PERFORMANCE OF STONE MASTIC ASPHALT INCORPORATING CELLULOSE FIBER

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ABSTRAK

Stone Mastik Asfalt (SMA) terkenal sebagai yang tinggi kandungan agregat kasar yang saling kunci untuk membentuk satu rangka batu yang menentang perubahan bentuk kekal. Walau bagaimanapun, ia menghadapi banyak masalah seperti aluran dan pelucutan kerana suhu yang tinggi dan berulang beban paksi (Zulhaidi et al., 2018). Ia juga mempunyai rayapan dalam beban dan suhu yang berbeza. Serat selulosa boleh meningkatkan kelikatan asfalt diubahsuai. Oleh itu, tujuan kajian ini adalah bertujuan untuk menggunakan gentian selulosa kekuatan tegangan yang tinggi untuk mengatasi masalah yang berkaitan dengan SMA. Penambahan serat selulosa ke dalam asfalt kawalan meningkat keupayaan pemulihan bitumen. Kertas kerja ini membentangkan hasil siasatan makmal ke atas Marshall Kestabilan, Resilient Modulus, Creep dinamik dan Cantabro Kehilangan lelasan Stone Mastic Asphalt (SMA) yang digabungkan dengan gentian selulosa pengikat diubah suai. Penembusan Gred 60-70 (PEN60-70) jenis pengikat telah bercampur dengan serat selulosa 0%, 0.2%, 0.3%, 0.4%, 0.5% dan 0.6% mengikut berat campuran. Campuran telah diuji untuk lelasan, Marshall Kestabilan, Resilient Modulus dan Dinamik Creep untuk menilai prestasi SMA diubah suai. Daripada keputusan, ia menunjukkan bahawa kewujudan serat selulosa mampu meningkatkan prestasi campuran asfalt, dan penambahan 0.2% serat selulosa menyumbang kepada nilai terendah lelasan, 0.4% serat selulosa menghasilkan ketumpatan yang tinggi, manakala 0.3% menghasilkan nilai tertinggi modulus berdaya tahan dan rayapan dinamik, 0.2% untuk kestabilan dan 0.4% untuk ketegangan. Untuk kajian masa depan, menguji untuk menganalisis kelakuan fizikal SMA dengan adanya serat selulosa untuk membuktikan keandalannya dalam pelbagai aplikasi dalam campuran asfalt.

ABSTRACT

Stone mastic asphalt (SMA) is well known as a high coarse aggregate content that interlocks to form a stone skeleton that resist permanent deformation. However, it facing a lot of problems such as rutting and stripping because of the high temperature and repeated axial load (Zulhaidi *et al.*, 2018). It also have creep in different loads and temperatures. The cellulose fibre can improve the viscosity of the unmodified asphalt. Thus, the aim of this study is intended to utilize the cellulose fiber high tensile strength to overcome the problem that is related to SMA. The addition of cellulose fiber into the control asphalt improved the recovery ability of asphalt binder. This paper presents the outcome of a laboratory investigation on Marshall Stability, Resilient Modulus, Dynamic Creep and Cantabro Loss Abrasion of Stone Mastic Asphalt (SMA) that incorporated with cellulose fiber modified binder. Penetration Grade 60-70 (PEN60-70) types of binder were mixed with cellulose fiber of 0%, 0.2%, 0.3%, 0.4%, 0.5% and 0.6% by weight of mixture. The mixtures were tested for Abrasion, Marshall Stability, Resilient Modulus and Dynamic Creep in order to evaluate the performance of modified SMA. From the results, it shows that the existence of cellulose fiber is capable of enhancing the performance of asphalt mixture, and the addition of 0.2% cellulose fiber contributes to lowest value of abrasion,0.4% cellulose fiber produce the high density, while 0.3% producing highest value of resilient modulus and dynamic creep, 0.2% for stability and 0.4% for stiffness. For future study, it is recommended to analyses the physical behaviour of SMA with the existence of cellulose fiber in order to prove its reliability in various applications in asphalt mixture.

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LIST OF SYMBOLS

g	Gram
mm	Milimeter
^{o}C	Celcius
%	Percent
min	Minute
s	Second
o	Degree

LIST OF ABBREVIATIONS

SMA	Stone Mastic Asphalt
HMA	Hot Mastic Asphalt
DGA	Dense Graded Asphalt
OFC	Optimum Fiber Content
JKR	Jabatan Kerja Raya
AIV	Aggregate Impact Value
ACV	Aggregate Crushing Value
RAP	Reclaimed Asphalt Pavement
PA	Porous Asphalt
PCC	Portland cement concrete
PE	Polyethylene
PP	Polypropylene
EVA	Ethylene–vinyl acetate
EBA	Ethylene–butyl acrylate
SBS	Styrene–butadiene–styrene
SIS	Styrene–isoprene–styrene
SEBS	Styrene–ethylene/butylene–styrene
BS	British Standard
VTM	Voids in Total Mix
VFA	Voids in Filled Asphalt
LA	Los Angeles
UTM	Universal Testing Machine

CHAPTER 1

INTRODUCTION

1.1 Background of study

Stone Mastic Asphalt (SMA), which has been utilized in Europe for around 40 years ago, was to begin with created to supply resistance to scraped spot by studded tires (Woodward et al., 2016). Within the 1970s, studded tires were prohibited in Germany, and the utilize of SMA blends declined since of the higher material and construction costs and there now not showed up to be a basic require for these mixtures (Brown, 1992).

Rutting of Hot Mix Asphalt (HMA) got to be a greater issue in Germany within the 1980s due to increased tire weight, wheel loads, and activity volume, and SMA mixtures started to be used again (Brown, 1992). Studded tires have kept on be utilized in Sweden, and SMA blends have proceeded to provide great execution beneath these extreme stacking conditions. Other European nations have used SMA mixtures with comparable victory to that observed in Germany and Sweden.

Stone Mastic Asphalt (SMA) which basically involves higher extent of coarse total, lower extent of moderate size total and higher extent of mineral filler contrasted with conventional blends is utilized (Panda, Suchismita and Giri, 2013). SMA has a high coarse total substance that join to construct a stone skeleton that withstand permanent deformation (M.A, 2018). The sweeping whole of coarse totals inside the mix frames a skeleton-type structure giving a superior stone-on-stone contact between the coarse total particles, which offers high protection from rutting.

The SMA mixtures give an unpleasant macro texture, forming small path between the coarse aggregate, which valuable for a productive surface drainage (Panda, Suchismita and Giri, 2013). The voids of the asphalt mastic are loaded up with a high consistency bituminous mastic of bitumen and aggregate, to which strands are included in arrange to supply agreeable stability of the bitumen and to anticipate drainage of the binder amid transport and placement. (Woodward et al., 2016).

Figure 1.1 Cross section of Stone Mastic Asphalt

Sources: M. A (2009).

The deformation resistant capacity of SMA originates from a coarse stone skeleton giving more stone-on-stone contact than with standard dense graded asphalt (DGA) blends. Improved folio sturdiness could be an aftereffect of higher bitumen substance, a thicker bitumen film and, lower of voids substance. This high bitumen content additionally makes progress of flexibility. Expansion in addition of cellulose or mineral fiber as addictive in asphalt mixture can prevents drainage of bitumen amid transport and placement. There are no exact design rules for SMA mixes (M.A, 2018). The essential features, which are the coarse total skeleton and mastic organization, and the following surface texture and mixture stability, are to a great extent chosen by the choice of total grading and the sort and extent of filler and cover.

1.2 Problem Statement

Since Malaysia is located in the tropical region with temperature, high humidity and copious rainfall throughout the year, it causes damage of road pavement (Zulhaidi *et al.*, 2018). Hence, the asphalt concrete facing a lot of problems such as rutting, moisture stripping and binder drain down. Subsequently, the mixture’s resistance to moisture damage and rutting specifically will effects the life span of the paving mixture.

Further more, the heavy traffic load also tends to influence the performance of asphalt mixture in terms of its resilient modulus and dynamic creep. A common method to overcome these problem is by modifying the asphalt binder properties by adding the cellulose fiber with Stone Mastic Asphalt (SMA). Since fiber tends to provide improvement of the properties for asphalt, this study aim to promote the cellulose fiber as asphalt binder modifier in order to enhance the properties of asphalt mixture.Rutting is a pavement distress instrument that can significantly affect the ride-ability, asphalt judgment and safety that's a common indication of 'aqua planning' on a road surface. The form of fatigue cracking in asphalt and embedment in spray seals, regularly too followed by surface failure.

Drain down is determined when mixture (fines and bitumen) that isolated itself from the sample and flows downward through the mixture (NAPA, 2017). The main problems with SMA mixtures are drainage and bleeding. In order to control these problems, storage and placement temperatures can not be lowered due to the difficulty in obtaining the required compaction. Stabilizing additives have therefore been added to improve the mastic, reduce the drainage of the mixture at high temperatures and achieve even higher binder content for increased durability (NAPA, 2017).

1.3 Objectives of study

The aim of this study to enhance the properties of SMA in terms of resilient modulus, dynamic creep, Marshall stability and Cantabro Loss with the existence of fibre content. Among the objectives are;

1. To evaluate the mechanical performance of cellulose fiber – stone mastic asphalt (SMA) interms of resilient modulus, dynamic creep, Marshall Stability and Cantbro Loss.

2. To determine the optimum fiber content (OFC) cellulose fiber modified asphalt binder SMA20.

1.4 Scope of research

The asphalt mixture use for this study was Stone Mastic Asphalt (SMA). The aggregates used in stone mastic asphalt is a combined aggregates consist of coarse aggregate and fine aggregate by the gradation in sieve analysis to determine sizes of aggregate in a mix. Coarse aggregate shall be screened crushed hard rock and free from vegetative, dust, clay, and other organic matter, and other deleterious substances (JKR, 2008). Meanwhile for fine aggregate, it shall be in the form of screened quarry fines. They shall be free from clay, loam, aggregations of material, vegetative and other organic matter, and other deleterious substances and non-plastic (JKR, 2008).

In order to select the suitable aggregate for use in pavement construction, aggregate tests had been carried out by Aggregates Impact Value (AIV) test, Aggregate Crushing Value (ACV) test, LA Abrasion test, Flakiness and Elongation test.

Furthermore, the bituminous binder use for this study was in penetration 60/70 grade. Bitumen 60/70 usually use for road construction and for the asphalt pavements with superior properties. In order to achieve the result performances in physical properties, it had been carried out by Softening Point of Bitumen test, Ductility test, and Penetration of bituminous materials test. Cellulose fibre was act as the modified asphalt binder to enhance the properties of asphalt mixture of SMA. During the preparation of the sample, different amounts of cellulose fibers were mixed with asphalt binder. To prevent unnecessary drainage during transport and bleeding during their service life, the cellulose fibers are used as a stabilizing agent and incorporated into the SMA mix. The organic stabilizing agent dosage rates are 0.3% by weight of the total mix (JKR, 2008).

The lab compacted specimen undergoes the design mixture for the asphalt binder. Each combination of laboratory design mix aggregate gradation and bitumen content are subjected to the Marshall test procedure and volumetric analysis by preparation of laboratory specimens for the standard stability and flow test, determination of the bulk specific gravity, determination of the stability and flow values, and the analysis of the specific gravity and air voids parameters to determine the percentage air voids in the compacted aggregate, the percentage air voids in the compacted aggregate filled with bitumen and the percentage air voids in the compacted mix (JKR, 2008).

Meanwhile, for mechanical properties carried out by the resilient modulus and dynamic creep via laboratory test subjecting with the cylindrical specimen to loads using Universal Testing Machine (UTM). UTM test helps to measure asphaltic specimens elasticity, assess material quality and provide input for pavement design (Ahmad Kamil Arshad, 2017).

1.5 Significant of Research

Figure 1.2 Cellulose Fiber

To improve the pavement performance of asphalt mixture, cellulose fibers were selected as the modified binder. Adding cellulose fibers to the asphalt control enhanced asphalt binder recovery capability. Natural cellulose fibers are fibers that are still recognized as being from part of the original plant because they are processed to clean the fibers for use only as much as is necessary.

