (kPa)	0.0	0.0	0.0	0.0	0.0	0.0
Average micro-strain	89	86	87	107	105	118
Load drift (%)	-0.3	0.9	2.0	-0.2	0.0	-0.3
Load standard error (%)	2.8	3.7	1.8	0.5	0.3	0.6
Average deformation drift (%)	-490.2	-367.7	-189.6	-24.6	8.9	74.0
Average deformation standard error	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.3
Deformation uniformity (%)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	24.8
Phase uniformity (Degrees)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.0

Figure 3-3: Dynamic Modulus Software Output

In Asphalt Mix Performance Tester, we select the frequency and temperature at which the test is performed and test outcomes are obtained in the form of dynamic modulus and phase angle. A typical software output for dynamic modulus test is shown in Figure 3-4:

AASHTO standard for dynamic modulus is TP 62-07(AASHTO, 2007) which is now very popular method for laboratory evaluation of dynamic modulus. In this method, a sinusoidal axial compressive stress is applied to the sample and resulting strain is measured and dynamic modulus is calculated. This test is performed at 6 various frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz) and 4 various temperatures (14, 40, 70, 100 and 130 ºF). Strains are measured using LVDTs attached to the specimen.

3.6.1 Developing Dynamic Modulus |E*| Master Curves for HMA mixes.

Master curves are used as a material input in the MEPDG software for the structural design of pavements. They are developed by the application of time-temperature superposition (TTS) principle to E* test results. According to this principle, data at various temperatures and frequencies is shifted to a reference temperature and various curves are merged to form a smooth curve that is called a master curve. This is done by nonlinear optimization technique to minimize sum of square error using excel solver add on. Time-temperature principle is appropriate for the materials that are thermo-rheological and Hot mix asphalt mixtures are assumed to be thermorheological (Ekingen 2004).

Data at different frequencies and temperatures is shifted with the help of a shift factor a(T). Reduced frequency *fr* can be determined by dividing the actual frequency by the shift factor a (T) as represented by equation 3-7,

$$\mathrm{f}_r = \frac{\mathrm{f}}{\mathrm{a(T)}} \qquad Eq.\ 3.7$$

Where,

fr= Reduced frequency

f = Actual frequency

a(T)= shift factor

Sigmoidal function is used for the representation of Master curves due to its S-shape and two asymptotes. Equation 3-8 represents a sigmoidal function.

$$\left[\log|E\ | = \delta + \frac{a}{1 + e^{\beta+\gamma(\log Tr)}}\right] \qquad Eq. 3.8$$

Where,

δ= minimum |E*

δ+ a= maximum |E*

β,γ = shape parameters

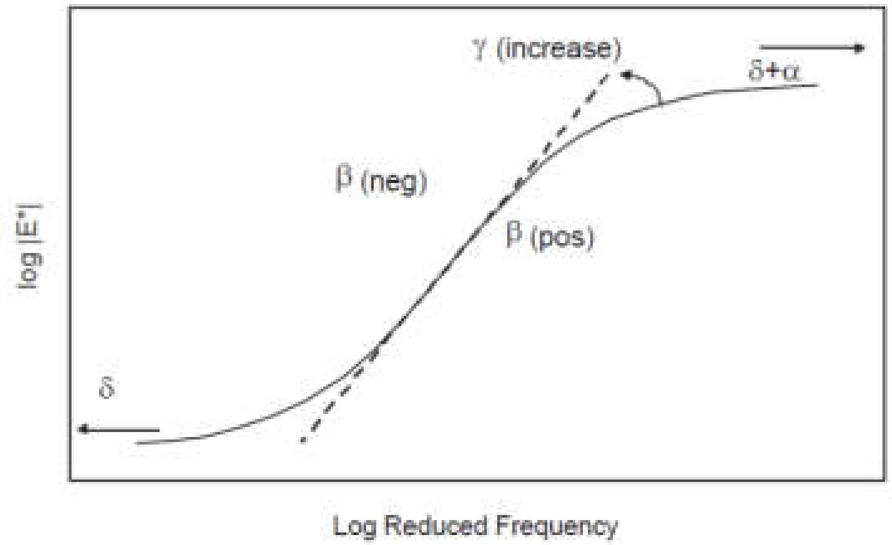


Figure 3-5: Master Curve Shape Parameters (Witzak 2002)

The shift factor can be written as shown in the following equation 3.9:

$$\mathrm{a}\ (\mathrm{T}) = \frac{\mathrm{t}}{\mathrm{tr}} \qquad Eq. 3.9$$

Where,

a(T)= shift factor

T = reduced time (Seconds)

tr= reference temperature

t = time of loading

For increasing the accuracy, a 2nd order polynomial equation among the logarithm of the shift factor and the temperature is used as shown in equation 3.10:

$$\log a(Ti) = aT_i + bT_i + c \qquad Eq. 3.10$$

Where,

a (Ti)= shift factor

T_i = temperature of interest

a, b, c =coefficients

3.7 FLOW TIME TEST

The Flow Time (FT) test is also called as static creep test. It is used by researchers to evaluate the basic characteristics of asphaltic concrete related to rutting performance. This is usually achieved by applying a static stress level to the sample and induced deformations are determined. These induced strains are used to assess the visco-elastic behavior of HMA. The test seeks the visco-elastic behavior of an HMA specimen due to a static stress level. The induced compliance, D(t), can be determined by dividing the induced strain by applied stress

$$D(t) = \frac{\varepsilon_t}{\sigma_o} \qquad Eq.3.11$$

Where,

ε_t = measured strain

σ = applied stress

When compliance is plotted against time on a log-log scale, the resulting graph is divided into three flows; first one is primary flow, secondary flow and tertiary flow presented in Figure 3-6:

3.7.1 Primary Zone:

Primary zone is observed at the start of the test when the strain rate lessens rapidly under static load and becomes stable.

3.7.2 Secondary Zone:

Secondary zone is the strain rate stays nearly unchanged.

3.7.3 Tertiary Zone:

Tertiary Zone is where the strain rate again starts increasing rapidly. A graph containing log of compliance as ordinate and log of time as abscissa can be used to determine flow number because the point where rate of change is minimum is clearly visible in this graph. It is the point where tertiary flow zone starts.

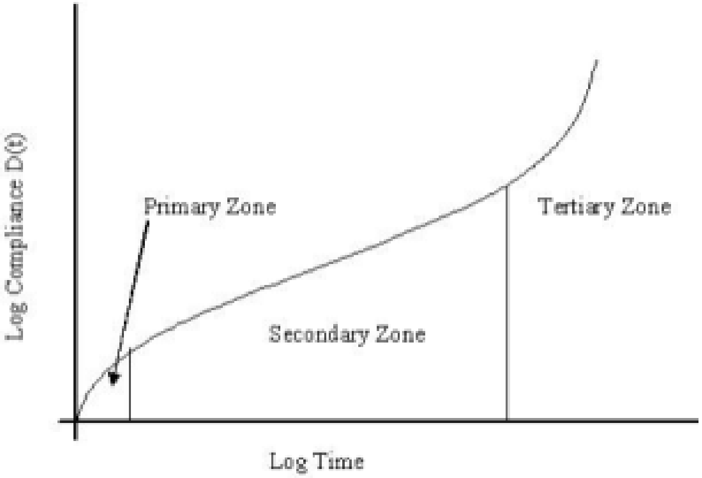


Figure 3-6: Flow Zones (Witzak 2002)

In general, the overall compliance in the secondary zone at any specified time, D(t), can be articulated as a power function as follows:

$$D(t) = at^{m} \qquad Eq.\ 3.12$$

Where,

t = time (sec) and

a, m = regression constants

In order to determine the regression constants a log-log scale graph of compliance vs time was plotted in the secondary zone. As shown below in Figure 3.6

$$\log D(t) = m \log t + \log a \qquad Eq. 3.13$$

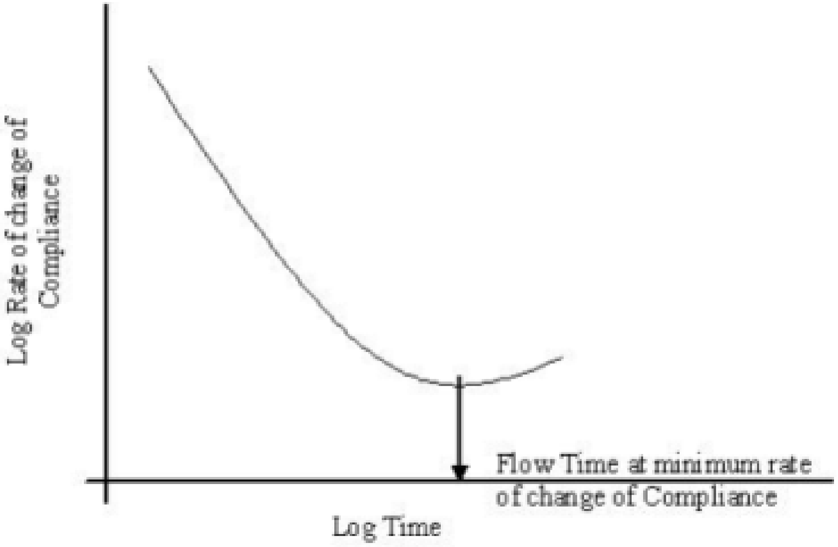


Figure 3-7: Flow Time Determination (Witzak 2002)

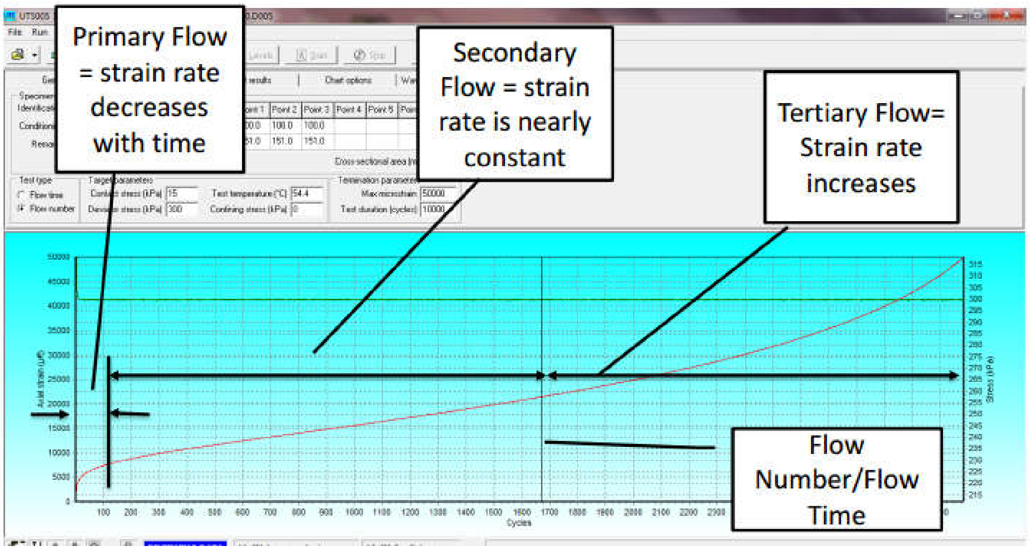


Figure 3-8: Software Output of Flow Time

When conducting a flow time test on Asphalt Mix Performance Tester we select a stress level and a temperature at which test is performed, test termination strain and test termination cycle. A typical software snap while conducting a flow time test is given in Figure 3-8.

3.8 FLOW NUMBER TEST

The flow number (FN) test measures basic characteristics of an HMA mix related to pavement resistance to permanent deformation (rutting) performance. This is achieved by applying a predefined dynamic stress level is on to the HMA specimen with a loading period of 0.1 s pursued by a rest phase of 0.9 s at a given temperature.

As shown in Figure 3-9 three flow zones can be seen, primary, secondary and tertiary zones. In primary zone the strain rate increases slowly and reaches to point where strain rate becomes nearly constant. From this point, secondary flow zone starts with a stable strain rate. After some time, the strain rate again starts increasing rapidly, and the zone of tertiary flow starts. When the applied stress level is low it is very common to observe only primary and secondary flow zones only. Tertiary flow zone is mostly seen when the applied stress level is high.

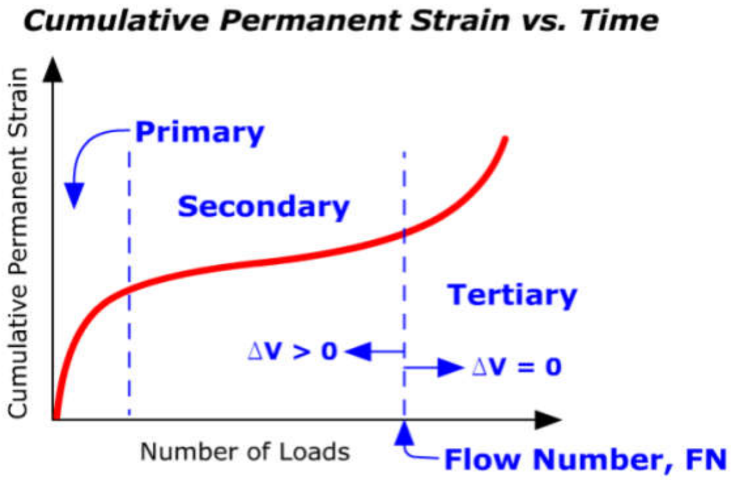


Figure 3-9: Flow Number, Flow Zones (Witzak 2002)

During the test the strain data is recorded and can be exported as an excel file. The following Equation 3.14 represents a typical model in which permanent strains are represented as a function of loading cycles

$$\varepsilon_p = xN^y \qquad Eq.\ 3.14$$

Where,

ε_p = permanent strain,

x,y = model parameters

N = number of load cycles at which ε_p recorded

3.9 INDIRECT TENSILE STRENGTH TEST

In According to ASTM 6931 standard Indirect tensile strength at 25º C of gyratory compacted specimen was conducted for the determination of tensile strength. The Specimen is placed in temperature controlled cabinet. Actuator is moved up and specimen is placed in the jig. To hold the specimen loading strip is placed over the top of core cut specimen. Actuator is moved down and makes a contact with the loading strip. The stress-strain template was opened and stress-strain test was conducted by applying compressive load of 50 mm/min at 25º C as shown in Figure 3.14 and 3.15 respectively.