Cellulose fibers are generally used and generally protected in SMA in Europe and the USA. In order to avoid asphalt drainage, the fibers had been improved on the mix's service properties by performing the micromesh in the asphalt mix to increase the asphalt stability and durability (Yadykina *et al.*, 2015).

As the various stabilizing additives on drain down have been considered, the fibers perform better work on avoiding drain down than polymers. They also tended to mix volumetric and stone-to-stone contact achievement in SMA. Cellulose fibers have been shown in previous research that as the percentage of fine aggregate in a mixture decreases, the voids in coarse aggregate (VCA) decreases. (Kumar, Chandra and Bose, 2007).

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In this project, the cellulose fibers was used as an additive with aggregate gradation in Stone Mastic Asphalt (SMA). This chapter consist of discussion on previous researches that have been done, which was related to this study. Moreover, there were a lot of researchers have been done about the problems undergoes on stone mastic asphalt and was selected the cellulose fiber as additives in asphalt mixture. It consists of many elements such as the background of pavement and the types of pavement consist of flexible pavement and rigid pavement.

Moreover, for the third element of this chapter deals with the types of flexible paving such as dense graded asphalt, porous asphalt, polymer-modified asphalt and stone mastic asphalt, and the fourth element deals with the overview of stone mastic asphalt (SMA) as the main background of this asphalt mixture project.

Furthermore, the fifth element is about the fibers as selected addictive of asphalt binder such as steel fibers, kenaf fibers, bamboo fibers and cellulose fibers. The further element is about the overview of cellulose fibers as the selected asphalt modified binder for this study.

For the last element, it described the resilient module and dynamic creep of stone mastic asphalt by means of a laboratory test to determine the mechanical properties of the asphalt mixture subject to loads using the Universal Testing Machine (UTM) with the cylindrical specimen.

2.2 Background of pavement

Pavement is defined as the structure constructed above the existing subgrade soil whose primary function is to distribute the applied vehicle loads to the sub-grade., typically placed in distinct layers and including compacted/natural subgrade, subbase, base, and the riding surface. The figure below shows the basic components of a typical pavement system (White *et al.*, 2017).

Figure 2.1 Basic components of a typical pavement system

Sources: O'Flaherty (2007).

The pavement structure should provide an acceptable riding quality surface, adequate skid resistance, favorable characteristic reflecting light, and low noise pollution. The ultimate goal is to ensure that the transmitted stresses due to wheel load are adequately reduced so that they do not exceed the subgrade's bearing capacity.

The following requirements should satisfy an ideal pavement. First of all, sufficient thickness to distribute the stresses of the wheel load to a safe value on the sub-grade soil, which is structurally strong to withstand all types of stresses. And adequate coefficient of friction to prevent skidding of vehicles, smooth surface to provide comfort to road users even at high speeds, and produce least noise from moving vehicles, Dust proof surface so that traffic safety is not impaired by reducing visibility, impervious surface so that sub-grade soil is well protected, and long design life with low maintenance costs (O'Flaherty, 2007).

2.3 Types of pavement

The pavements can be classified into two, flexible pavements and rigid pavements based on structural performance. In flexible pavements, wheel loads are transferred through the granular structure through grain-to-grain contact of the aggregate. The flexible pavement acts as a flexible sheet such as bituminous road with less flexural strength. On the contrary, wheel loads are transferred to sub-grade soil in rigid pavements by the pavement's flexural strength and the pavement acts as a rigid plate such as cement concrete roads. Besides these, there are also composite pavements available. An ideal pavement with most desirable characteristics is a thin layer of flexible pavement over rigid pavement. However, due to high cost and complex analysis, such pavements are rarely used in new construction.

2.3.1 Flexible Pavement

Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure (Sharma, 2016). The wheel load acting on the pavement will be distributed to a wider area, and with depth the stress will decrease. Using this characteristic of stress distribution, flexible pavements usually have a lot of layers. The design of flexible pavement therefore uses the layered system concept. Based on this, flexible pavement can be built in a number of layers and in addition to wear and tear, the top layer must be of the highest quality to maintain maximum compressive stress. The lower layers will experience less stress and use of material of low quality (Sharma, 2016).

Flexible pavements are built with bituminous materials. These can be either in the form of surface treatments (such as bituminous surface treatments usually found on low-volume roads) or asphalt concrete surface treatments (usually used on high-volume roads such as national highways). Flexible pavement layers reflect the deformation of the lower layers on the surface layer as there is no undulation in the sub-grade, then it is transferred to the surface layer. In the case of flexible pavement, the design is based on overall flexible pavement performance, and the stresses produced should be kept well below the permissible stresses of each pavement layer.

2.3.2 Rigid pavement

Rigid pavements have sufficient flexural power to convey wheel load pressure to a broader region below. A typical cross-section of stiff asphalt. Stiff pavements are placed in comparison to flexible pavement either directly on the prepared subgrade or on a single layer of granular or stabilized material. Since between the wood and the sub-grade there is only one layer of covering, this layer can be called as a foundation or sub-base course (Muench, 2006).

The movement of the slab distributes the load in stiff concrete, and on a viscous medium the concrete functions as an elastic layer. Portland cement concrete (PCC) builds stiff pavements and should be evaluated by plate hypothesis instead of layer theory, assuming an elastic plate sitting on a viscous base. Plate theory is a simpler layer theory variant that considers the concrete sheet as a medium dense plate that is air before charging and stays air after charging. Bending the sheet owing to load and heat variations in the frame, leading in tensile and flexural strain.

The pavements had several faults. Traditionally, cracking fatigue has been regarded the significant or only criterion for the stiff construction of the asphalt. The number of load repetitions allowed to cause fatigue cracking depends on the stress ratio between flexural tensile stress and concrete rupture module. Pumping is identified as an important criterion of failure of late. Pumping is the ejection of soil slurry under heavy wheel loads through the joints and cracks of cement concrete pavement. In rigid pavements, other major types of distress include failure, spalling, and deterioration.

2.4 Types of flexible pavement

The following types of road construction have been used in flexible pavement such as Dense Graded Asphalt, Porous Asphalt, Polymer Modified Asphalt and Stone Mastic Asphalt.

A dense combination is a well-graded HMA for particular use. A dense-grade blend is comparatively impermeable when correctly designed and built. The nominal peak overall size of dense-graded mixes is usually referred to and can be further categorized as either fine-graded or coarse-graded. Fine graded mixes contain more particles of good and sand size than mixes of coarse grade. For all pavement layers and all traffic situations, the thick graded asphalt pavement is appropriate. Works well for the requirements of structure, friction, leveling and patching (Interactive, 2010).

There are great parking spaces and highways in Porous Asphalt pavements. They tend not to have issues with the creation of rust and potholes. The surface has a good wear. For centuries, porous asphalt has been demonstrated to last, even in severe environments and even in regions with many freezing periods (MacDonald, 2006). There are even more benefits to using the underlying stone bed to manage storm water for adjacent impermeable areas such as roofs.

Polymer Modified Asphalt is the inclusion by mechanical blending or chemical response of plastics in bitumen (Lu, 1997). The different materials studied included plastomers (e.g. polyethylene (PE), polypropylene (PP), ethylene-vinyl acetate (EVA), ethylene-butyl acrylate (EBA)) and thermoplastic elastomers (e.g., styrene-butadiene-styrene (SBS), styrene-isoprene-styrene (SIS), and styrene-ethylene / butylene-styrene (SEBS)). These plastics have been noted to have enhanced bitumen characteristics, such as greater rigidity at elevated temperatures, greater corrosion strength at low temperatures, stronger resistance to humidity or longer fatigue lives (S. Tayfur, 2007).

Stone Mastic Asphalt is made from a mineral mixture which is gap-graded, it also has bitumen as a binder and uses stabilizing additives. The primary features of the structure of concrete mastic asphalt are discovered in the definitions of elevated chipping quantities, elevated chip size quantities, elevated binder content and stabilizing additives. The additives that stabilize function act as binder suppliers (Schäfer, 2005).

2.4.1 Dense Graded Asphalt

A dense mixture for particular use is a well-graded HMA combination. When correctly designed and built, a thick blend is comparatively impermeable. Their peak nominal aggregate size generally refers to dense mixes. (Interactive, 2010). For all pavement layers and for all traffic circumstances, the aim of thick graded asphalt is appropriate. For structural, friction, levelling and patching requirements, dense graded pavement also operates well. It includes products such as well graded aggregate, asphalt binder with or without modifiers, and also with Reclaimed Asphalt Pavement (RAP). Water permeability depends on air voids content (< 3 % impermeable, 6-7% usually no interconnected pores) (Pellinen, 2016).

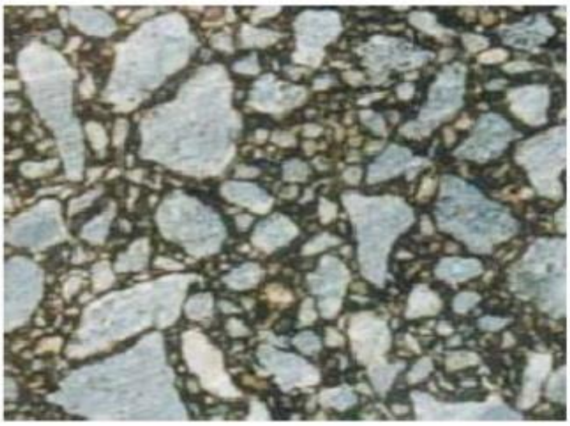


Figure 2.2 Dense Graded Asphalt composition

Sources: M.A (2018).

2.4.2 Porous Asphalt

Porous asphalt (PA) is a kind of versatile blend designed to solve the issue of rain rain and precipitation, particularly in the vehicle parking area and low-traffic alternatives. PA allows early precipitation and local drainage to pass through the open graded gravel surface of the road from which the fluid percolates into the natural soil below. (Yalcinkaya, 2009).

The other concept of PA, is an asphalt building blend that minimizes the fine bits in the gravel blend. It will enable precipitation to drain through the floor instead of thick or conventional asphalt concrete allowing water to flow off the groundonly (C.A. Michele, 2004). Figure 2.3 shows the porous asphalt.

Figure 2.3 Porous asphalt

Sources: Arshad, Masri and Ahmad (2017).

Michele C Adams It stated that the first porous pavement developed at the Franklin Institute in Philadelphia, PA in the 1970s consists of standard bituminous asphalt in which the aggregate fines (particles below 600 μm or No. 30 sieve) were screened and reduced, allowing water to pass through the asphalt. A bed of evenly graded and clean-washed blend with a blank fortieth house is put beneath the sidewalk. Storm water drains through the asphalt, controls the stone bed, and slowly infiltrates into the underlying mantle of the soil.

Porous asphalt (PA) is known for its benefits in enhancing concrete skid strength during rain, decreasing splashing impacts and reduced driving noise. (Liu, 2009). These requirements have been developed because of the elevated porosity of the concrete coating, which then allows the ground run-off to be highly drained. PA usually has a complete proportion of vacuum between 20% and 25%, which is comparatively large relative to standard warm blended asphalt. The elevated concentration of voids in PA is allowed by the use of open-graded aggregate type. PA is the conventional bituminous asphalt in which good aggregate use has been lowered, allowing fluid to pass through the pavement (Arshad, Masri and Ahmad, 2017). PA is one various answer to the matter of storm water drainage from parking and alternative low traffic density areas.

Furthermore, porous asphalt is an highly environmentally friendly and low-speed surface alternative. Porous pavement is ideal for parking lots, driveways, or pedestrian walkway and in fact, their characteristics may prove detrimental. For paving these surfaces, porous pavements offer clear advantages over their rigid cousins. Moreover, porous pavement was effective for surface water management. The effective management of surface water is important for any place or community that's subject to heavy rainfall. This water precipitation can be seriously harmful in areas where storm drain infrastructure is limited or insufficient (Elswick, 2017). When roads are paved with asphalt or concrete, the water cannot soak into the bottom, leading to vital runoff. Frank Elswick, 2017 stated, not only does this storm water runoff cause dangerous flooding and the erosion of fragile ecosystems, it also often carries with it pollutants found in traditionally paved surfaces. Porous pavement significantly reduces these risks by allowing the water to permeate the earth.

However, PA also includes a couple of drawbacks. One of them is that PA is extremely sensitive to raveling. Raveling is the loss of aggregates on the highest paving surface layer. Raveling has a negative impact on PA's capacity to reduce noise and requires early maintenance (Q.T. Liu, 2011). Raveling took place because of the mixture of vehicle charging and serious weather circumstances. This disadvantage adds to PA's main harm and affects PA's service lives (Y. Zhang, 2012). Raveling is caused mainly by increasing the stiffness of the relaxation capacity and the formation of micro cracks in the binder due to aging (Q.T. Liu, 2011). The surrounding contaminant will also lead to preventive disadvantage of PA (C. Syrrakou, 2010). PA tends to have shorter service life (sometimes only half) compared to dense graded asphalt mix (X. Qiu, 2009) .

2.4.3 Polymer Modified Asphalt

Polymer alteration of asphalt binders has become progressively the standard in the design of pavements with optimum performance, especially in the United States, Canada, Europe and Australia. Particular materials used include rubber, SBR, SBS and Elvaloy (Yildirim, 2007). The term polymer simply relates to very big molecules produced to generate lengthy chains by chemical reaction of many tiny molecules. A particular polymer's physical properties are determined by the sequence and chemical structure of the monomers it is created from, its molecular weight and mass distribution. When adding plastics to asphalt, the properties of the altered asphalt concrete rely on the features of fabric, fabric features, blending circumstances and air compatibility of polymer (Becker, 2011).

Rutting (continuous distortion), heat exhaustion, tension exhaustion, and aging are the most prevalent asphalt concrete flaws. Asphalt alteration by adding polymer was used to solve these disadvantages. The potential constraints with altered bitumen are price increases, potential compatibility and stabilization issues, and some difficulties may occur in storing bitumen, blending conditions, and the duration of moment the product is kept at high temperatures before laying.

The main advantage of using these high-performance asphalts is enhanced rutting strength, with secondary advantages being less heat (cold-temperature) cracking and generally enhanced durability of the blend. In addition, some altered binders provide enhanced resistance to removing the humidity harm) (Vaitkus and Paliukaite, 2013).

2.4.4 Stone Mastic Asphalt

Stone Mastic Asphalt (SMA), an asphalt paving combination, was born in Germany in the 1970s to provide peak rutting strength induced by stubborn tires on European highways (AAPA, 2002). Strabag, a big German construction firm, resulted the growth of SMA. After stopping the use of studded tires, it was discovered that SMA supplied long-lasting surfaces that were so resistant to heavy vehicle traffic routing and demonstrated to be highly efficient in wear fighting. In 1984, a domestic standard was laid in Germany in appreciation of its outstanding results. SMA has since distributed across Europe, North America and Asia-Pacific (Vaitkus et al., 2017).

Modern pavement technology needs to be developed to speed up the significant increase of pavement quality on roads, airport runways and urban roads. Figure 2.4 exhibits the stability in a SMA-mix is obtained through the internal friction in the self - supporting stone skeleton.

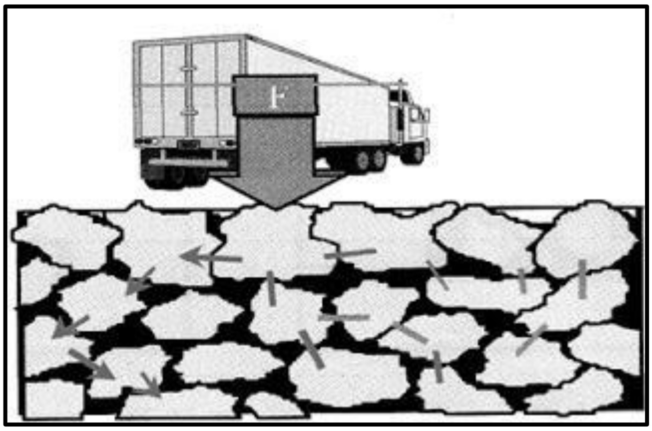


Figure 2.4 The stability in a SMA-m ix is obtained through the internal friction in the self -supporting stone skeleton

Sources: Greer (2006).

2.4.4.1 Advantages of Stone Mastic Asphalt

SMA meets the following demands and benefits for asphalt pavement. First of all, SMA provides excellent strength at elevated temperatures, for example the SMA blend has a self-supporting rock structure with a broken high-quality coarse material that increases inner friction and sharpening strength and therefore exceptionally high strength. SMA also has excellent stability at low temperatures because the SMA blend has a high-binding mastic cement with superior characteristics over thick graded asphalt to resist heat breaks.

In addition, SMA also offers elevated wear strength due to the small water void of the SMA blend, which makes the blend practically impermeable and offers a suitable resistance to aging, humidity sensitivity and durability. In addition, owing to the macro-texture of the concrete ground, SMA has excellent skid strength and hence the use of coarse aggregates with a large glossy rock value, SMA asphalt achieves a much higher skid strength rate.

In addition, SMA enables to reduce water spray owing to its greater density of texture, less water spray, and less glare reflected from the paved ground at night and greater visibility of street signs. Last but not least, SMA displayed the reduced traffic noise due to the fabric characteristics that the SMA highway surfaces typically provide reduced sound rates.

2.4.4.2 Disadvantages of Stone Mastic Asphalt

The disadvantages of SMA such as higher costs connected with greater binder and filler content can be found and elevated fiber additive filler content in SMA can result in lower efficiency. Appropriate crop changes can solve this. In addition, there may be delays in traffic entry as the SMA blend should be cooled to 40 C to avoid the binder layer from being flushed, and original skid strength may be small until the dense binder layer is taken off the ground by traffic (Prof. B. E. Gite, 2019).

2.4.4.3 Applications of Stone Mastic Asphalt

The construction of Stone Mastic Asphalt can be seen in the road construction as a extremely traffic-friendly roadway with outstanding deformation resistance and stable ground content. SMA is generally used for road building, covering and carrying surfaces. (Vaitkus et al., 2017).

2.4.4.4 Design Consideration of Stone Mastic Asphalt

Stone Mastic Asphalt is characterised by its high stone content which forms a gap-graded skeleton - like stone structure (Hasamudin and Soom, 2002). The voids of the structural matrix are full of a high consistency bituminous mastic. The high stone content of a minimum of seventieth ensures stone-on-stone contact once compaction. SMA mixes have a bitumen content of minimum 6.5%. The bitumen within the gap-graded mix is stabilized throughout the blending process, intermediate storage, transportation, emergence and compaction through the addition of cellulose fibre stabilizing additive. Figure 2. Show that SMA forms a gap-graded skeleton - like stone structure.

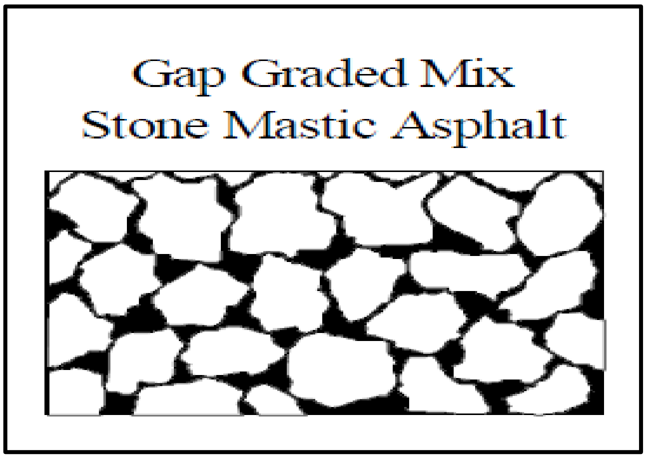


Figure 2.5 SMA forms a gap-graded skeleton - like stone structure

Sources: Hasamudin and Soom (2002).

2.5 Overview of Stone Mastic Asphalt (SMA)

Stone mastic asphalt or SMA can be define as a gap graded wearing course mix with a high proportion of coarse aggregate content which interlocks to form a stone on stone skeleton to resist permanent (Hainin, Reshi and Niroumand, 2012a). The coarse aggregate creates a high-strength skeleton with excellent inner friction and aggregate interlocking to withstand load-induced deformation and offers a lasting texture that is resistant to cracking and routing. (JKR, 2008).

Usually SMA blended with altered binder such as cellulose fiber to boost continuous strain strength and boost asphalt surface life span. Modified binder can also decrease the risk of implementation and harm, particularly in very small parts, and can decrease the need for an inhibitor of irrigation.

The Optimum Binder Content (OBC) for SMA20 are determined based on Marshall Mix Design Method. This process was carried out in accordance with the Malaysian Public Works Department JKR/SPJ/2008, 2008.

2.6 Fibers

2.6.1 Definitions of Fibers

It is not a new concept when the Warren Brothers Company of Boston, MA, obtained patents for their use of asbestos fibers in sheet asphalts through the use of fibers in asphalt mixtures. The aim of the fibers was to prevent asphalt bleeding during hot weather and to stabilize the mixture in essence. (Putman, 2011).

The primary function of polymers is to prevent being drained from the blend as a stabilizing additive by the comparatively elevated percentages of oil binder. The materials used as stabilizers primarily include cellulose and mineral fibers like slag wool and stone wool. Indicated that the polymers more efficiently lowered the flow of the asphalt binder from the blend while demonstrating excellent rutting strength, low heat distortion and low humidity harm as the mixtures integrated with polymers. Putman and Amirkhanian also assessed the efficiency of various fabric kinds in SMA mixtures. The findings of this research stated that cellulose fibers, waste nylon fabric fibers, waste tyre fibers, and polyester fibers were assessed and conducted likewise in terms of indirect tensile intensity, humidity sensitivity, and rutting resistance.

2.6.2 Types of Fibers

The following types of fibers have been used in asphalt modified mixture by the addition of steel fiber, knaft fiber, bamboo fiber and cellulose fiber.

2.6.2.1 Steel Fibers

Steel fiber is a metal strengthening and used to reinforce concrete and modify asphalt blend is described as discrete lengths of steel fibers with an aspect proportion (length-to-diameter ratio), brief, cross-sectional and tiny enough to disperse randomly in an unhardened asphalt mix. It acts as an asphalt blend additive and can provide rutting strength at greater temperatures and cracking strength at reduced temperatures.

Figure 2.6 Steel Fibers

2.6.2.2 Kenaf Fibers

It has been discovered that Kenaf (Hibiscus cannabinus, L. Malvacea family) is the primary cause of composite fibre and other agricultural applications (Raman Bharath, Vijaya Ramnath and Manoharan, 2015). Using kenaf fiber can assist create employment in both rural and urban regions; as well as helping to decrease waste, thereby adding to healthier work. Kenaf fiber enables to improve asphalt combination for asphalt alteration by demonstrating excellent rutting strength, low heat cracking, and low humidity harm.

Figure 2.7 Kenaf Fibers

2.6.2.3 Bamboo Fibers

Bamboo belongs to a grass family known as Bambusoideae (Jan E.G. van Dam, 2018). Bamboo has a strong capacity to photosynthesize, low density, elevated development speed and low price, which provides it speciality over other crop fibers.

Bamboo culm is built by a hollow tube, while it is the location where roots develop and is made up of many vascular cords that were intended to provide the bamboo factory with force (Jan E.G. van Dam, 2018).

Bamboo also can be used as the additive for asphalt mixture. Bamboo is a new fiber additive that give the good impact on enhancing the Stone Mastic Asphalt (SMA) that provide them from rutting and binder drain down.

Figure 2.8 Bamboo Fibers

2.6.2.4 Cellulose Fiber

Cellulose fiber are fiber made with ether or ester of cellulose which can be obtained from the bark wood or leaves of plants or from a plant-based natural besides cellulose. These fibers are compound of hemi cellulose and different percentages of these components are responsible for different mechanical properties observed.