Figure 3-10: ITS Test sample before and after breakage

3.9.1 Jig Setup

After the determination of indirect tension test, resilient modulus is performed on the all the remaining specimens. The test is performed by placing the core cut specimen in the jigs.

Figure 3-11: Sample in Jig Setup of UTM-25P

Firstly, the specimen is placed freely over the steel plate then jig is placed and steel plate is placed over the specimen. The yoke cross arm is moved up before the specimen is screwed with the help of jigs screw to stop specimen freely movement and LVDTS are introduce in jigs to measure the horizontal displacement of specimen. LVDTs must be placed at the center of specimen and the height is adjusted with the help of screws. Now the LVDTs must be closely linked with the specimen and pendent movement in the software is continuously is observed.

3.9.2 Loading

After the specimen is placed in the jig the whole assembly is placed in UTM-25. LVDTs were placed in the jigs and contact the specimen. Hydraulic setup is turned on with the help of virtual pendent window. The actuator is moved down to contact the steel plate bulb placed over the specimen in such a way that no load is applied to the specimen. Now the levels of LVDTS were set with the help of level controlled window. According to ASTM standard D 4123, peak load is 20% of IDT value whereas 10% was kept for seating load and 0.4 Poisson ratio was assumed for the IDT test.

Now, all the variables were put in the UTM-25 software for testing like temperature, loading pulse, seating load and passion ratio then Haversine load for was selected for testing. The specimen was placed in temperature controlled cabinet for conditioning. When the UTM-25 achieved the desired temperature and all variables were set and LVDTS lies in range the test was conducted. After 100 load pulse test the LVDTS were out of range and then levels window is open and LVDTs were set again for another load pulse testing.

3.9.3 Input Parameters

The input parameters were the same as the Tensile Strength Test. First the Tensile Strength Test was performed to see the true strength of gradation. Based on that results hit a trial method was deployed to see what force is required to input. The input parameters fed into the UTM - 25 software were as follows:

- Peak Loading Force: 2000 N with a Variation of 500 N every time
- Seating Force: 10% of Peak Loading Force
- Poisson's Ratio: 0.4
- Conditioning Pulse: 100
- Load Repetition: 1 Hz

3.10 RESILIENT MODULUS TEST

Resilient modulus test setup includes the LVDTs (linear variable differential transformer) which are used to observe the horizontal movement. It also comprises of jigs which hold the sample in testing machine. LVDTs remained fixed and used to work inside their domain. The 20% of the failure load of indirect tensile strength was used as peak loading force and 10% of the peak loading force was kept as seating force. 0.4 Poisson's ratio was supposed. By entering the target, 25°C temperature, 100 ms load pulse width, pulse reiteration period (1000 ms), and habituation pulse count, test arrangement initiated and sample was exposed to haversine loading.

When the conditioning stage starts load force and its respective horizontal movement is documented by indirect tension modulus software tool. when 100 habituation pulses were counted the LVD's level were checked if they were out of range. To complete the test procedure

5 continuous pulses were applied attaining nearly same deformation. The data of these last 5 pulses were documented and its mean value was used to calculate the resilient modulus of HMA.

3.11 MOISTURE SUSCEPTIBILITY TEST

Evaluation of the moisture susceptibility of a bituminous mixture can be done using the ITS test (IDT) and is used as a performance test for its calculation. TSR is a measure of Moisture susceptibility. It is defined as the proportion of the tensile strength of water conditioned specimen, (ITS wet sample conditioned at 60 ˚C for 24 hr.) to the tensile strength of unconditioned specimen (ITS dry sample) which is expressed as a percentage (%). TSR value characteristically designates the mix that will achieve favorably with a strong confrontation to moisture damage. A high TSR value, above 85% normally indicated a good pavement performance with slight stripping predictable while a low TSR, lesser than 85% classically designates a poor pavement performance with more stripping predictable. The TSR is also helps to assess the cracking potential in bituminous mix. The rest of the testing and analysis procedure is very same as that of the ITS.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

The Test results for the control and modified HMA mixes are analyzed using different techniques presented in this chapter. Software's used for analysis includes Microsoft excel, SPSS PSAW and MINITAB. Results are presented in the forms of tables and graphs and are divided into main two phases as simple performance testing and universal testing system. There are 3 test in each phase as Dynamic Modulus test, Flow Number (Fn) test, Flow Time (Ft) test and Indirect Tensile Strength (ITS), Resilient Modulus (Mr) test and Tensile Strength ratio (TSR).

4.2 OPTIMUM BINDER AND POLYMER CONTENT RESULTS

Determination of Optimum binder content is an important parameter.so in first step OBC was determined and checked against the wearing course specifications. The results of the OBC is presented in Table 4.1

Table 4-1: Optimum Binder Content Results

Binder	Gmb	Gmm	% Gmm @Ndesign	%Gmm @ Nini	PAV	VMA	VFA	Flow	Stability	P200/Pbe
%								mm	KN	
3.5	2.34	2.50	93.64	87.28	6.36	14.59	56.84	7.70	27.4	1.44
4	2.36	2.48	94.96	88.29	5.04	14.53	65.79	8.88	31.96	1.26
4.5	2.37	2.45	96.78	88.57	3.22	14.39	77.93	9.21	32.5	1.12
5	2.39	2.44	97.77	89.21	2.23	14.32	84.55	8.92	34.95	1.01
5.5	2.40	2.42	99.24	91.09	0.76	14.20	94.62	11.44	32.56	0.91
4.36	**2.37**	**2.46**	**96.27**	**88.49**	**4.00**	**14.43**	**74.53**	**9.12**	**32.35**	**1.16**

2nd step is to determine the optimum polymer content (OPC) for each modifier as HDPE and LDPE.OPC is selected at maximum stability value which is presented in Table 4.2 for HDPE and Table 4.3 for LDPE

Table 4-2: Optimum Polymer Content Results for HDPE

HDPE	**Gmb**	**Gmm**	**% Gmm**	% Gmm	**VMA**	**VFA**	**Stability**	**Flow**
%			**@Ndes**	@Nini			**KN**	**mm**
4	2.35	2.44	96.29	87.97	15.04	75.35	33.33	9.52
6	2.34	2.44	95.72	87.41	15.55	72.66	33.62	9.48
8	2.34	2.44	95.74	87.97	15.53	72.59	33.64	8.68
10	**2.34**	**2.43**	**96.09**	**88.48**	**15.57**	**74.89**	**34.56**	**8.80**
12	2.29	2.43	94.05	86.64	17.36	66.97	31.36	9.16
14	2.30	2.41	95.52	87.73	16.76	73.46	31.13	9.00

Table 4-3: Optimum Polymer Content Results for LDPE

LDPE	**Gmb**	**Gmm**	**% Gmm**	**% Gmm**	**VMA**	**VFA**	**Stability**	**Flow**
%			**@Ndes**	**@Nini**			**KN**	**Mm**
4	2.37	2.45	96.53	89.32	14.48	76.05	37.31	9.21
6	2.37	2.45	96.90	89.56	14.16	78.10	37.91	9.15
8	2.37	2.45	96.90	89.28	14.16	78.21	38.07	8.50
10	2.36	2.44	96.91	89.12	14.50	78.69	38.55	9.02
12	**2.35**	**2.44**	**96.35**	**89.03**	**16.38**	**70.48**	**40.18**	**8.27**
14	2.35	2.44	96.50	88.87	14.86	76.43	38.42	6.07

4.3 ENERGY-DISPERSIVE X-RAY SPECTROSCOPY (EDX) RESULTS

Chemical composition of HDPE and LDPE is analyzed by EDX test. the results are shown in figure below. LDPE has high percentage of $caco_3$ which shows high performance. Calcium carbonate is a chemical name of lime. addition of lime in asphalt mixes increases the stiffness of mixes.

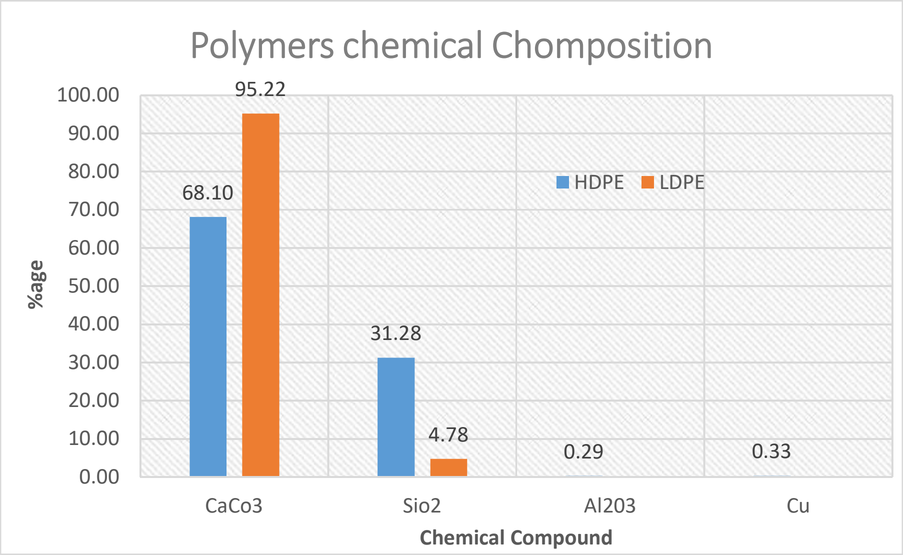


Figure 4-1: Polymers Chemical Composition

4.4 DYNAMIC MODULUS TEST RESULTS

Dynamic modulus of asphalt mixture displays that the dynamic stiffness of polymer modified mixtures were slightly higher as compare to Controlled mixture. Results of polymer modified HMA and Control mixtures were compared. The comparison was done on the results obtained on all corresponding frequencies at given temperature. Significant increase in dynamic modulus was noted for polymer modified HMA with increase in temperature. These curves depicted a drop in dynamic modulus values with increasing temperature. This is because as the temperature increases the stiffness of the mix decreases and more strains are produced in response to the same applied stress resulting in a decreased dynamic modulus value. This can be better visualize by Isochronal curves shown in Figure 4-2. Isochronal curves are drawn at different temperatures. Another noticeable observation can be seen at highest test temperature

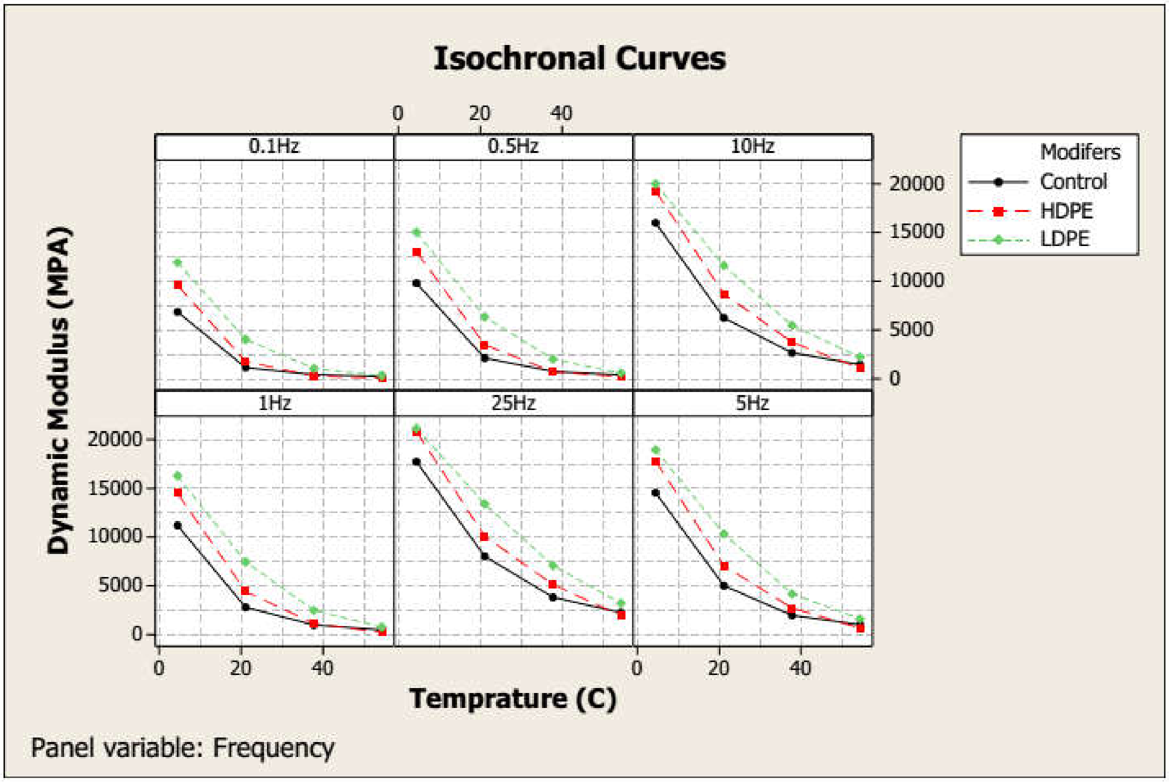


Figure 4-2: Effect of frequency on dynamic modulus of asphaltic mixtures

that is 54.4ºC. At 54.4ºC the curves for both the binders come closer so that the change in dynamic modulus values decreased for both binders at higher temperature. It was also noted that dynamic modulus test results are more sensitive at higher temperature and have higher coefficient of variation than at lower temperatures for almost all the mixes. So, it is necessary to take great care and avoid errors while conducting the dynamic modulus test at higher temperatures.

Dynamic Modulus values were increased with increased loading frequency because as the frequency increases, loading time decreases producing lesser strains due to linear visco-elastic nature of hot mix asphalt in which stress strain relationship also depends on loading duration. This effect is shown with the help of isochronal curves representing dynamic modulus loading frequency relationship at constant temperature. Figure 4-3 represents Isothermal curves for modified and control mixtures. Dynamic modulus value rises with increase in frequency. The trend is almost similar Polymer modified and controlled sample. overall LDPE has higher dynamic modulus as equated to control mixtures and slightly better than HDPE

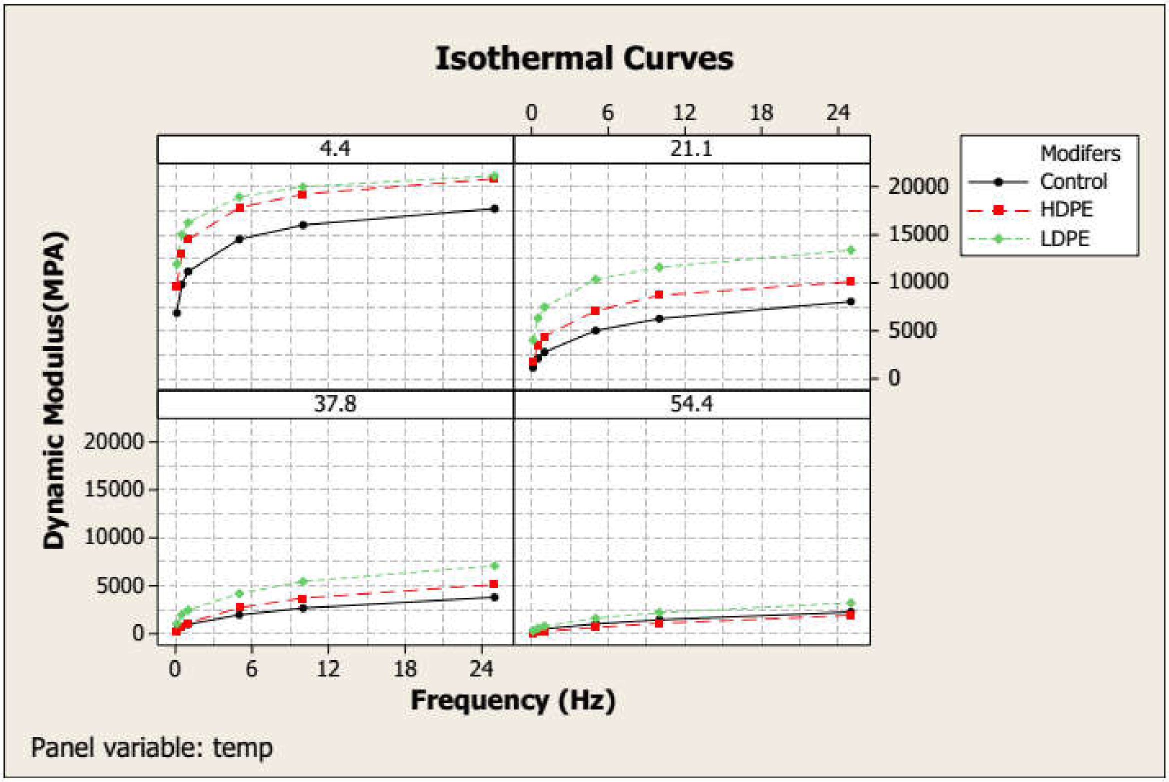


Figure 4-3: Effect of temperature on dynamic modulus of asphaltic mixtures

4.4.1 Master Curves Development

Master Curves were developed using the dynamic modulus test results which are helpful in determining pavement behavior while designing process and are used as a material contribution in Mechanistic-Empirical Pavement Design Guide (MPEDG) software. These curves are developed using time-temperature superposition principle by the help of Master solver excel sheet which is produced as a part of NCHRP Project 9-29 (Bonaquist 2008). It is developed by using the concept of minimizing the sum of square of errors using the MS Excel solver add in tool to best fit the curve. This excel tool utilizes the sigmoidal function to build the master curves. For development of master curve, a reference temperature is selected for example I our case this reference temperature is 21.1°C and data at other temperatures is shifted with respect to reduced frequency till they all combine into a single smooth function. The amount of shift represented by shift factor shows the temperature dependency of the material. Figure 4-4 presents master curves for control, HDPE modified and LDPE modified mixtures.

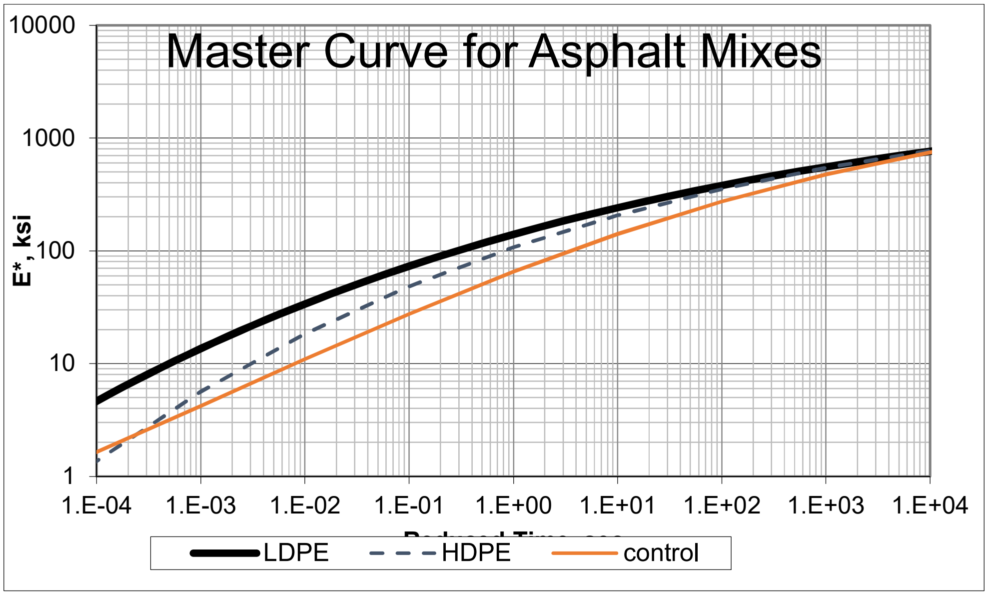


Figure 4-4: Master Curves for asphaltic mixtures

At low temperature presented by higher reduced frequency, master curves for almost all the mixtures merge at a single point irrespective of the binder penetration grade and gradation. as the temperature increases shown by frequency decrease in master curves a drop in dynamic modulus master curves can be seen. As the highest test temperature reaches the master curves are merged once again. But this time this merging of the master curves is attributed to aggregate gradation and aggregate interlock. Because at higher temperature the binder reaches softening point sot their role in mixture stiffness is limited by the aggregate interlock and gradation with higher aggregate interlocking are showing higher stiffness as compared to aggregate gradations with lower aggregate interlocking properties. At intermediate temperatures mix stiffness is governed by combined interaction of binder's stiffness and aggregate.

4.5 FULL FACTORIAL DESIGN OF DYNAMIC MODULUS

The statistical analysis of dynamic modulus data for each stage was carried out with factors i.e. frequency, test temperature and modifiers percentage each with two levels. Therefore, 2_3 full factorial design of experiment was performed using MINITAB-15 software. Table 4-4 shows the factors that have been considered in the factorial design with their high and low levels and abbreviations for both stages.

Table 4-4: Factors for Factorial Design

Notations	Parameters	Low	High	Units
A	Temperature	4.4	54.4	$_{o}$C
B	Frequency	0.1	25	Hz
C	Modifiers	0	12	%

4.5.1 Effects and Coefficient Table

Table 4-5 shows the effects and coefficients values obtained by Minitab15 software for the significant effects. The factors and interaction of factors with high (negative or positive) values of effects and coefficients indicate that they have a greater impact on dynamic modulus. The effect of each term is equal to the twice of coefficient. The factors or interaction of factors with P- value greater than significance level indicates that these main effects and two way interactions are notable and have greater effect on dynamic modulus at significance level of 5%. Also for each gradation the calculated value of t-statistic for the terms greater than the critical value of t-statistic (t critical= 2.05 for degree of freedom 23 and 5% significance level) shows that the interactions and main effects are significant.

Table 4-5: Effects and Coefficients Table

Term	Effects	Coefficient	SE Coefficient	T-Test	P-Value
Constant		7257	361.6	20.07	0.000
Frequency (HZ)	-14917	-7459	485.0	-15.38	0.000
Temperature (C)	5440	2720	431.8	6.30	0.000
Modifiers (%age)	2618	1309	400.6	3.27	0.002
Frequency * Temperature	-3248	-1624	579.2	-2.80	0.007
Frequency * Modifiers	-1985	-993	537.3	-1.85	0.069
Temperature * Modifiers	346	173	478.4	0.36	0.719
Frequency * Temperature * Modifiers	429	214	641.7	0.33	0.739

R-Sq = 85.59%, R-Sq (pred) = 81.60%, R-Sq (adj) = 84.01%

4.5.2 Significance Effects and Interaction Plots

The factors and interaction of factors, which are most significant and affect dynamic modulus of asphalt mixtures, are also shown in terms of Normal probability plot and Pareto plot generated using Minitab 15 software. Figure 4-5 shows the Pareto plot of prepared mixtures having a reference line with red color which shows that the main effect and two way interactions beyond this reference line are significant and have greater effect on the dynamic modulus. The main effects frequency, temperature and the 2-way interactions of frequency and temperature are significant and have greater influence on dynamic modulus of prepared mixtures at 5% significance level. The other plot is the normal probability plot which also shows the significant main effect and two-way interaction as shown in figure 4-6 respectively.

In the normal probability plot the factors or interactions away from the reference line are significant at 5% significance level and the factors which are near the reference line or on the reference line, are insignificant.

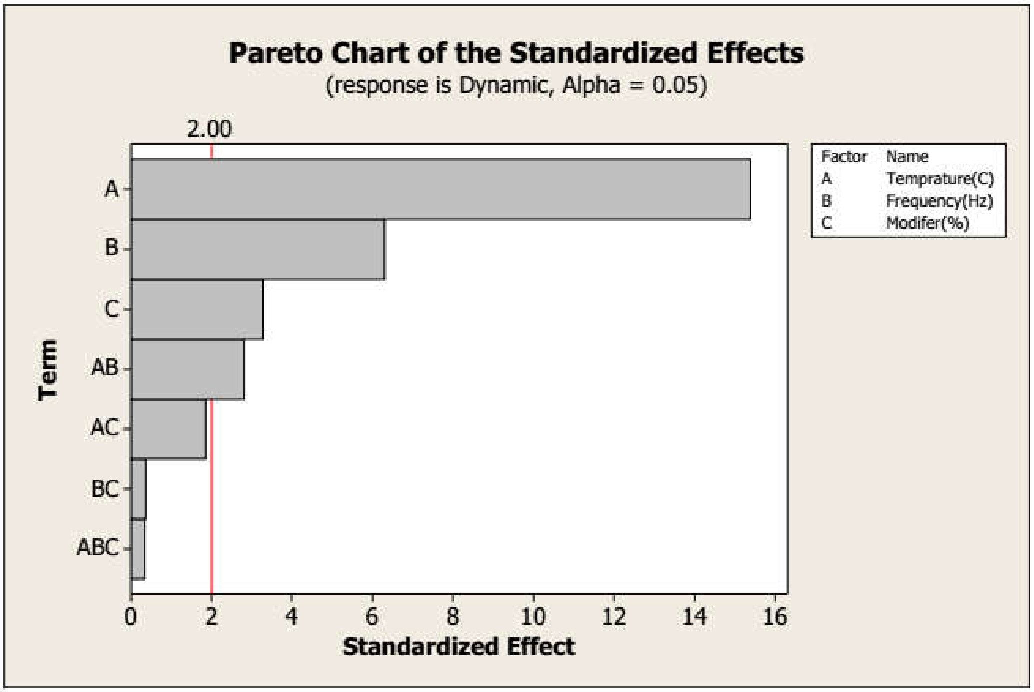


Figure 4-5: Pareto Chart of Asphaltic Mixtures

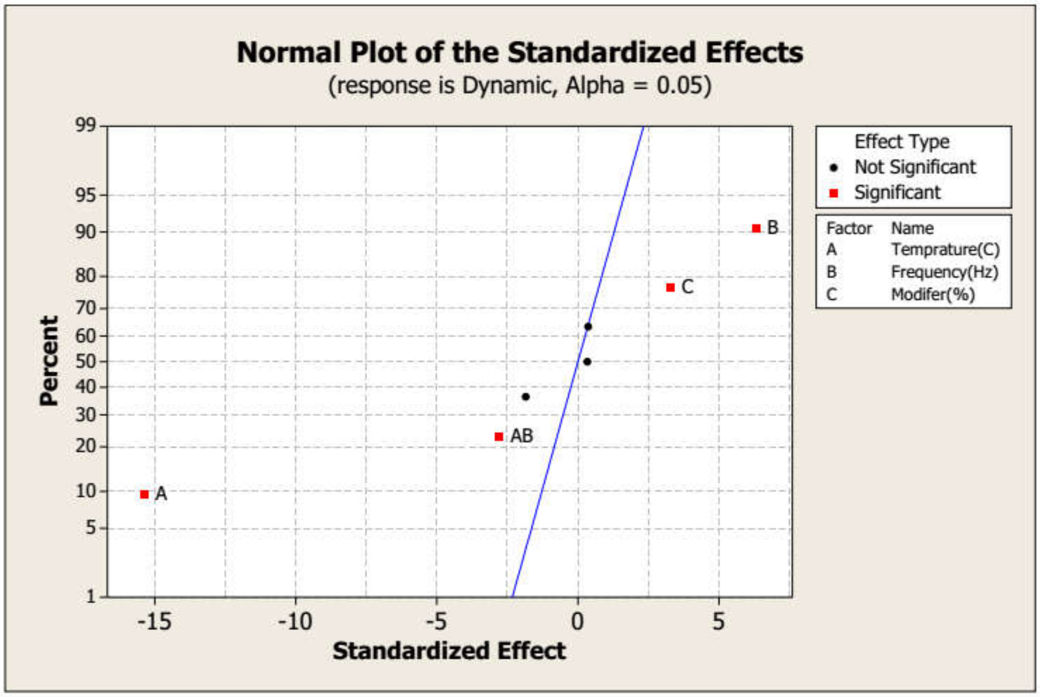


Figure 4-6: Normal plot of asphaltic mixtures

4.5.3 Factorial Plots

The interaction and significant effects obtained from the Pareto chart and Normal Probability Chart can be described in detail by factorial plots. The effects of main factors are shown by main effects plot, 2-way interactions by interaction plots.