Cellulose fiber act as the stabilizing capacity (ability to prevent binder draindown) (Putman, 2011). The most comprehensive research of fiber-modified asphalt was conducted included the cellulose fibers. This fiber had the ability to stabilize the mastics, ultimately resulting in richer mixes having high resistance to moisture, aging, fatigue, and cracking. Besides that, this fibers can lowered the temperature susceptibility of the asphalt mixture providing resistance to rutting at higher temperatures and resistance to cracking at lower temperatures.

However, cellulose fiber also have its disadvantages encountered during the addition of natural fibers, including cellulose fiber into a stone mastic asphalt, Is the lack of good interfacial adherence between the two components resulting in poor final product properties. (Raman Bharath, Vijaya Ramnath and Manoharan, 2015). Other disadvantages are the cellulose tens fibers, the chance of degradation, the biological fungal and mildew assault readily, which tend to show poor effect power, moisture sensitivity, limits uv resistance and a restriction of elevated heat handling.

The cellulose fiber also improves the efficiency of stone mastic pavement by adding cellulose fiber and increases the strength and weight of the device and decreases the flow and water voids (Panda, Suchismita and Giri, 2013). The addition of cellulose fibers to the mixing process as a stabilizing agent to strengthen the SMA by increasing the binder content, increase the aggregated films thickness, increase the stable mixing and enhanced fibers ' interlocks and the strength of aggregates. (Hainin, Reshi and Niroumand, 2012b).

CHAPTER 3

METHODOLOGY

3.1 Introduction

3.2 Experimental process

For this study, the material properties test was conduct on its asphalt binder and aggregate. For the aggregate test, it undergoes sieve analysis, La Abrasion test, Aggregate Impact Value (AIV) test, Aggregate Crushing Value (ACV) test, Flakiness and Elongation test. The asphalt binder was being tested on its physical properties test by penetration test, softening point test and ductility test.

The preparation of SMA20 by using Marshall mix design with addition of cellulose fiber of 0%, 0.2%, 0.3%, 0.4%, 0.5% and 0.6%. From the Marshall mix design test, we can identified the Marshall Stability, stiffness, density, flow value, the percentage air voids in mineral aggregate (VMA) and the percentage in mineral aggregate (VMA) and the percentage of air voids in the compacted mix (VIM) obtained.

The asphalt mixture performance test for the modified cellulose fiber had been obtained by the Marshall Stability, Resilient Modulus, Dynamic Creep and Cantabro Loss test.

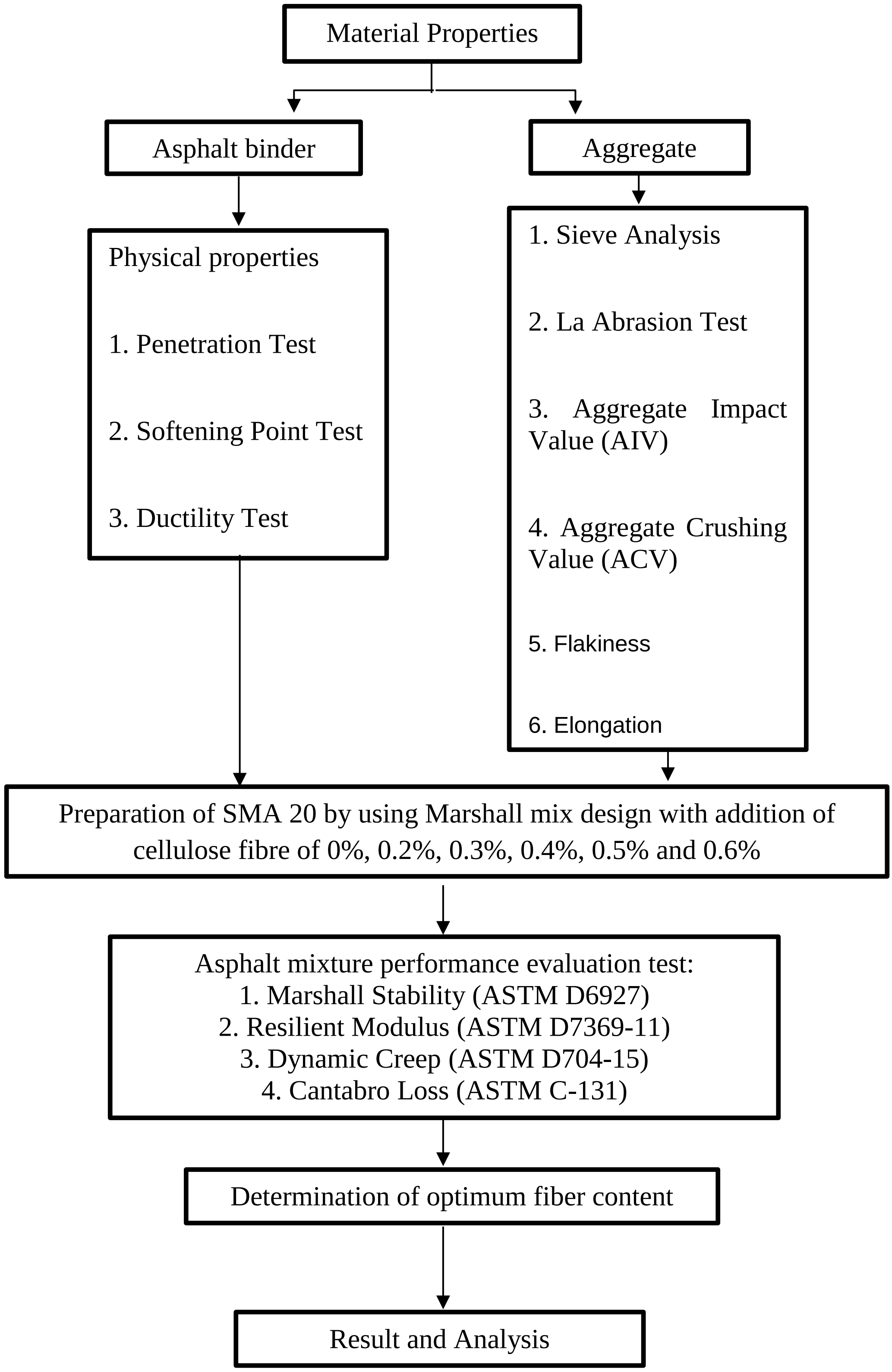


Figure 3.1 Preparation of Material Properties and Preparation of Performance Test

3.3 Material Properties

The binder used in this study was PEN 60/70 grade. Asphalt binder was blended with 0.2%, 0.3%, 0.4 %, 0.5% and 0.6% of cellulose fiber. The amount of each addictive was selected based on the weight of mixture. To conduct the mixing process, the aggregate and the asphalt bitumen was mixed with addition of the cellulose fibers before compact the asphalt binder after reaching 180^{o} C. The asphalt mixture use for this study was Stone Mastic Asphalt (SMA). Cellulose fiber was act as the modified asphalt binder to enhance the properties of modified asphalt binder of SMA.

3.3.1 Aggregate Testing

3.3.1.1 Sieve Analysis Test

The objective of this test was to produced a “Grading Curve” for fine and coarse aggregate according to ASTM.

The apparatus used for this test was sieve set size 19.0, 12.5, 9.5, 4.75, 2.36, 0.6, 0.3 and 0.075. This test used was balance accurate to 0.1 g and mechanical sieve shaker.

Figure 3.2 A set of sieve

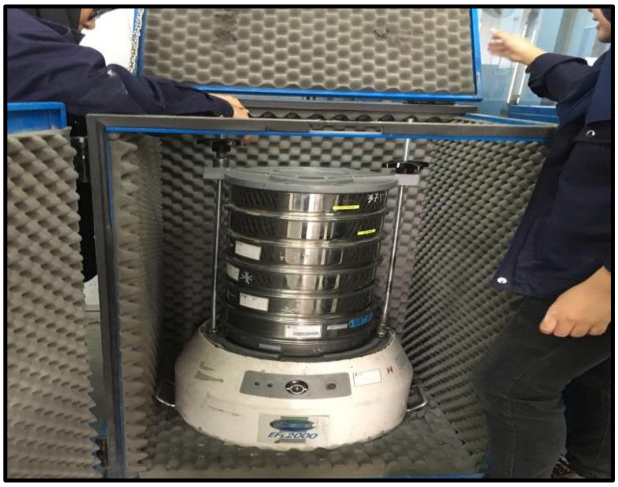


Figure 3.3 Mechanical sieve shaker

Figure 3.4 Weighing balance

The combined aggregate sample was prepared for sieve for the procedure. After that, a stack of sieves whereby sieves were placed above those with smaller opening sizes with larger opening sizes. After that, the sample was placed on the sieve inside with the largest opening on top of the remaining sieves. And then shake the material individually through each sieve into the collection tray in the mechanical sieve shaker for about 10 to 15 minutes. The weight of the aggregates retained in each sieve was measured and graded on the weighing balance table. Repeat the same procedures for the other aggregate sample.

3.3.1.2 Los Angeles Abrasion Test

The objective of this test is to obtain the Los Angeles number in the form of percentage wear of aggregates which reflects their resistance to degradation using the Los Angeles testing machine.

The apparatus consists of the Los Angeles abrasion machine, metal tray and a set of test sieves consist of 19mm, 12.5 mm, 9.5 mm, 1.7 mm and pan, sieve shaker and weighing balance are used for the aggregate. Los Angeles machine consists of a hollow steel cylinder, closed at both ends, having an inside diameter 70 cm and an inside length of 50 cm, mounted on stub shafts about which it rotates on a horizontal axis. Abrasive charges, consist of steel spheres averaging approximately 46.8 mm in diameter and each weighing between 390 and 445 g in weight are used.

Figure 3.5 Los Angeles abrasion machine and metal tray

Figure 3.6 Steel spheres

For the sample preparations, approximately 5000g of aggregates of 2500 $\pm$ 10g of 19mm to 12.5mm, and 2500 $\pm$ 10g of 12.5mm to 9.5mm sizes are used in this test. This is for aggregates graded mainly between 20mm and 10mm size.

After that, the test sample is washed and oven dry at 105 to 110^{o}C to a substantially constant weight as stated in the sample preparation in (a) above. The test sample was placed in L.A Abrasion Machine with the load (11 steel spheres). According to designation ASTM C131, the machine was switched on to rotate for about 500 revolutions at 30 to 33 rpm. The sample is discharged onto a tray after being rotated and sealed on No. 12 sieve (1.70 mm). Finally, at a temperature of 105 to 110^{o}C, the sample retained on the sieve was washed and dried. The weight of the sample is taken after the sample is cooled down.

3.3.1.3 Aggregate Impact Value (AIV)

The objective of the experiment is to determine the aggregate impact value of road stone in the laboratory.

The apparatus used for this test such as the impact testing machine, measure, tamping rod, sieves, balance and laboratory oven. The impact testing machine consists of a cylindrical hammer sliding freely between two vertical supports (called guides). Its fall is automatically adjusted to a height of 38 cm. There is a brass plate over which an open cylindrical steel cup of internal diameter is placed and fixed to the brass plate. Besides that, measure is being used, it is a cylinder of internal diameter for measuring aggregate. Furthermore, the tamping rod of 1 cm diameter and 23 cm long rounded at one end and pointed at the other end and as set of sieves. The balance consists of 5000g capacity and laboratory oven also were used for this test.

The aggregate was first sieved and the portion passing 14 mm was obtained and retained on a sieve of 10 mm. In addition, at a constant temperature, the aggregate was washed and dried and the sample cooled. The weight of the aggregate was M1 (first weight). In this cylindrical measure, the aggregate was filled in 3 layers, tapping 25 times each layer with the tamping rod. The surface tamping rod used the straight edge as level. The measure weighed the aggregate. This aggregate weight is used on the same material for the duplicate test. The aggregate was transferred in 3 layers from the cylindrical measure to the cup and compacted each layer by tamping with the tamping rod in 25 strokes. The hammer was released after that to free fall on the aggregate. A total of 15 blows is subjected to the test sample. The aggregate sample through the 2.36 mm sieve was removed from the cup and sieve. The fraction that passed the sieve was weighed 2.36 mm as M3 (weight loss).