4.5.4 Main Effect Plots

The effects of frequency, temperature and Polymer %age are shown in figure 4-7 and 4-8. The graph between frequency and dynamic modulus reveals that with decrease in frequency the dynamic modulus also decreases the reason being that with decrease in frequency the loading duration increases and more strains would be produced and ultimately the dynamic modulus would be decreases. Dynamic modulus at 25HZ frequency is high as compare to 0.1 HZ. The graph between dynamic modulus and temperature indicates an inverse relationship i.e. the dynamic modulus decreases with increase in temperature the reason being that the stiffness of mixtures reduces with increase in temperature and the graph between dynamic modulus and polymer %age shows a very mild slope the reason being that the effect of polymer modification is less as compare to temperature and frequency.

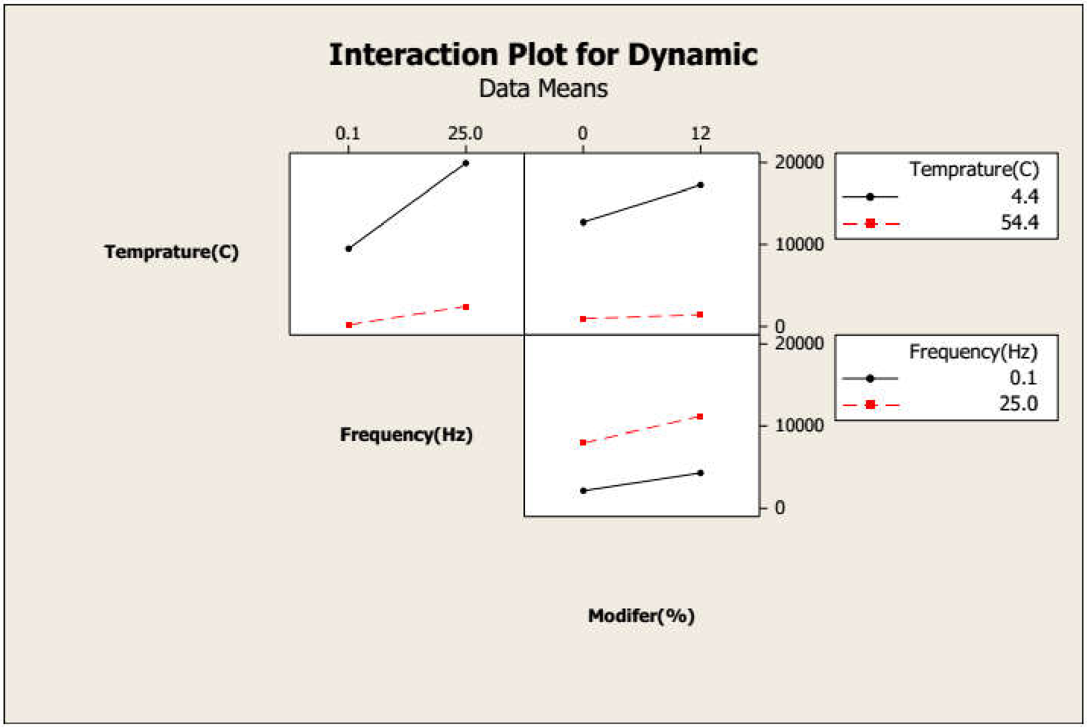


Figure 4-7: Interaction Plot for Dynamic Modulus

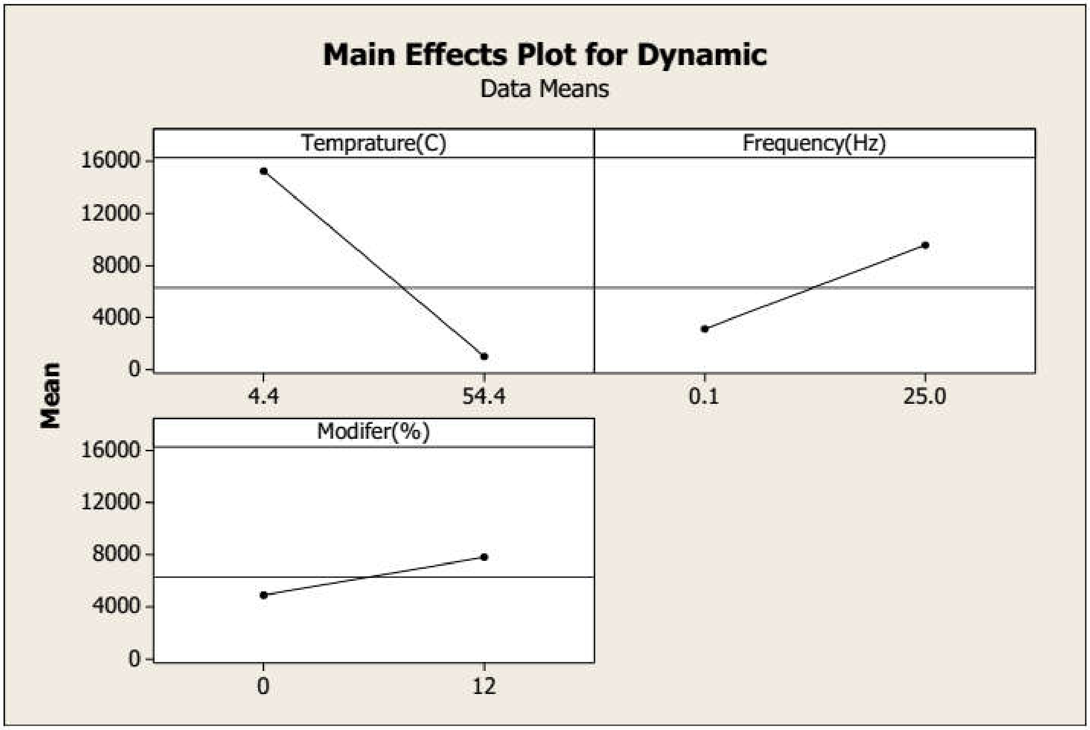


Figure 4-8: Main Effect plots

4.5.5 Analysis of Variance (ANOVA)

In Analysis of Variance ANOVA, three F-Test are made. To assess these tests, probability values are given below in table 4-6 of HMA. The P value < 0.05 designates that these tests are satisfied.

Table 4-6: Analysis of Variance Results

Source	DF	Seq SS	Adj SS	Adj MS	F-Test	P-Value
Main Effect	3	2298604763	1840082817	613360939	97.72	0.000
2-Way Interactions	3	86643444	81608442	27202814	4.33	0.008
3-Way Interactions	1	700613	700613	700613	0.11	0.739
Residual Error	64	401713963	401713963	6276781		
Total	71	2787662783				

4.6 PHASE ANGLE RESULTS

Phase angle can be well-defined as the angle by which the axial strain holdups behind the stress. An increased in phase angle was initially observed with increasing temperature and decreasing frequency but when the temperature reached up to 54.40oC the phase angle start decreasing with some exceptions as shown in figure 4-9 and 4-10 phase angle results. The graph shows that when the temperature is increased the phase angle also increases initially when reached a maximum value it starts decreasing. Phase angle and temperature are directly proportional each other at low temperature and high frequencies phase angle is usually effected by the binder and at high temperature and low frequency, the phase angle is effected by the aggregates so therefore when the frequency is decreases the phase angle also decreases and similar behavior is noted by increasing temperature the reason being greater influence of aggregates.

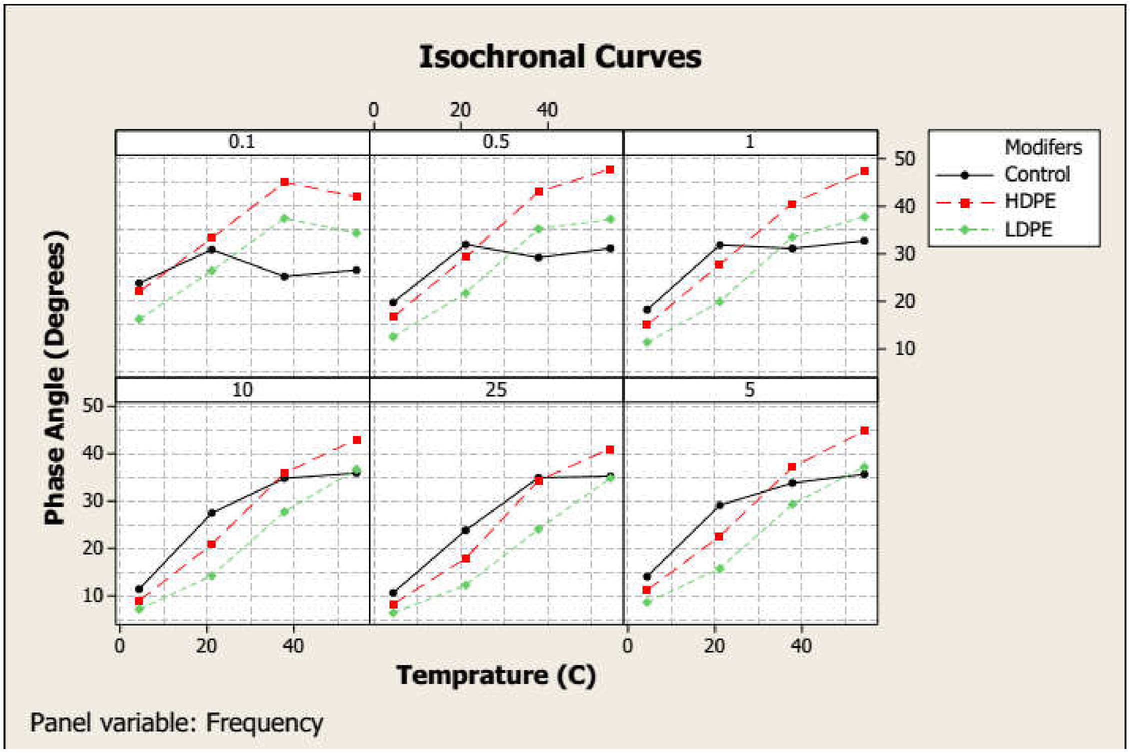


Figure 4-9: Effect of Temperature on Phase angle

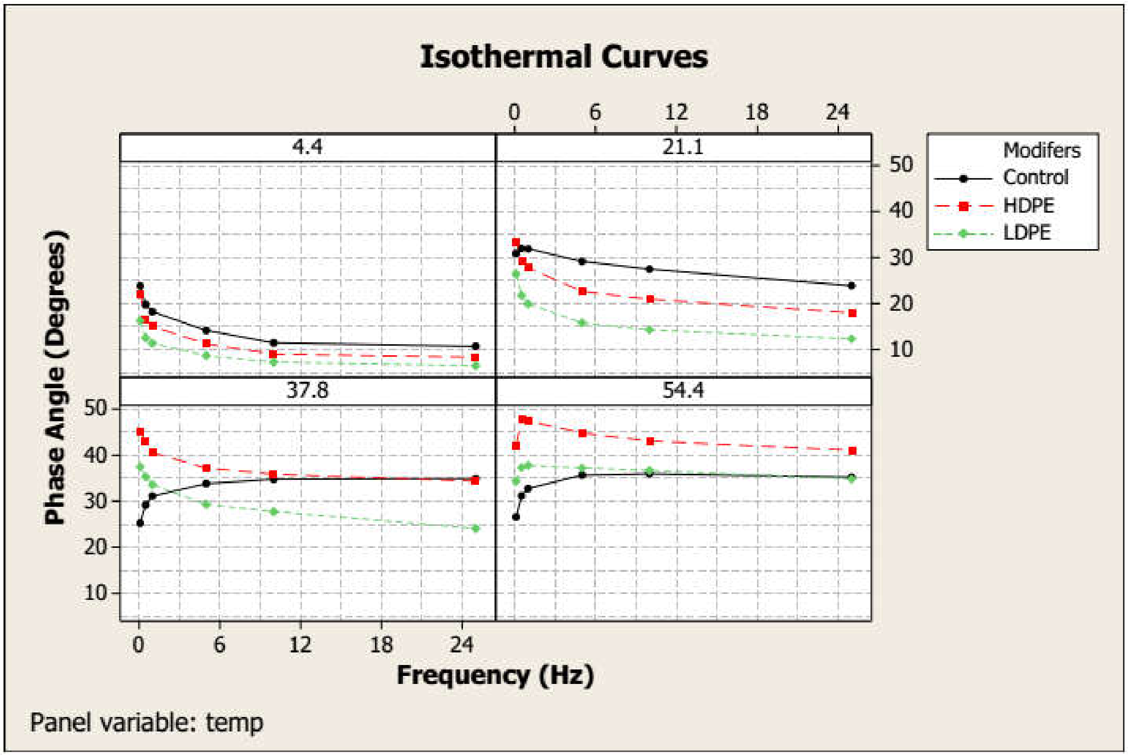


Figure 4-10: Effect of Frequency on Phase Angle

Scatter plot between dynamic modulus and phase angle is shown in Fig 4-11. There is decrease in dynamic modulus as the phase angle is high. LDPE has low phase angle as compared to HDPE and control mixes showed better performance.

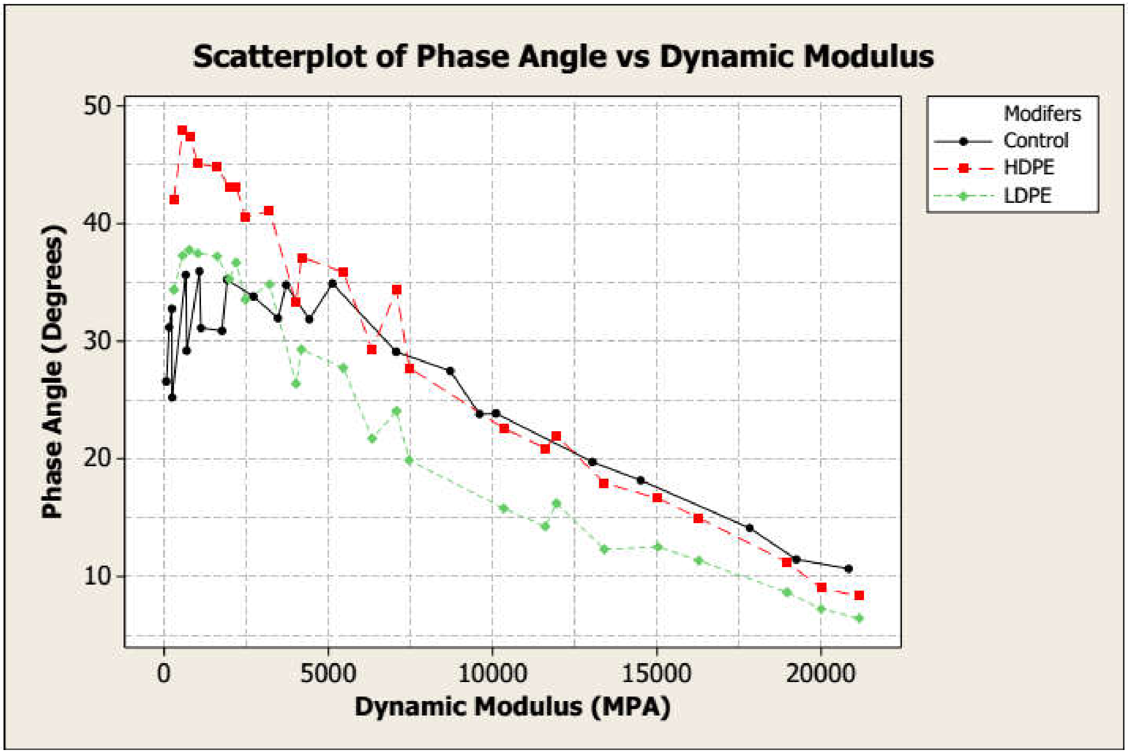


Figure 4-11: Scatter Plot of Phase Angle and Dynamic Modulus

4.6.1 Fatigue Parameter

In general, fatigue is process in which pavement weakens and develop cracks due to the repeated traffic loading. As pavement is exposed to recurring traffic loading and unloading, if loading go beyond certain limit, crack initiates at top and bond between binder and aggregate reduces. This results in propagation of cracks. E* and phase angle ( ) results can be combined to calculate fatigue parameter which is used for the approximation of the fatigue susceptibility of HMA mixes. Fatigue Parameter= |E*|xSin , where |E*| is dynamic modulus and is phase angle. It has an inverse relationship with resistance to fatigue cracking. Higher value of fatigue parameter represents lower resistance to fatigue cracking and vice versa (Ye et al. 2009). Figure 4.12 represents the fatigue parameters of HMA mixtures at 4.4 °C, 21.1 °C, 37.8 °C and 54.4 °C temperature and Fig 4-14 represent Fatigue parameter at six different frequencies i.e. 25, 10, 5, 1, 0.5, 0.1 Hz. Fatigue parameter at 21.1 C is more significant because at higher temperatures HMA pavements are more prone to rutting instead of fatigue which is shown by Figure 4-12. HMA layers are more susceptible to fatigue at medium temperatures.

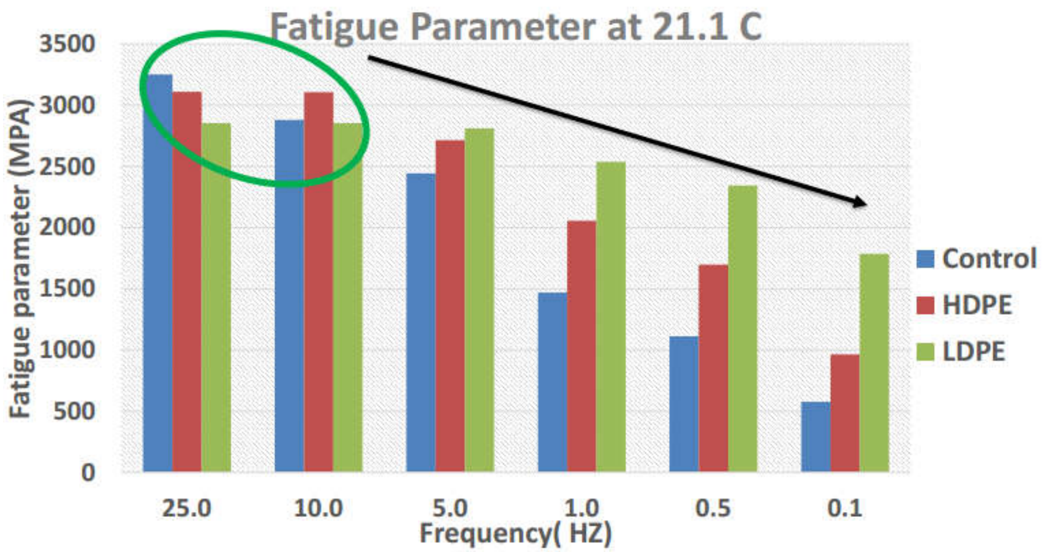


Figure 4-12: Fatigue Parameter at 21.1

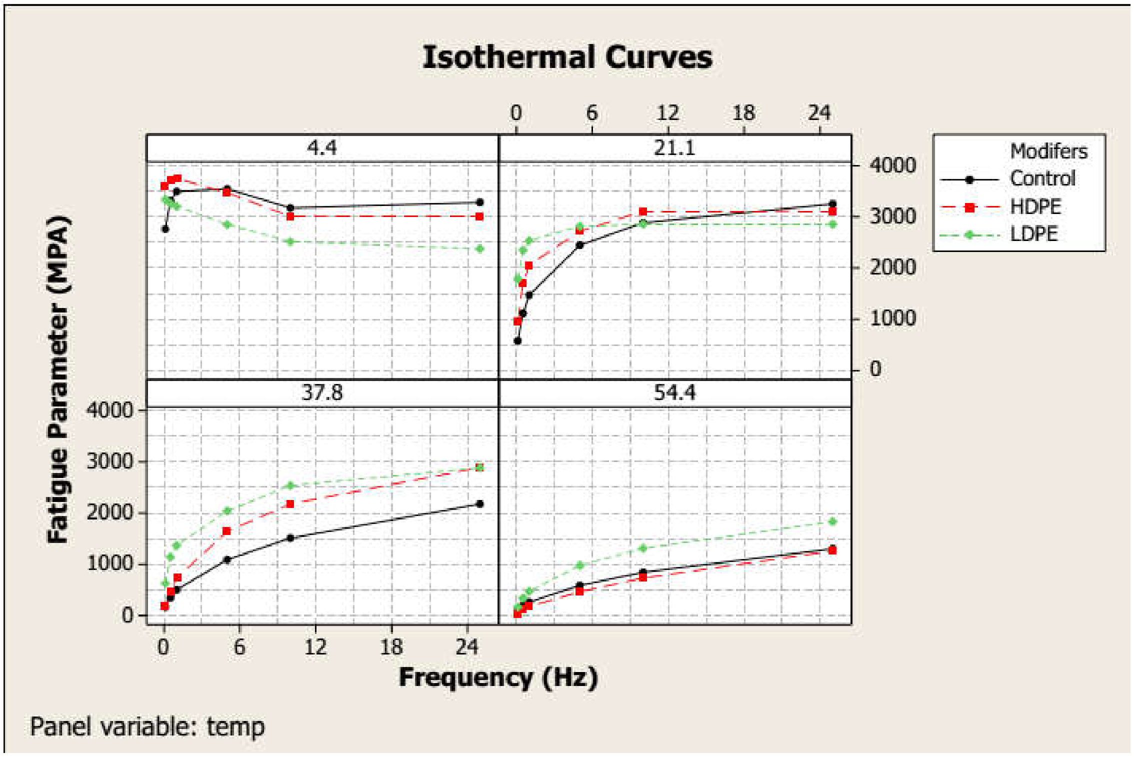


Figure 4-13: Effect of Frequency on Fatigue Parameter

From the Figure, LDPE mixtures show the smaller fatigue parameter at 4.4 °C than HDPE and control mixtures. This is because of lower phase angle of the HMA mixes fabricated using

LDPE. This lower values of phase angle leads to the improved flexibility/ductility resulting in higher dynamic modulus values. As fatigue parameter is product of dynamic modulus and sine of phase angle, lower dynamic modulus leads to lower fatigue parameter and lower phase angle leads to lower fatigue parameters, which shows higher fatigue resistance of the mixtures. And if we compare the fatigue parameter at 21.1 °C here LDPE mixes show high fatigue parameter at low frequency up to 10HZ but beyond that its value reduces as compared to HDPE. Figure 4-13 represents a comparison of Fatigue parameter at different frequency ranges. Here it is clear that at low loading frequency fatigue parameter is high for stiff mixes like LDPE. And individually fatigue parameter of LDPE is smaller than HDPE and control mixes at 25HZ and 10HZ at lower temperature.

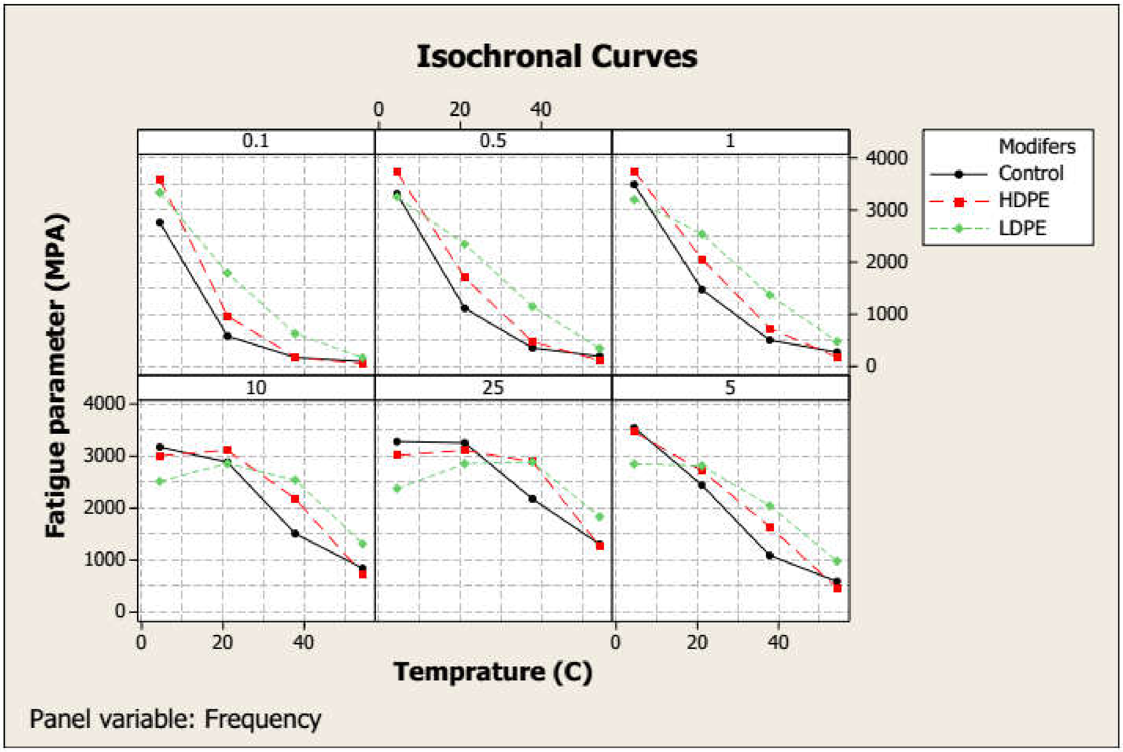


Figure 4-14: Effect of Temperature on Fatigue Parameter

4.7 FLOW NUMBER TEST RESULTS

Flow Number (Fn) tests have been performed on the laboratory prepared specimens to assess rutting vulnerability of the HMA mixes under study. The tests were conducted at 54.4ºC because of increased rutting susceptibility of HMA mixes at higher temperatures. Stress level was chosen as 300 KPa to be reliable with the previous studies. And the test termination was set to 50,000 micro strains or 10,000 cycles whichever occurring first. Flow number test is considered to best simulate the field conditions as it allows some rest period between the load applications as in actual pavements. Table 4-7 shows end results of the flow number test for wearing course

mixtures. Mixes prepared by HDPE showed more accumulated strains and reached the termination strain before completion of load cycles. But mixes prepared by LDPE and conventional binder showed less accumulated strains and load cycles were completed

first before reaching the termination strains.