Figure 3.7	Impact testing machine

Figure 3.8	A cylinder of internal for measuring aggregate

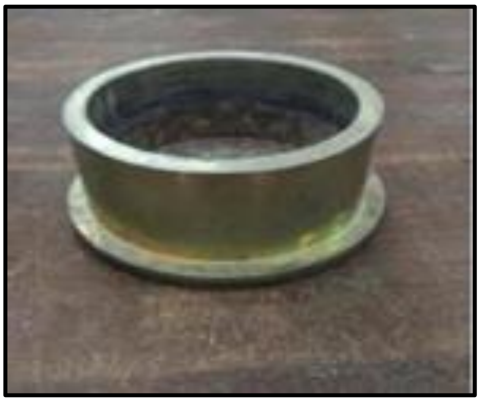


Figure 3.9	Steel cup

Figure 3.10 Tamping rod

Figure 3.11 Sieve

Figure 3.12 Laboratory oven

3.3.1.4 Aggregate Crushing Value (ACV)

The main mechanical characteristics needed in highway rocks are satisfactory resistance to cracking under the roller during building and appropriate resistance to traffic ground abrasion. The aggregates used in road construction should be strong enough to withstand traffic wheel loads crushing. If the aggregates are weak, it is likely to adversely affect the stability of the pavement structure. The aggregate crushing value provides a relative measure of crushing resistance under a compressive load gradually applied. To achieve high pavement quality, it should be preferred to use aggregates with low aggregate crushing value.

The objective of this experiment is to determine the mechanical strength of the aggregate. The apparatus used for this test was the open ended steel cylinder of nominal 150 mm internal diameter with plunger and base plate, a tamping road with a 16 mm diameter and 600 mm long, balance of 3 kg minimum capacity, sieves of sizes 14.0 mm, 10.0 mm and 2.36 mm beaker, compression testing machine which capable of applying force of 400kN and cylindrical metal measures for measuring the sample.

Figure 3.13 Open ended steel cylinder with plunger and base plate, tamping rods and the cylindrical metal measures.

Figure 3.14 Compression testing machine

Figure 3.15 Balance of 3 kg minimum capacity

For the procedure , the aggregate is sieved and the 14 mm portion that was retained on the sieve of 10 mm is obtained. At a constant temperature of 105 ^{o}C to 110 ^{o}C, the aggregate is washed and dried. The sample is then refrigerated and stored in a storage box. The aggregates are filled into the cylinder in thirds and 25 blows of tamping rod released at 50 mm above the aggregate surface are subjected to each third. Thus, aggregate weight is recorded as M1 before it is crushed. The plunger is then inserted on the aggregate surface. It records the maximum force used to produce the required penetration. Then the force is released and the crushed material is removed into a tray in the cylinder. On the 2.36 mm sieve, the entire specimen in the tray is sieved. The passing and retaining fraction on the sieve is weighted and recorded respectively as M3 plus M2. If the total mass (M3 and M2) differs by more than 10 g from the initial mass M1, discard the result and test additional specimen.

3.3.1.5 Flakiness and Elongation

Flaky particles are those whose smallest size is 0.6 times smaller than the mean size and for elongated particles whose one size is 1.8 times larger than the other two. The maximum permissible limit for flaky particles in the mix is 30 %. If this value exceeds then the mix is considered inappropriate for the purpose of construction. Flaky and elongated particles reduce concrete mixes ' workability due to a high surface-to-volume ratio. The degree of one-size particle packaging depends on its shape. The flakiness index is calculated as a percentage of the sample's total weight by assessing the weight of flaky particles. By expressing the weight of elongated particles as a percentage of the total sample weight, the elongated index is calculated.

The apparatus for this test consists of thickness/flakiness index gauge, length/elongation index gauge, aggregate sample to be tested, coarse sieve shaker, and weighing balance.

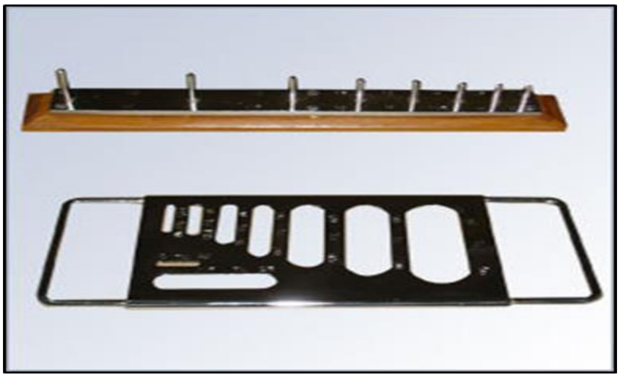


Figure 3.16 Flakiness index gauge and elongation index gauge

Figure 3.17 Coarse sieve shaker

Figure 3.18 Aggregate sample to be tested

Figure 3.19 Weighing Balance

The aggregates are first sorted on the size of test sieves to determine the Flakiness Index. Each group is weighted and tested for thickness when the thickness gage is properly opened by passing each particle through the specified thickness slot along the slightest dimension, repeat this for all the fractions. For each fraction, the weight of particles passing through the thickness is recorded. This is the Flaky particle weight.

The aggregates are first sorted on the size of test sieves to determine the Elongation Index. In a longitudinal gauge, each group is weighted and tested. The aggregate pieces from each fraction are separated and weighed, which do not pass through the specified length gauge with its long side. They are called the Particles of Elongation. For all the fractions, repeat this process.

Figure 3.20 Elongation test

3.3.2 Asphalt Binder Testing

3.3.2.1 Penetration Test

The principles of bitumen penetration are defined as the distance (in 1/10 mm) to which a conventional probe penetrates the fabric under established circumstances of moment, charging and heat. The load utilization is 100 g, which at a temperature of 25 ºC is applied vertically to the bitumen specimen for 5 seconds. The experiment's goal is to determine bitumen hardness and consistency before it can be applied on the road. The apparatus used for this test is penetration needle, water bath, penetration container, asphalt sample, thermometer and penetrometer. while, the material used was the bitumen 60/70 and the cellulose fiber.

Figure 3.21 Water bath

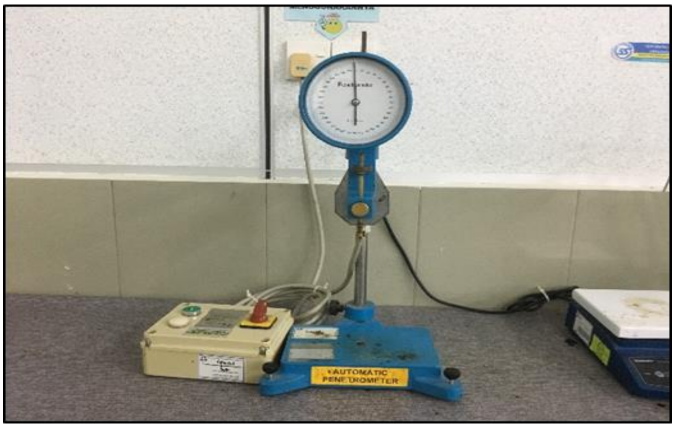


Figure 3.22 Penetrometer

Figure 3.23 Penetration Container

The penetration device is defined worldwide in many norms but always has the same fundamental ASTM D5 specifications. The samples are ready at the specified temperature in a test box and put in a water tub. The wound of entry is washed and corrected to the owner and guide of the device. The exactly dimensioned needle is charged to 100 + 0.05 g for ordinary testing. We also need to ensure that the reading of the penetrometer panel is set to zero. To pierce the bitumen for 5 + 0.1s, the tool holder is removed while the specimen temperature is retained at 25 + 0.1ºC. The range was assessed in 10ths of a mm (deci-millimeter, dmm). After that, it is necessary to read and record the depth of penetration. The same operation is performed by incorporating 0.2%, 0.3%, 0.4%, 0.5% and 0.6% of cellulose fiber as binder modifier.

Figure 3.24 Pouring the bitumen into Penetration Container

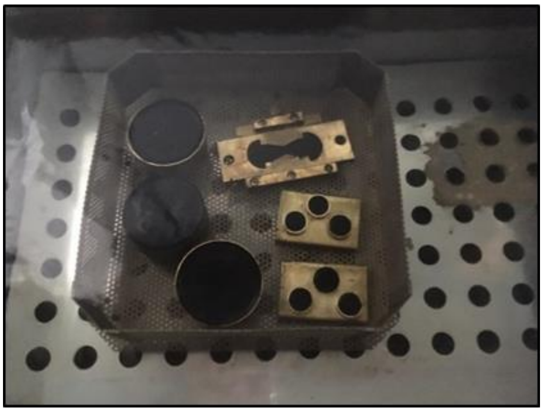


Figure 3.25 The specimens in a water bath

3.3.2.2 Softening Point Test

The objective of the experiment is to determine the temperature at which given bitumen reaches a certain degrees of softness.The apparatus for this experiment is using the diameter steel ball of 9.53 mm and weighing 0.05g, ball guide and ring holder, tapered ring made of brass and conforming to the standard, thermometer, beaker and burner.

Figure 3.26 Steel ball and ring holder

Figure 3.27 Tapered ring made of brass

Figure 3.28 Steel balls is being placed on the surface of the bitumen in the ring

For this softening point, we started with melted the bitumen and the liquid is poured into a pair of ring placed on plate. Thermometer is placed in the center of ring holder leveled with the bottom of the ring. After specimen has cool, ring is suspended in the distilled water in the bath/ beaker at 5^{o}C $\pm$ 2^{o}C. Bath temperature is maintained at that temperature for 15 minutes. After that**,** the steel balls is being placed on the surface of the bitumen in the ring. Stirred and heated the bath liquid to 5^{o}C $\pm$ 2^{o}C per minutes and the temperature is noted just after the ball is passed and dropped into the base plate. The same procedure is repeated with mixed the bitumen within the adding of each variant amount of cellulose fiber.

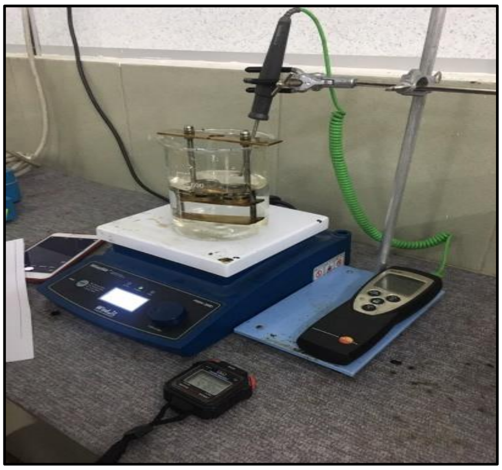


Figure 3.29 Softening point test

3.3.2.3 Ductility Test

The objective of the ductility test of bitumen is to measure the ductility of a given sample of bitumen and to determine the suitability of bitumen for its use in road construction and the apparatus required for ductility test of bitumen is briquette mould. It is made of brass. Circular holes are provided at ends called clips to grip the fixed and movable ends of the testing machine, water bath: A bath maintained within 27.0° ±0.1 °C of the specified test temperature containing not less than 10 litres of water and testing machine

Figure 3.30 Briquette mould

Figure 3.31 Water bath

Figure 3.32 Testing Machine

Firstly, the bituminous test material was being melted completely at a temperature of 75°C to 100 °C above the approximate softening point until it becomes thoroughly fluid. After that, the fluid must be strain in sieve. After stirring the fluid, it being poured in the mould assembly and place it on a brass plate. After about 30-40 mins, the plate being keep assembly along with the sample in a water bath. The sides of the mould being removed. After that, the clips being hook carefully on the machine without causing any initial strain. Lastly, the machine was being started and the chips are pulled horizontally at a speed of 50 mm per minute. The distance at which the bitumen thread of the specimen breaks had being noted to be recorded.