Table 4-7: Flow Number Results

Sample	Flow Number	Strain @ Flow Point	Accumulated Microstrain
CN	1650	8191	45720.5
HDPE	2452	17139	50008
LDPE	4891	4955.5	6301.5

From the results, it is noticeable that the LDPE modified mixture undergo fewer strains as compared to the other two mixtures. This is due to the fact that LDPE modified mixture has high stiffness than other two mixtures. In Figure 4-14 shows that three different types of asphalt mixes are compared. Mixtures with LDPE modified mixtures are compared using accumulated axial strains and mixture with HDPE and control mixtures are compared according to their cycles to termination strain limit that is 50,000 micro strains.

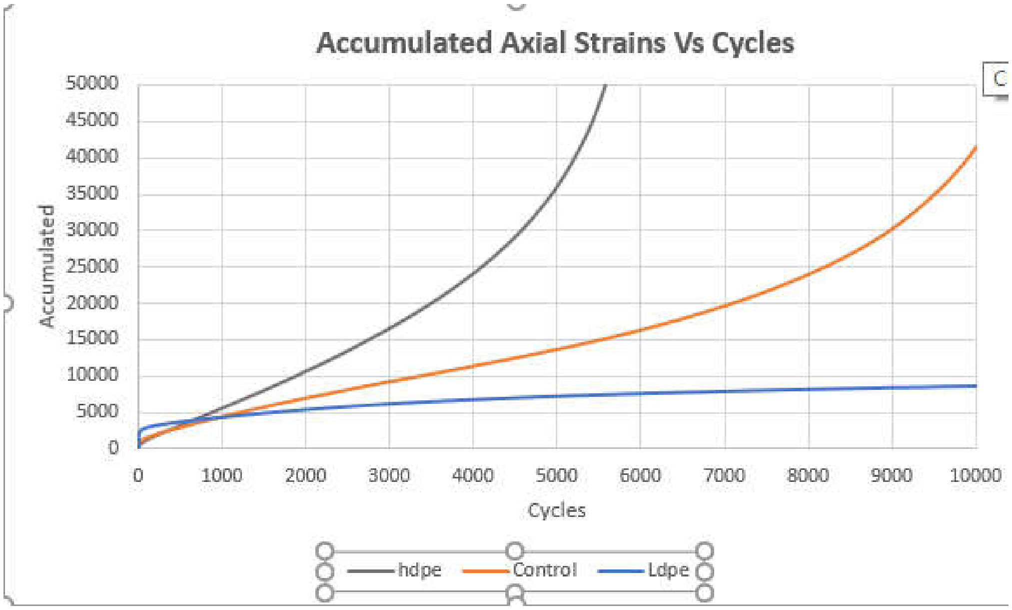


Figure 4-15: Accumulated Axial strain VS Cycles

Figure 4-15 shows the accumulated axial strains for different mixes are due to the applied 10,000 load cycles. It is clear that HDPE and Control mixture is more rut susceptible due to its greater accumulated axial strains as compared to LDPE.

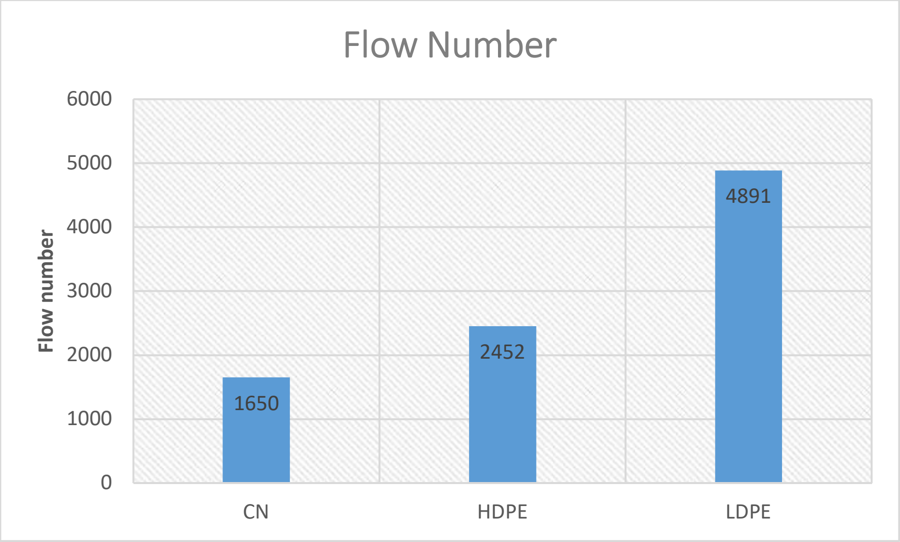


Figure 4-16: Flow Number

Figure 4-16 showed that LDPE has high flow number as compared to HDPE and Control mixtures. LDPE is 66.26% more rut resistant than control mixture and HDPE is 32.71% more rut resistant that control mixtures.

4.8 FLOW TIME RESULTS

The outcomes attained from flow time (Ft) tests illustrates that no mix produces the tertiary phase of deformation. When some of the mixes produces the tertiary floe deformation then data smoothing is required. So, in our case data smoothening is not required and outcomes attained from AMPT software is helpful for additional analysis. As discussed above, the tertiary stage of deformation is not detected in any mix, the investigation is constrained to the assessment of accumulated strain only.

Table 4-8: Flow Time Results

Sr no	Flow Time	Strain @ Flow Point	Accumulated Microstrain
CN	4768	8115	9259
HDPE	4796	9732	9959
LDPE	7951	9149	9193

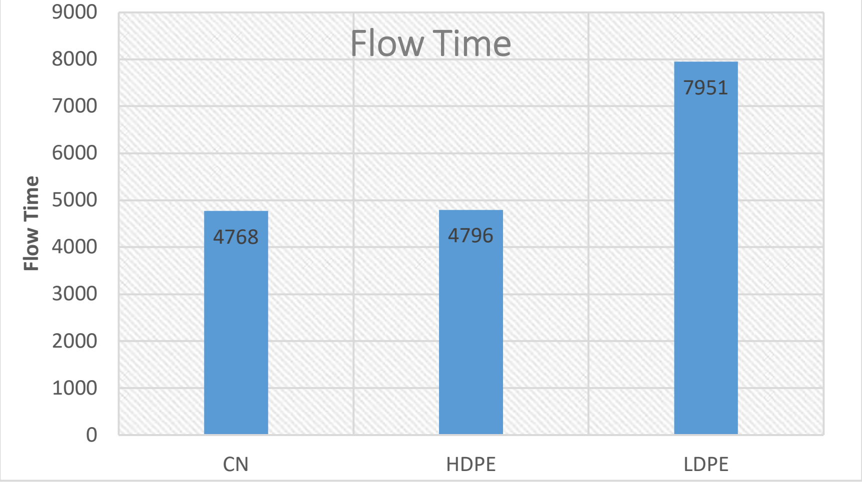


Figure 4-17: Flow Time Results

Figure 4-17 showed that LDPE has higher flow time as compared to HDPE and control mixtures. Flow time of LDPE showed 40% and HDPE is 0.58% higher than control mixtures.

LDPE has high stiffness and caco3 content than HDPE which results improvement in Flow time.

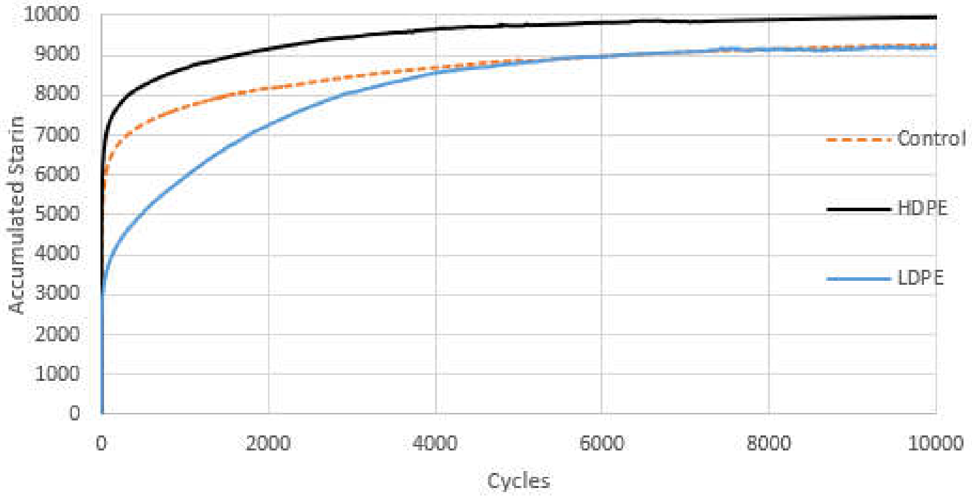


Figure 4-18: Time vs Accumulated Strains

Figure 4-18 showed that HDPE mixtures has higher accumulated micro strain than control and LDPE mixtures. HDPE has more content of sio2 which shows higher brittleness than LDPE mixtures.so it has higher strain.

4.9 INDIRECT TENSILE STRENGTH (ITS) TEST

ITS test was executed at 25 ^{0}C in the HMA specimens with polymer modified and conventional asphaltic mixtures. Tnesile stregth is measured in dry and wet condtion. The test has briefly discussed in chapter 2 whereas the conclusions are presented herein table 4-9 and results are plotted in figure 4-19.

Table 4-9: ITS Test Results

Sr	CN	LDPE	HDPE
Dry	8.553	7.8317	7.766
Wet	7.918	7.6435	7.226

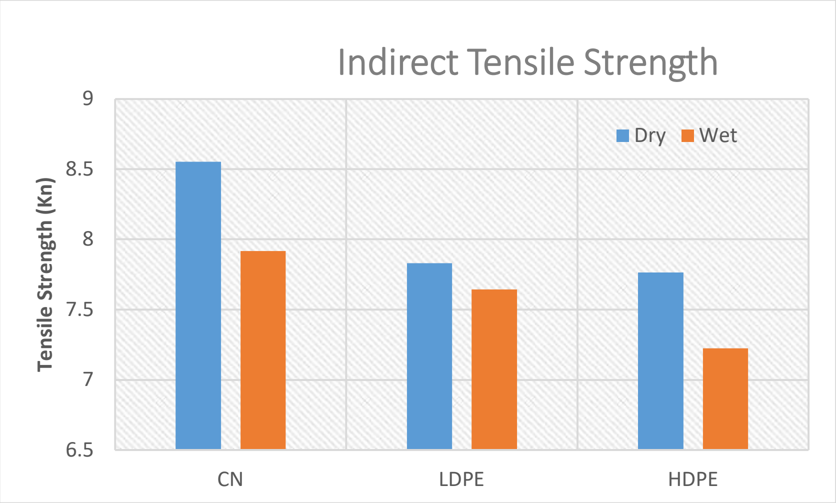


Figure 4-19: Indirect Tensile Strength (ITS) Results

4.10 RESILIENT MODULUS (MR) TEST RESULTS

Resilient modulus (MR) test was executed on both Polymer modified HMA and conventional HMA specimens at 25 0C temperatures and two loading durations (100 ms & 300 ms). The test has briefly discussed in chapter 2 whereas the results are presented herein table 4-10 and potted in figure 4-20.

Table 4-10: Resilient Modulus Test Results

Test	CN	LDPE	HDPE
MR	5397	8100	5490

Resilient modulus measures the stiffness of a mixture. LDPE has higher stiffness than HDPE and control mixtures. The presence of Caco3 is high in LDPE which improves the stiffness of HMA.LDPE is 33.37% and HDPE is 1.7% higher stiffness than control mixtures.

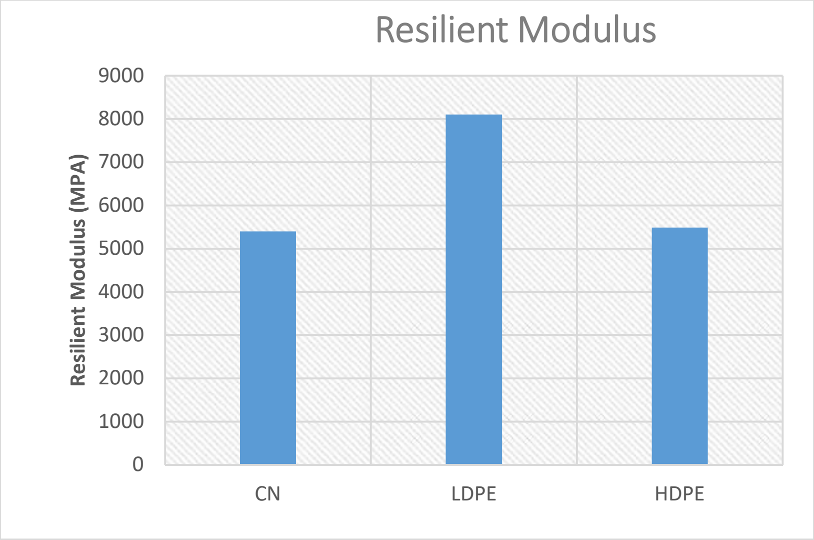


Figure 4-20: Resilient Modulus Results

4.11 MOSITURE SUSCEPTIIBLTY TEST

AASHTO T283 is the test procedure which helps us to describe the resistance of bituminous mixes against stripping. This is evaluated by the Indirect Tensile Strength (ITS). The results obtained for the ITS for dry and wet samples are presented. Three (03) replicate samples were tested for each gradation and the averaged value was taken individually. The Average Tensile strength in dry and wet condition and Tensile strength ratio is presented in Table 4-11 TSR was calculated too. Figure 4-21 shows the Average Tensile Strength Ratio.

Table 4-11: Tensile Strength Ratio

Sr	CN	LDPE	HDPE
Dry	8.553	7.8317	7.766
Wet	7.918	7.6435	7.226
TSR	0.9258	0.97597	0.93047

Figure 4-21 showed that TSR of all the mixture is more than 85% which shows better against performance in moisture damage. However, LDPE has improved 5.13% its performance against moisture damage.

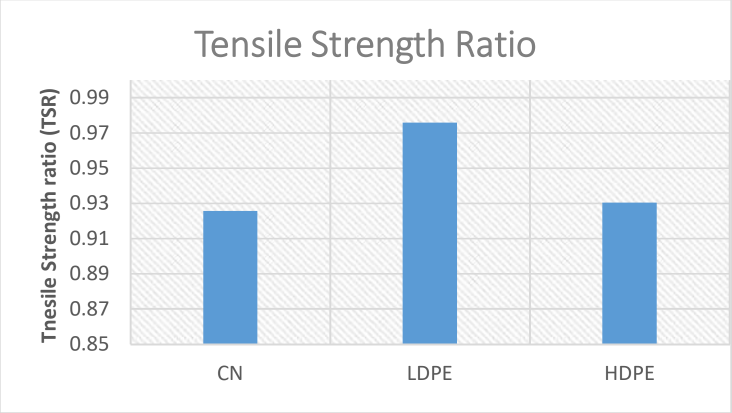


Figure 4-21: Tensile Strength Ratio

4.12 COST EFFECTIVENESS ANALYSIS

Cost effectiveness analysis was carried out by considering only one parameter which is the unit material cost (bitumen) which is being replaced by the unit cost of modifiers (LDPE and HDPE). Rest all the parameters (Environmental cost, Life cycle cost) were assumed constant. However, incorporating those parameters can enhance the cost effectiveness analysis of this research work. We used the optimum modifier dose to replace the bitumen and replace its cost with the cost of the modifiers. The calculation steps are described in tabular form as shown in Table 4-12

Table 4-12: Cost Effeteness Analysis

Discerption	Bitumen	LDPE	HDPE
Material Cost Per kg (PKR)	59	35	54
Optimum Modifier (%)		12	10
After Modification Cost Per kg (PKR)		(0.12*35) + (0.88*59)	(0.1*54) + (0.9*59)
		56.12	58.5
Saving Per kg (PKR)		2.88	0.5
% Saving		4.881	0.85

So, LDPE and HDPE both are cost effective by just considering their material cost. LDPE (shopping bags) is a major source of pollution in our environment because they are not being collected and reused frequently.so their use inn pavement industry should be encouraged to benefit both environment and pavament structure. The results in graphical form are shown in figure 4-22

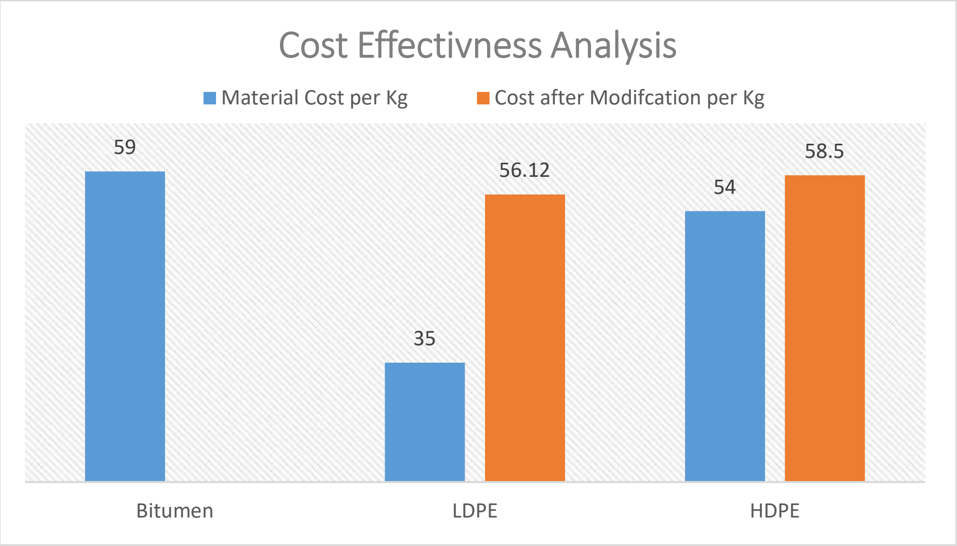


Figure 4-22: Cost Effeteness Analysis

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 GENERAL

This study was carried out to evaluate the performance of waste polymers modified HMA by Superpave mix design and to find out the improvement in the properties of HMA Results of simple performance test and universal testing system are used to evaluate the performance of asphalt mixtures. HDPE and LDPE were used as modifiers in the asphalt mixtures to characterize the performance. The optimum binder content and The optimum polymer content was determined. The factors selected for dynamic modulus tests were temperature, modifier, and loading Frequencies an. The key findings of dynamic modulus test, flow number test, flow time test, indirect tensile strength test, resilient modulus test and moisture susceptibility test results are concluded as follows:

5.2 CONCLUSION

5.2.1 Optimum Polymer Content

Optimum binder content is 4.36% and optimum waste polymer content of LDPE is 12% and HDPE is 10%

5.2.2 Flow and Stability

At OBC stability value is 32.48 KN and flow is 9.116 mm. There is significant improvement in stability and flow values by the use of waste polymers. At 10% HDPE stability value improves by 6% and flow reduced by 3.5%. At 12% LDPE stability value improves by 19.16% flow reduced by 9.36%. So LDPE has showed better performance than HDPE.

5.2.3 Stiffens Parameter

Generally, in all asphalt mixes Dynamic modulus decreases by the increase of temperature and decrease of loading frequency. Overall LDPE has showed higher dynamic stiffens followed by HDPE and control samples. Resilient modulus is also a measure of stiffness which confirms the trend followed by dynamic modulus test. Materials having high stiffness value indicate more rut resistant mixtures.

5.2.4 Moisture Susceptibility

Indirect Tensile strength is compared in dry and wet condition to compute tensile strength ratio(TSR) which indicate moisture vulnerability of asphalt mixes. TSR of LDPE, HDPE and control samples are 0.975 ,0.93 and 0.925 respectively. LDPE performs 4.61% better than HDPE and 5.13% better than control samples.

5.2.5 Rutting Potential

Flow number test results showed that LDPE mixtures perform better than HDPE and control mixtures. The analyses were made by comparing the accumulated axial strains at the termination cycle or cycle number at the termination accumulated axial strains. LDPE, HDPE and Control samples have flow number of 4891,2452 and1650 respectively. LDPE is 66.26% and HDPE is 32.71% more rut resistant than control samples.

Flow time results also followed the same trends the LDPE samples showed lesser accumulated strains as compared to HDPE and Control mixtures. Flow time of LDPE. HDPE and control sample is 7951,4796 and 4768 respectively. LDPE performs 40% and HDPE 0.6% better than control samples.

5.2.6 Fatigue Parameter

Fatigue parameter calculated using dynamic modulus and phase angle values showed that LDPE samples at high frequency of 25hz has less fatigue parameter than HDPE but higher than Control samples. Overall Control sample has less dynamic stiffness so it is more fatigue resistant due to their lower fatigue parameter value as compared to HDPE and LDPE samples. Mixtures with higher stiffness are good against rutting but more prone to fatigue cracking.

5.2.7 Cost Effectiveness Analysis

Cost effectiveness analysis was also carried out keeping all the factors constant and calculating the cost of bitumen replaced by polymers. which shows that LDPE is 4.63% and HDPE is 0.53% cost effective than Conventional mixtures because both the waste polymers have less cost as compared to the bitumen.

5.3 RECOMMENDATIONS

- The use of waste polymer modified HMA should be encouraged in our country as it has excellent performance and reduce the burden on environmental agencies to minimize its adverse effect on environment.
- Superpave mix design is being followed in developed countries to better analyzed pavement performance, so it should also be used in Pakistan.

5.3.1 Future Recommendations

- Further evaluation of waste polymer modified HMA should be carried out by changing parameters such gradation curve and binder grade.
- Further studies should be carried out to study the effect of modification on rheological properties of binder
- Correlation between dynamic and resilient modulus should be developed to analyzed their effect on HMA in detail.

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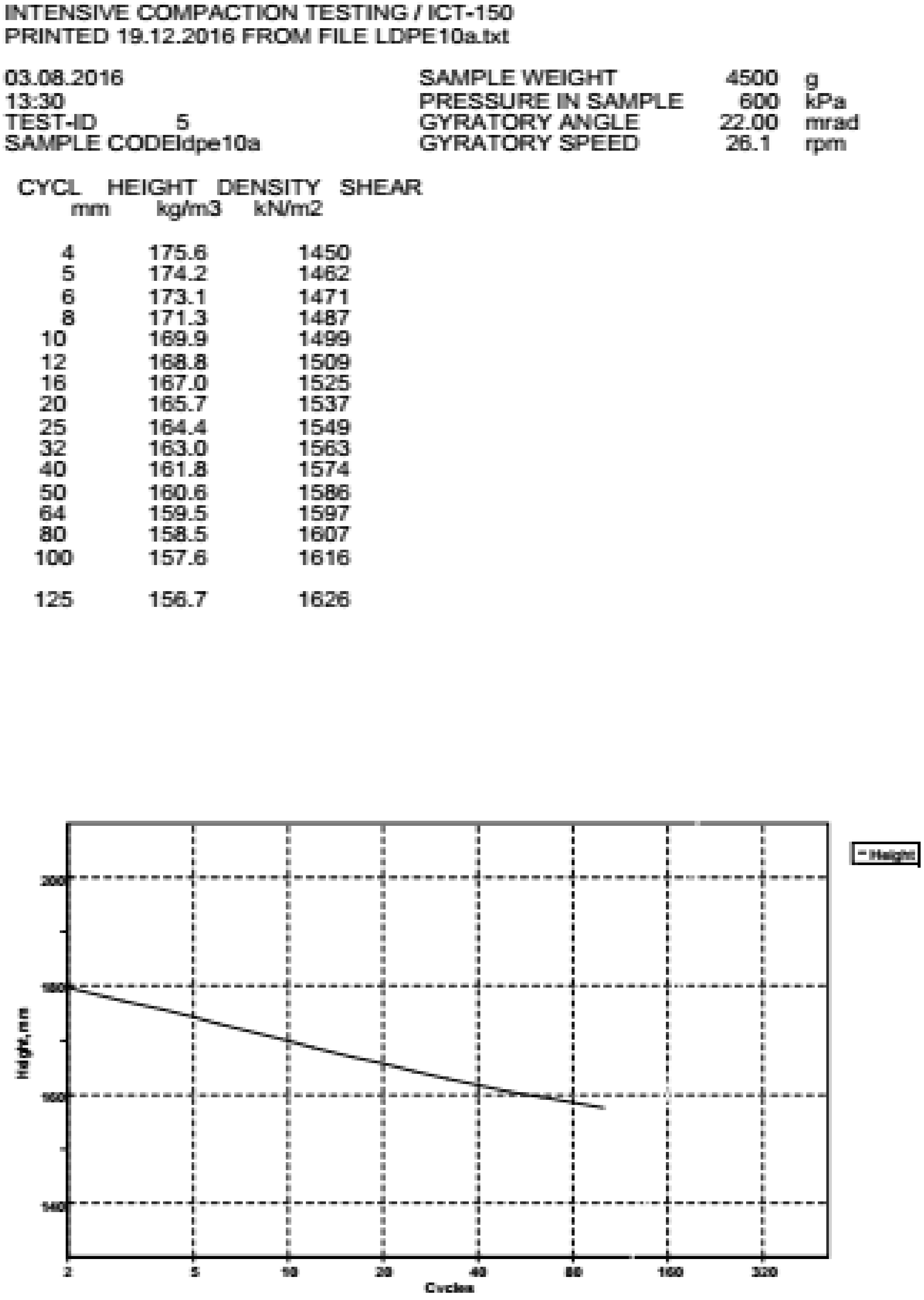
APPENDIXES

Figure A1: Software Output of Superpave Gyratory Compactor (SGC)

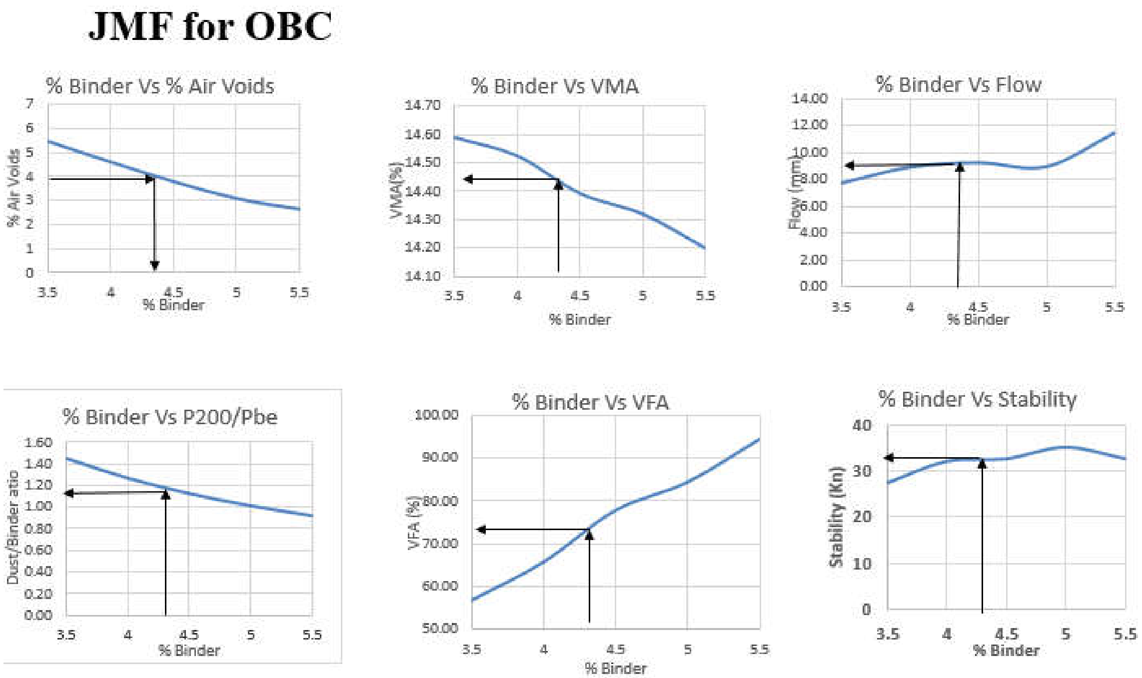


```
INTENSIVE COMPACTION TESTING / ICT-150
PRINTED 19.12.2016 FROM FILE LDPE10a.txt

03.08.2016                              SAMPLE WEIGHT          4500   g
13:30                                   PRESSURE IN SAMPLE      600   kPa
TEST-ID         5                       GYRATORY ANGLE        22.00   mrad
SAMPLE CODEldpe10a                      GYRATORY SPEED         26.1   rpm

 CYCL   HEIGHT  DENSITY  SHEAR
          mm      kg/m3    kN/m2

    4      175.6      1450
    5      174.2      1462
    6      173.1      1471
    8      171.3      1487
   10      169.9      1499
   12      168.8      1509
   16      167.0      1525
   20      165.7      1537
   25      164.4      1549
   32      163.0      1563
   40      161.8      1574
   50      160.6      1586
   64      159.5      1597
   80      158.5      1607
  100      157.6      1616

  125      156.7      1626
```