Figure 3.33 Ductility test

3.4 Performance Test

3.4.1 Marshall Mix Design

Marshall Mix Design process was carried out in accordance with the Malaysian Public Works Department JKR/SPJ/2008, 2008 (JKR, 2008). Two specimen were prepared for each binder within the range of 0.2% - 0.6% of cellulose fiber for SMA within the increment of 0.1%. The bulk specific gravity, the stiffness, the stability and flow value, the percentage air voids in mineral aggregate (VMA) and the percentage of air voids in the compacted mix (VIM) was obtained and plotted separately against the fiber content and a smooth curve was drawn through the plotted values.

The objective of marshall mix design is to prepare standard specimens of asphalt concrete for the determination of stability and flow in the marshall apparatus and to determine density, percentage air voids and percentage of aggregate voids filled with binder. The apparatus required for this test is marshall compactor , mixer , water bath, marshall compression machine, marshall mould, sieve shaker and oven.

Figure 3.34 Marshall Compactor Machine

Figure 3.35 Marshall Compression Machine

Figure 3.36 Marshall Mould

Figure 3.37 Condition of sample after testing

3.4.2 Resilient Modulus

Resilient modulus test was conducted to determine if the addition of cellulose fiber brought any significant change in the stiffness properties of modified mixtures. Moreover, to indicates the ability of compacted specimen to recover from repeated load cycles without reaching the failure limit by measuring the time dependent deformation under constant compressive stress. The apparatus required for this test was the universal testing machine (UTM).

The test was conducted under the indirect tensile mode using (UTM) at a controlled temperature of 25oC and 40oC with two orientations of 0o and 90o and three different pulse repetitions (1000ms, 2000ms and 3000ms) accordance to ASTM D 4123. Two specimen for each modified cellulose fiber asphalt binder (0%, 0.2%, 0.3%, 0.4%, 0.5% and 0.6%) were tested. Resilient modulus was used to be selected as an index for evaluating fatigue, stripping and low temperature cracking of asphalt mixtures. The result of resilient modulus obtained by the total resilient axial deformation response of a specimen subjecting with the cylindrical specimen to loads using Universal Testing Machine (UTM).

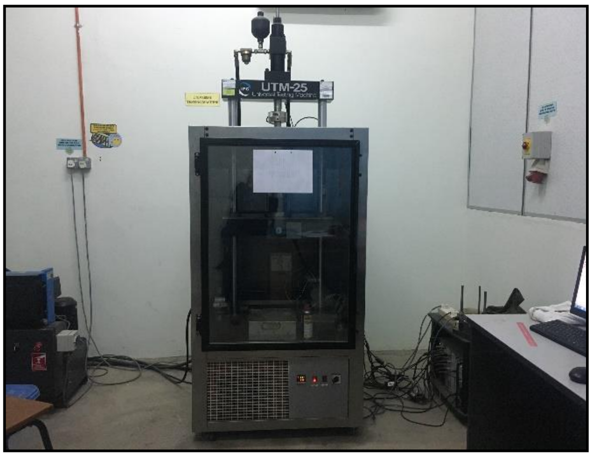


Figure 3.38 Universal Testing Machine

Figure 3.39 Resilient Modulus Test

Figure 3.40 Condition of samples after Resilient Modulus Test

3.4.3 Dynamic Creep

The dynamic creep test was conducted to evaluate the effect of cellulose fiber on modified asphalt binder of SMA. The apparatus required for this test was the universal testing machine (UTM). The result were obtained by the response of a specimen subjecting with the cylindrical specimen to loads using Universal Testing Machine (UTM) Two specimen for each cellulose fiber modified asphalt binder content (0%,0.2%,0.3%,0.4%, 0.5% and 0.6%) were tested. All the samples were tested at fixed temperature of 25^{o}C and 40^{o}C.). Specimen heights and weights were measured to calculate the unit weight of each specimen. The following steps were followed:

Two specimens were tested, while widths and diameters were measured for each cellulose fiber content and temperature combination. At conditional temperatures of 25^{o}C and 40^{o}C before testing, the specimens were placed in a cabinet for 24 hours. The load cell and LVDT were subjected to the specimen. To distribute the applied pressure uniformly on the surface of the specimen, the stainless steel plates were used. Each specimen was subjected to a conditional stress of 100kPa for 3600 second before specimen was loaded. Haversine loading was applied without impact and with loads varying between (0 and 100 kPa). Tests were conducted at temperatures of 25, and 40^{o}C with 3600 cycles. The output of the test was computerized and edited, which made it printable.

Figure 3.41 Dynamic Creep Test

:

3.4.4 Cantabro Loss

The objective of this test is to evaluate abrasion resistance of for of compacted SMA asphalt specimens under moving traffic loads. The apparatus used for this test is Los Angeles abrasion machine.

Figure 3.42 Los Angeles abrasion machine

This test procedure measures the breakdown of compacted specimens utilizing the Los Angeles Abrasion machine. This test involves recording the initial weight of individual sample, placing it in the machine and rotating it for 300 revolutions at the rate of 30 rpm. The percentage of weight loss (Cantabro loss) is an indication of PFC durability and relates to the quantity and quality of the asphalt binder. The percentage of weight loss is measured and reported.

Figure 3.43 Condition of sample after Cantabro Loss testing

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

From this study, the overall result was obtained on the mechanical properties and performance test of the cellulose fiber modified asphalt binder. The tests are conducted to determine the performance of data and to analyze the pattern of the result

4.2 Material Properties

4.2.1 Result for Aggregate Testing

The result of aggregate testing was being observed and recorded. From the table shown below, all of the aggregate testing were fulfilled the limit and passed for the specification of the testing.

Table 4.1 Result of Aggregate Testing

Aggregate Testing	**Observed Test Result**	**Limit**	**Status**	**Reference Standard**
Aggregate Impact Value	15.00 %	< 25.00 %	PASS	MS 30
Aggregate Crushing Value	15.09 %	< 35.00 %	PASS	MS 30
Los Angeles Abrasion	21.00 %	< 25.00 %	PASS	ASTM C 131
Flakiness	24.90 %	< 25.00 %	PASS	MS 30
Elongation	24.90 %	< 25.00 %	PASS	MS 30

4.2.1.1 Aggregate Impact Value (AIV)

The AIV test is done to evaluate the aggregate impact value of coarse aggregate and the toughness of the stone materials. The apparatus used for aggregate impact value are by aggregate impact test machine, sieve, tamping rod and cylindrical metal measure. The lower the percentage of loss, the stronger the aggregate. The percentage of loss for Aggregate Impact Value testing is 15.00%. The result obtained had passed the requirement of MS 30 where the value of percent loss should be less than 25.00%. Aggregate that having AIV % value that less than 50% is better to use in construction, where as those having value more than 50% are bad for construction. It is reliable to calculate the AIV % of the aggregate use as road metal as it will give us the capacity of road to bear the load on it. Aggregate having AIV % equal or greater than 35 % is considered bad to use in road construction. The result obtained is fulfilling the requirement of MS30 where the value of percent loss should be in the range as shown in appendix A1.

4.2.1.2 Aggregate Crushing Value

This ACV test was being conducted to determine the mechanical strength of the aggregate. The lower percentage of loss in aggregate crushing value, the stronger the aggregate. Table 4.3 shown that the percentage of loss for Aggregate Crushing Value testing is 15.09%. The result obtained passed the requirement of MS 30where the value of percent loss should be less than 35.00%. From the result, ACV test had achieved a high quality of pavement as the aggregate possessing low crushing value had being preferred. ACV test can be applied to access the suitability of aggregates with reference to the crushing strength for various type of pavement components. To withstand the stresses due to wheel loads including the steel tyre of loaded bullock- carts, the aggregates used for the surface course of pavements should be strong enough. The result obtained is fulfilling the requirement of MS30 where the value of percent loss should be in the range as shown in appendix A2.

4.2.1.3 Los Angeles Abrasion

The test was conducted to obtain the Los Angeles number in the form of percentage wear of aggregates which reflects their resistance to degradation using the Los Angeles testing machine. The results in table 4.4 shown that the aggregate has Los Angeles Abrasion value 21.00%, this shown that the aggregate perform satisfactory in pavement, dam construction, slope stability, it is sufficiently hard to resist the abrasive effect and quite resistant to crushing effect of traffic over long period of time. The aggregate was categorized in hard rock type as it less than 40% and fulfilled the requirement as it less than 25% limit for SMA20. The other significance for LA abrasion test such as the test will determine the quality of the aggregate and widely used as an indicator of the relative quality or competence of mineral aggregates. The result obtained is fulfilling the requirement of MS30 where the value of percent loss should be in the range as shown in appendix B1.

4.2.1.4 Flakiness And Elongation

The Flakiness Index of aggregates is the percentages by weight of particles whose least thickness is less than 0.6 of their mean dimension. The test is not applicable to sizes smaller than 6.3 mm. While, the elongation index test is the percentage by weight of particles whose greatest length than 1.8 times their dimension. The elongation test is not applicable to sizes smaller than 6.3 mm.

In this test, flakiness index was used as independent variables and other properties such as gradation are kept constant as far as their properties met the specification required for the asphalt mixture. According the classifies aggregate into six classes that are rounded, irregular, angular, flaky, flaky and elongated and elongated. The aggregates that is flaky, elongated, flaky and elongated or equal dimension are determined by the ratio of the shortest, the largest and average diameter of the particles.

The value that we get from this result tests shown in table 4.5 is 25% for flakiness and 24.9% for elongation. The values indicated that it not exceeds 25% of the JKR requirements. The aggregate particles are classified as flaky aggregate as the aggregate thickness is small compared with width and length of that aggregate. The least dimension of aggregate is less than the 60% of its mean dimension then it is said to be flaky aggregate. The result obtained is fulfilling the requirement of MS30 where the value of percent loss should be in the range as shown in appendix C and appendix D.

4.2.2 Result For Asphalt Binder Testing

The result of asphalt binder testing was being observed and recorded. This process was carried out in accordance with ASTM designation and the Malaysian Public Works

Department JKR/SPJ/2008, 2008. From the table shown below, all of the asphalt binder testing were fulfilled the limit and passed for the specification of the testing.

Table 4.2 Result of Asphalt Binder Testing

Aggregate Testing	Observed Test Result	Limit	Status	Reference Standard
Penetration Test in 25oC	60 mm/10	60 – 70 mm/10	PASS	ASTM D 244 & ASTM D 5
Softening Point Test	50oC	49 – 56oC	PASS	ASTM D 244 & ASTM D 36
Ductility Test in 25oC	102.5 cm/s	>100 cm/s	PASS	ASTM D113

4.2.2.1 Penetration Test

The consistency of bituminous materials very depending upon several factors such as constituents, temperature, etc. The penetration test determines the consistency of these materials for the purpose of grading them, by measuring the depth to which a standard needle will penetrate vertically under specified conditions of standard load, duration and temperature. The softer the bitumen, the greater will be the penetration. Based on the table 4.2, the value of penetration was 60 mm/10 that is in the range of specification and passed the requirements.

It may be considered that any inaccuracy such as the pouring temperature, the size of the needle, the weight placed on the table, the temperature of the test and the length of release of the penetration needle should be influenced by the penetration value. High penetration values can be obtained if the temperature and/or weight increase. In addition, higher test temperatures give significantly higher levels of penetration. Higher pouring temperatures than specified can cause bitumen to harden and can result in lower penetration values. In order to obtain consistent results, it is also necessary to keep the needle clean before testing. The needle of penetration should not be placed from the side of the dish more than 10 mm. Mostly passed bitumen penetration test to determine the material's hardness. Using the bitumen of various degrees of penetration depends on the climatic condition and construction type. Bitumen with higher penetration values is used in colder regions while lower penetration levels are preferred in warmer regions..