Table A1: Excel sheet to calculate % GMM(des) and % GMM(ini)

Type	Margalla Wearing		Weight%	Specific Weight	specific gravity
Aggregate	Limestone		96.5	2645.88	2.646
				2647.89	2.648
Bitumen Type & % of Asphalt	ARL 60/70		3.50	1017.86	1.018
TMD Gmm, (kg/m$^{3)}$				2500.00	2.500
Gmb (measured), (kg/m^3)				2309.00	2.309
	No of Gyrations	Height, mm	Gmb(Estimated),kg/m^3	Gmb(Corrected)	% Gmm
	4	178.2	1429	2074.471942	82.97887767
	5	176.9	1440	2089.716789	83.58867157
	6	175.9	1448	2101.59693	84.0638772
	8	174.2	1462	2122.1062	84.88424799
Nini = 9	9	173.6	1467	2129.440668	85.17762673
averaged	10	173	1472	2136.826012	85.47304046
	12	171.9	1481	2150.499709	86.01998837
	16	170.3	1495	2170.704052	86.82816207
	20	169	1507	2187.401775	87.49607101
	25	167.8	1518	2203.044696	88.12178784
	32	166.5	1529	2220.245646	88.80982583
	40	165.3	1541	2236.363581	89.45454325
	50	164.2	1551	2251.345311	90.05381242
	64	163	1562	2267.919632	90.71678528
	80	162	1572	2281.919136	91.27676543
	100	161	1582	2296.092547	91.84370186
Ndes	125	160.1	1591	2309	92.36

Figure A2: Job Mix Formula Results for OBC



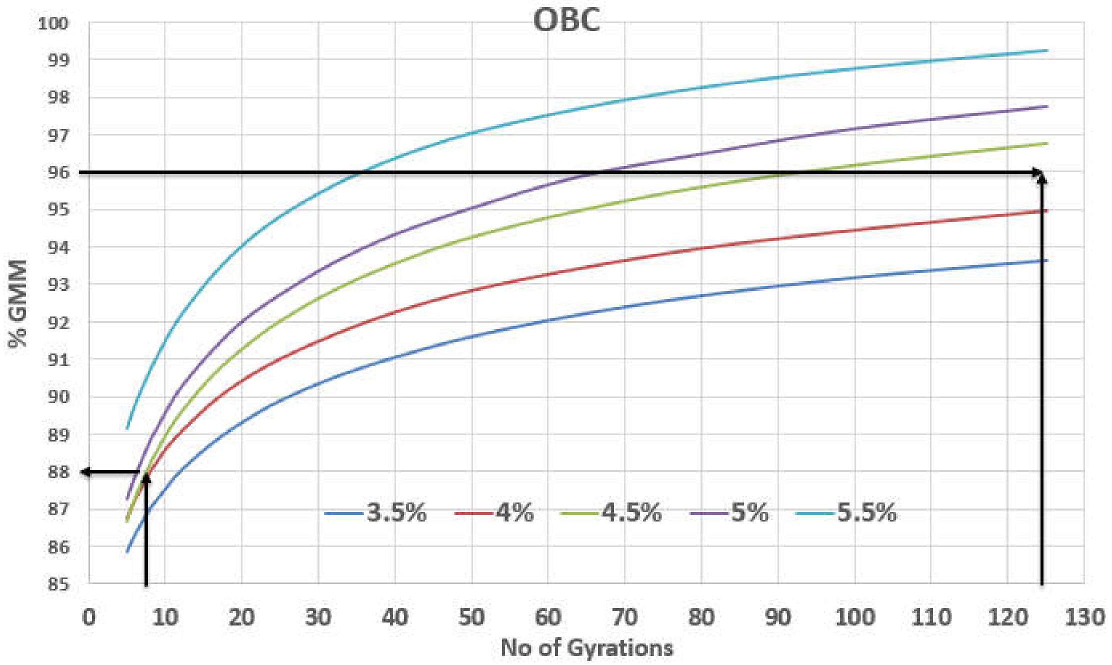


Table A2: Average Dynamic Modulus Test Results

Temperature	Frequency	Control	HDPE	LDPE
^{0}C	HZ	MPA	MPA	MPA
4.4	25	17737	20842	21165
	10	16007	19246	20005
	5	14540	17829	18965
	1	11191	14508	16276
	0.5	9811	13041	15034
	0.1	6844	9601	11948
21.1	25	8038	10100	13402
	10	6244	8708	11605
	5	5023	7065	10338
	1	2784	4416	7456
	0.5	2101	3469	6331
	0.1	1125	1754	4018
37.8	25	3805	5123	7071
	10	2647	3716	5453
	5	1958	2716	4179
	1	971	1123	2478
	0.5	715	687	1982
	0.1	392	242	1028
54.4	25	2256	1917	3210
	10	1434	1070	2194
	5	1012	658	1615
	1	490	232	768
	0.5	365	149	559
	0.1	208	66	293

Table A3: Average Phase Angle results

Temperature	Frequency	Control	HDPE	LDPE
^{0}C	HZ	MPA	MPA	MPA
4.4	25	10.7	8.3	6.4
	10	11.4	9.0	7.2
	5	14.1	11.2	8.6
	1	18.2	15.0	11.3
	0.5	19.7	16.6	12.5
	0.1	23.8	22.0	16.2
21.1	25	23.9	17.9	12.3
	10	27.5	20.9	14.2
	5	29.1	22.6	15.8
	1	31.9	27.8	19.9
	0.5	32.0	29.3	21.7
	0.1	30.9	33.3	26.4
37.8	25	34.9	34.3	24.1
	10	34.8	35.9	27.7
	5	33.8	37.2	29.3
	1	31.1	40.6	33.6
	0.5	29.2	43.1	35.3
	0.1	25.2	45.1	37.5
54.4	25	35.2	41.1	34.8
	10	36.0	43.1	36.7
	5	35.7	44.8	37.2
	1	32.8	47.4	37.8
	0.5	31.2	47.9	37.3
	0.1	26.6	42.1	34.4

Table A4: Fatigue Parameters

Temperature	Frequency	Control	HDPE	LDPE
^{0}C	HZ	MPA	MPA	MPA
4.4	25	3278	3014	2373
	10	3173	3009	2514
	5	3543	3465	2849
	1	3491	3749	3201
	0.5	3312	3733	3255
	0.1	2762	3595	3335
21.1	25	3251	3110	2854
	10	2880	3107	2855
	5	2443	2715	2812
	1	1470	2057	2537
	0.5	1112	1698	2344
	0.1	577	963	1786
37.8	25	2177	2889	2884
	10	1510	2178	2538
	5	1090	1641	2045
	1	502	731	1370
	0.5	349	469	1145
	0.1	167	171	625
54.4	25	1302	1260	1833
	10	842	731	1311
	5	590	464	977
	1	265	171	470
	0.5	189	111	339
	0.1	93	44	166

Figure A3: EDX Test results for LDPE

Spectrum processing :
No peaks omitted

Processing option : All elements analyzed (Normalised)
Number of iterations = 4

Standard :
C CaCO3 1-Jun-1999 12:00 AM
O SiO2 1-Jun-1999 12:00 AM

Element	Weight%	Atomic%
C K	92.36	94.15
O K	7.64	5.85
Totals	100.00	

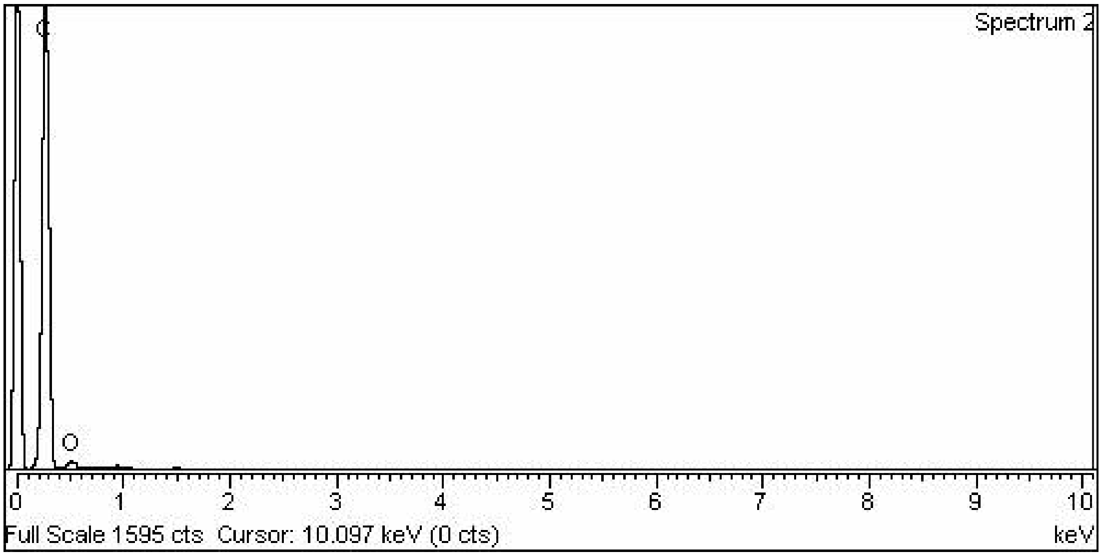
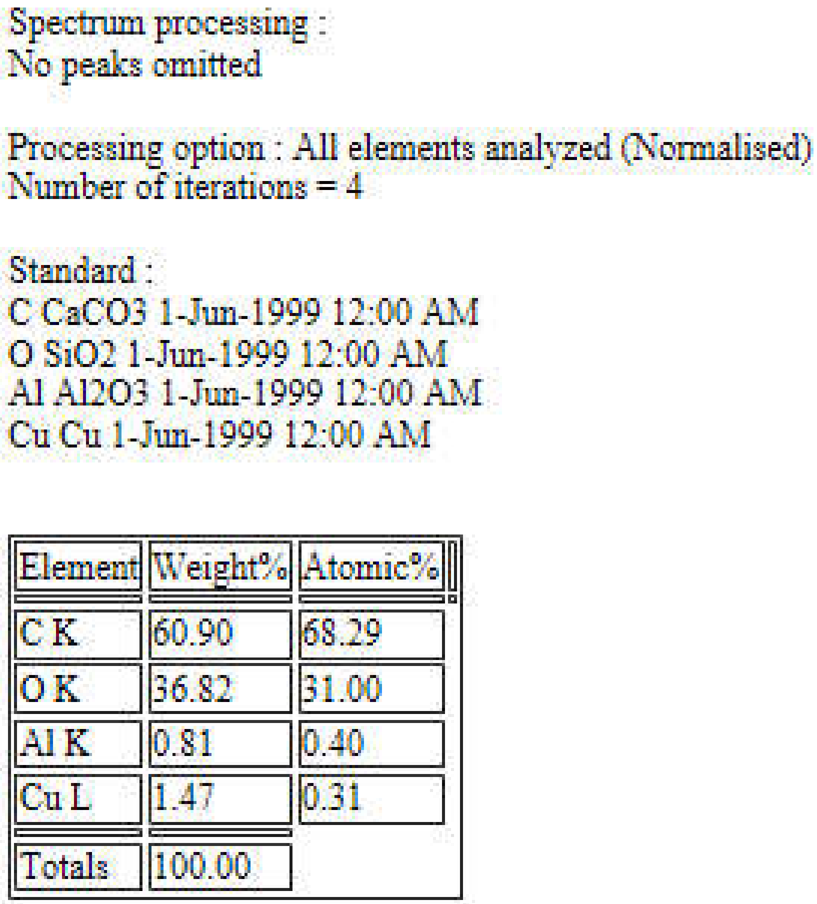


Figure A4 : EDX test Results for HDPE



Spectrum processing :
No peaks omitted

Processing option : All elements analyzed (Normalised)
Number of iterations = 4

Standard :
C CaCO3 1-Jun-1999 12:00 AM
O SiO2 1-Jun-1999 12:00 AM
Al Al2O3 1-Jun-1999 12:00 AM
Cu Cu 1-Jun-1999 12:00 AM

Element	Weight%	Atomic%
C K	60.90	68.29
O K	36.82	31.00
Al K	0.81	0.40
Cu L	1.47	0.31
Totals	100.00