4.2.2.2 Softening Point Test

The softening limit is the rate at which the material reaches a specific degree of softening under the given experiment situation. Usually it is determined by the Ring and Ball test for bitumen. The result achieved in the softening point value was 50°C, which is within the specification range and passed the requirements. As with the other bitumen physical tests, the specification limit is essential. The softening point may be affected by liquid quality and type, ball weight, distance between ring bottom and base plate bottom, and heating rate.

Water or glycerin impurity has been observed that can have a significant impact on the result. If the weight of the ball is excessive and the increased distance between the bottom of the ring and the bottom plate may increase the softening point, it indicates a lower softening point. The temperature at which the bituminous binders have an equal viscosity is essentially the softening point. In warmer places, bitumen with higher softening point can be selected. To specify hard bitumen and pitches, softening point is also being used sometimes.

4.2.2.3 Ductilty Test

For the asphalt bitumen, the ductility result was 102.5 cm / s, which is more than 100 cm / s in the specification. The true strain-rate decreases with time and elongation due to constant cross-head speed conditions in the asphalt ductility test. This decrease tends to cause the strain rate to change only after a few cm of elongation by more than an order of magnitude. The higher the temperature of the test, the higher the value of ductility as the higher level of stress of failure corresponds to lower temperature of the tets and higher stress rates.

4.3 Performance Tests

4.3.1 Marshall Stability

From this test, there were 6 graphs was plotted with values of fiber content against the value of bulk density, Marshall Stability, stiffness, flow value, the percentage air voids filled with asphalt (VFA), and the percentage of voids in total mix (VTM) obtained from this Marshall test.

i) Marshall Stability

Based on the figure 4.1, the fiber content of 0.2% that is 6379.20 N and was the highest stability value and was being selected as the most effective amount for the enhancement in SMA mixtures. It was found from the previous research that the stability value is low at lower fiber content. Thus, the stability value increases with the increase in fiber content, attains maximum and finally reduces with further increase in fiber content. The stability of the mix is primarily controlled by the cohesion and internal friction of the matrix which supports the coarse aggregate.

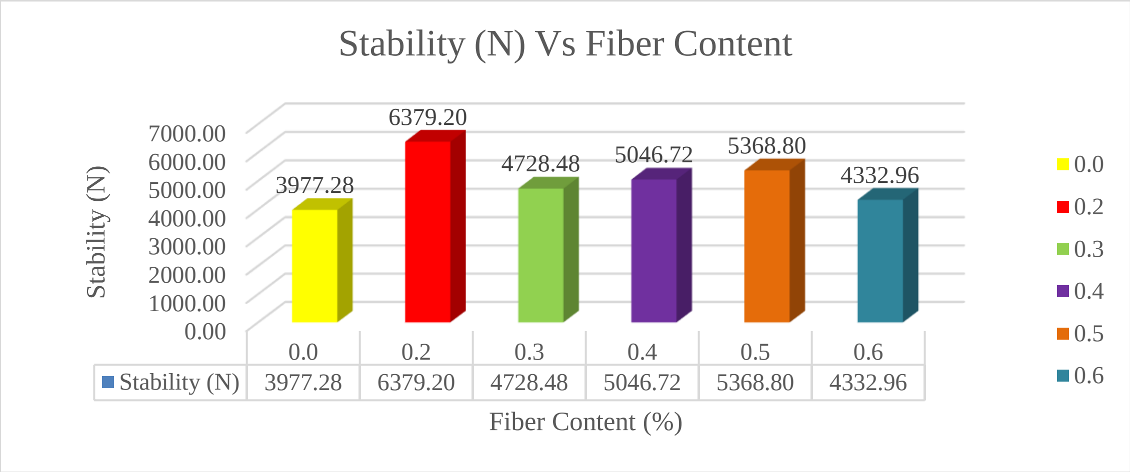


Figure 4.1 Graph of Stability vs Fiber content

ii) Bulk density

Based on the figure 4.2, the fiber content of 0.4% that is 2.22 g and was the highest density value and was being selected as the most effective amount for the performance enhancement in SMA mixtures.

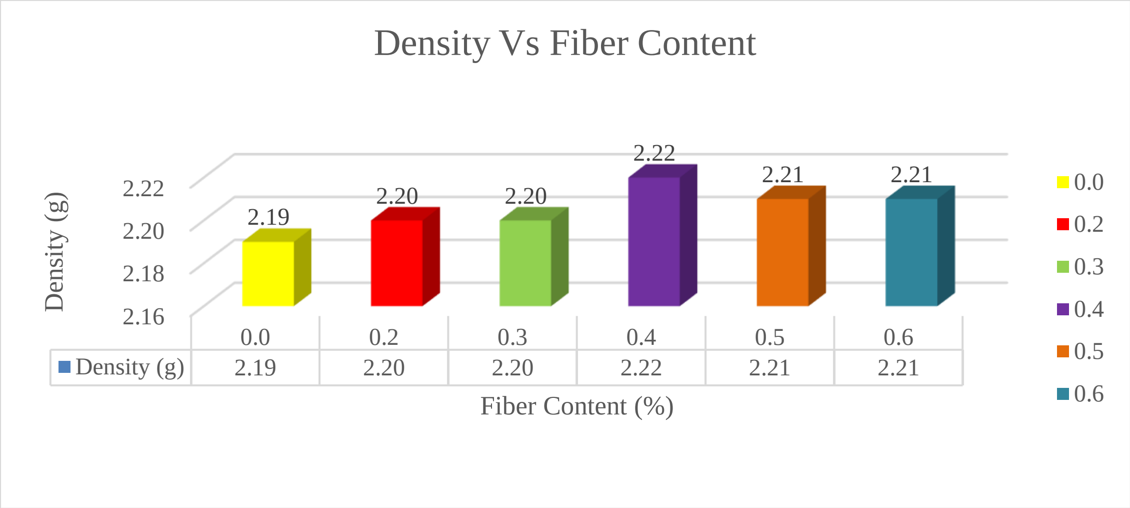


Figure 4.2 Graph of Density vs Fiber content

ii) Stiffness

Based on the figure 4.3, the fiber content of 0.5% that is 924.86 N/mm and was the highest stiffness value and was being selected as the most effective amount for the performance enhancement in SMA mixtures.

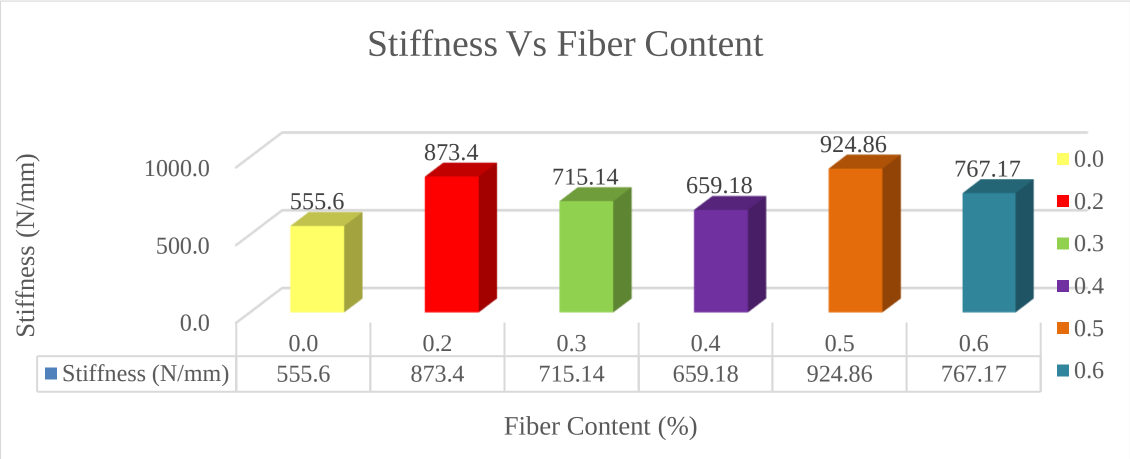


Figure 4.3 Graph of Stiffness vs Fiber content

iv) Flow value

Based on the figure 4.4, the fiber content of 0.0% that is 3.77 mm and was the highest flow value and was being selected as the most effective amount for the performance enhancement in SMA mixtures.

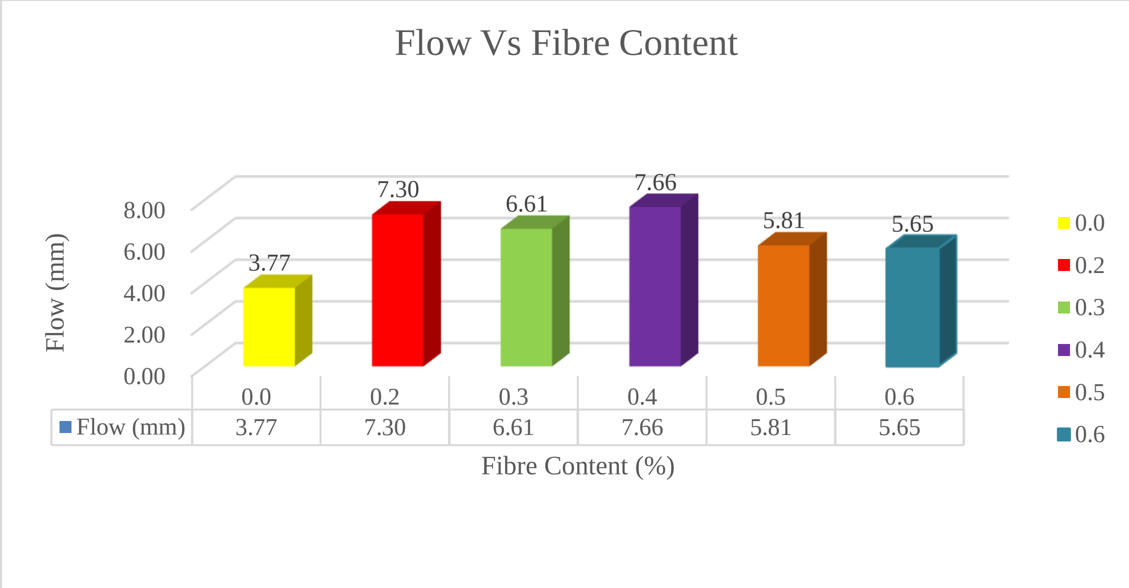


Figure 4.4 Graph of Flow vs Fiber content

v) Percentage of voids in total mix (VTM)

Based on the figure 4.5, the fiber content of 0.6% that is 14.34 % and was the highest percentage of voids in total mix (VTM) value and was being selected as the most effective amount for the performance enhancement in SMA mixtures.

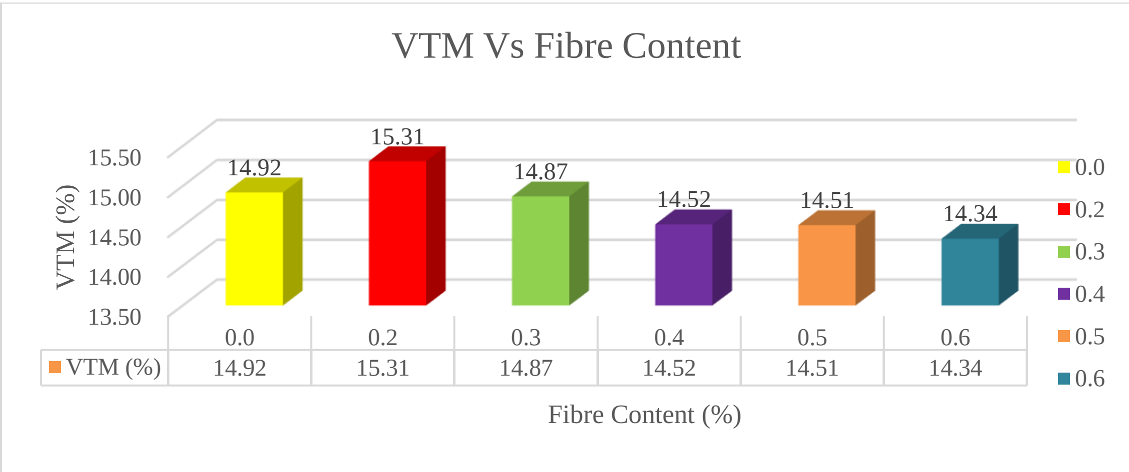


Figure 4.5 Graph of VTM vs Fiber content

vi) Percentage air voids filled with asphalt (VFA)

Based on the figure 4.6, the fiber content of 0.6% that is 8.22% and was the highest percentage air voids filled with asphalt (VFA) value and was being selected as the most effective amount for the performance enhancement in SMA mixtures.

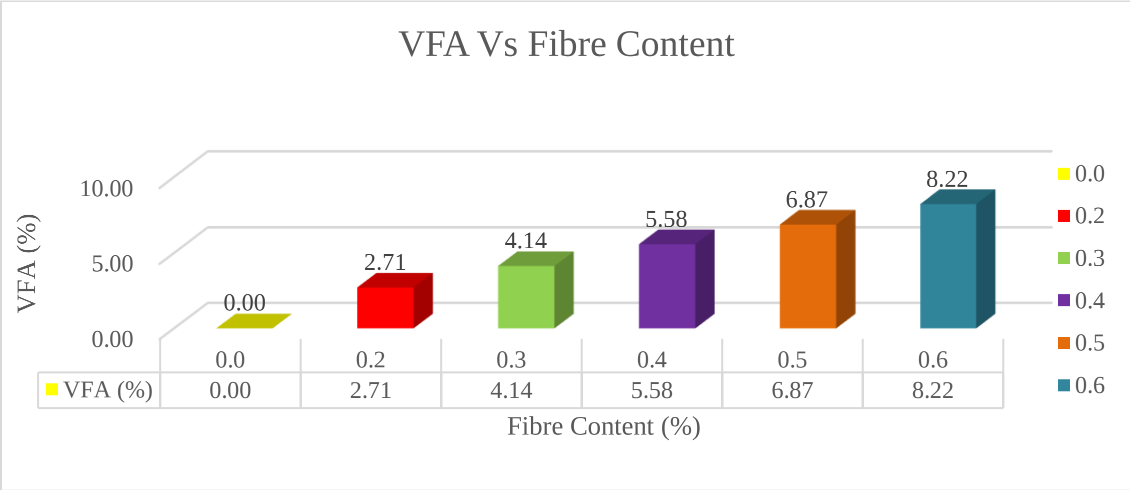


Figure 4.6 Graph of VFA vs Fiber content

4.3.2 Cantabro Loss

Refer from the figure below, in 300 rep, 0.3% of fiber content indicated that it was the highest value for abrasion and 0.2% of fiber content was the lowest value of abrasion loss. This is because the low the abrasion, the higher the tensile strength of the SMA. The cellulose fiber helps to promote the hardness of modified asphalt binder that can improve the performance of the SMA. The fiber content of 0.2% is the lowest loss abrasion and was being selected as the most effective amount for the performance enhancement in SMA mixtures. The cellulose fiber modified asphalt binder perform satisfactory in pavement, dam construction, slope stability, it is sufficiently hard to resist the abrasive effect and quite resistant to crushing effect of traffic over long period of time.

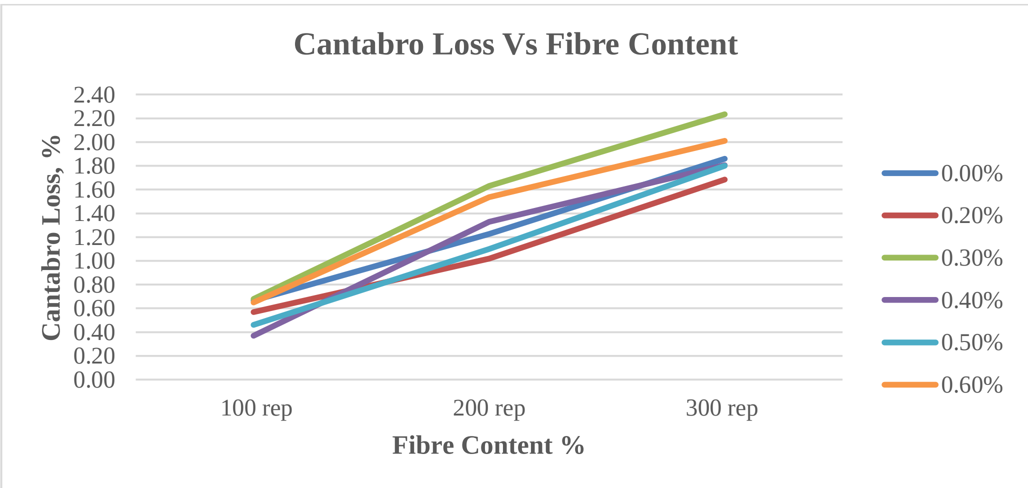
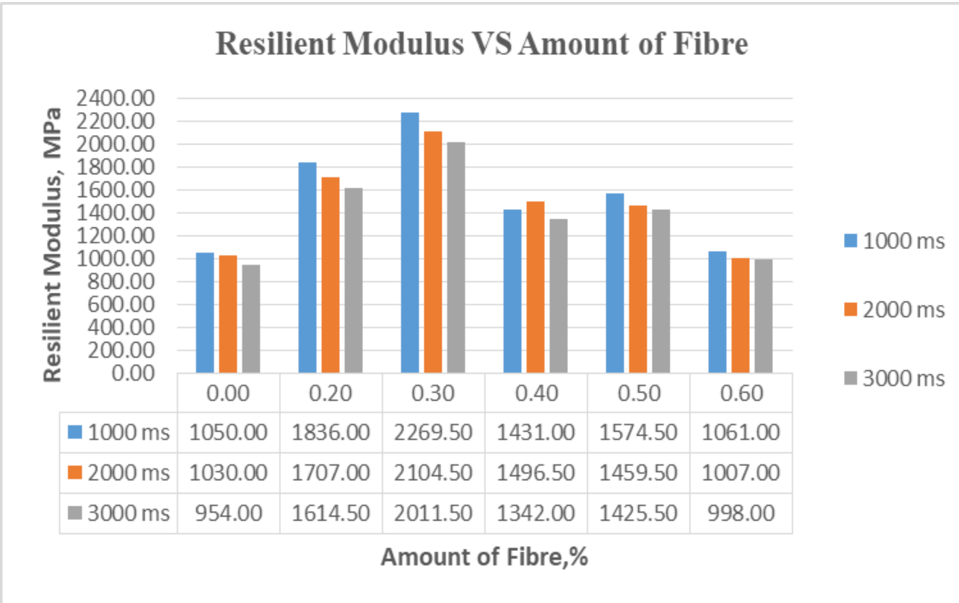


Figure 4.7 Graph of Cantabro Loss vs Fiber content

4.3.3 Resilient Modulus

Refer from figure 4.8 below, the fiber content of 0.3% is the highest of resilient modulus value in 25^{o}C is being selected as the most effective amount for the performance enhancement in SMA mixtures. For SMA20, the increment in the amount of cellulose fiber will result in a higher value of resilient modulus. For 25^{o}C, pulse at 1000ms period, the resilient modulus for 0.0% CF-SMA20 is 1050 Mpa. The highest M_r value obtain at 0.3% CF-SMA20 is 2269 Mpa. However, the result for resilient modulus values at different pulse period did not show any trend in uniformity. While, pulse at 3000ms period, the resilient modulus for 0.3% CF-SMA20 is 2011 Mpa. In general, higher pulse repetitive period loads the lower as the resilient modulus value. The figures shows 0.3% of CF is consider as the optimum amount for enhancing SMA performance.



	0.00	0.20	0.30	0.40	0.50	0.60
1000 ms	1050.00	1836.00	2269.50	1431.00	1574.50	1061.00
2000 ms	1030.00	1707.00	2104.50	1496.50	1459.50	1007.00
3000 ms	954.00	1614.50	2011.50	1342.00	1425.50	998.00

Figure 4.8 Graph of Resilient Modulus vs Fiber content in 25^{o}C

Refer from figure below, the fiber content of 0.6% is the highest of resilient modulus value in 40^{o}C is being selected as the most effective amount for the performance enhancement in SMA mixtures. For SMA20, the increment in the amount of cellulose fiber will result in a higher value of resilient modulus. For 40^{o}C, pulse at 1000ms period, the resilient modulus for 0.0% CF-SMA20 is 475 Mpa. The highest M_r value obtain at 0.6% CF-SMA20 is 1181 Mpa. However, the result for resilient modulus values at different pulse period did not show any trend in uniformity. While, pulse at 3000ms period, the resilient modulus for 0.6% CF-SMA20 is 1002.50 Mpa. In general, higher pulse repetitive period loads the lower as the resilient modulus value. The figures shows 0.6% of CF is consider as the optimum amount for enhancing SMA performance.

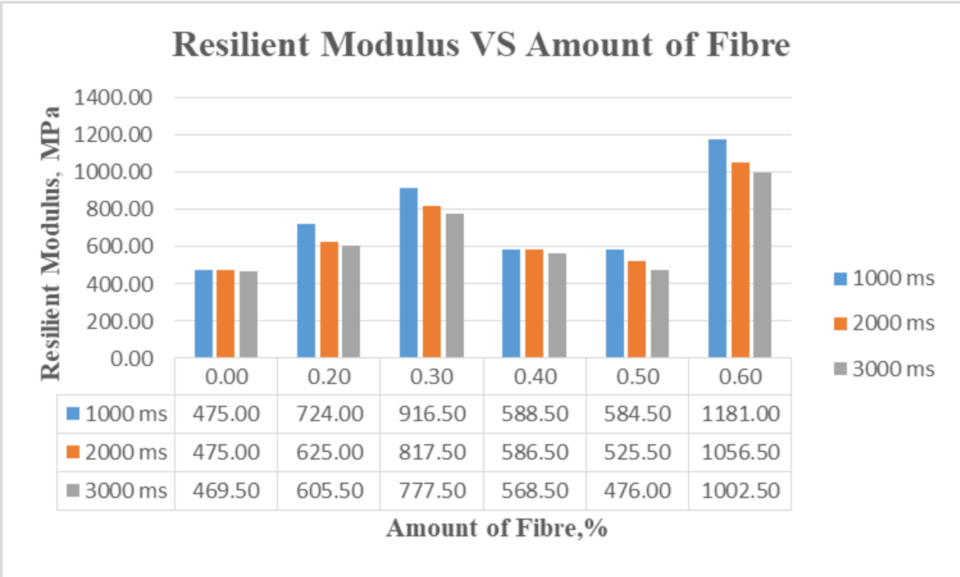


Figure 4.9 Graph of Resilient Modulus vs Fiber content in 40^{o}C

The performance of cellulose fiber in resilient modulus in 25^{o}C is more effective and give the best performance to be selected than resilient modulus in 40^{o}C based on the previous research. As expected, increasing temperature led to increase in asphalt concrete deformation which is due to lower asphalt cement viscosity in higher temperatures. Higher fiber content is needed when temperature is increased. In this situation a part of fiber is used to prevent asphalt drain down and adding fiber has not shown any remarkable impacts on mixture strength in low contents, so more fiber is needed to improve mixture property.

4.3.4 Dynamic Creep

Based on figure below, it presents the effect of 3600 strain cycles of dynamic creep by the cellulose fiber asphalt binder in 25^{o}C. From the figure, the pattern shows the same trend curve with a sudden growth in the first 100 load cycles and then slowly increase strain from 100 to 3600 cycles except for the 0.6% as its trend curve are tends to steadily in uniform. The result of the 0.6% of fiber content is the lowest micro strain over no of cycles that give the best result for the performance of dynamic creep test compared to unmodified asphalt mixture. The lower the strain per cycle, the higher the resistance from the creeps in SMA mixtures. The lowest strain per cycle is distributed by the cellulose fiber addition as the asphalt binder modifier for enhancing the performance of SMA. The adding fiber make a good results in the asphalt binder of SMA that it can withstand from the creeps that occurred usually on the pavement road.

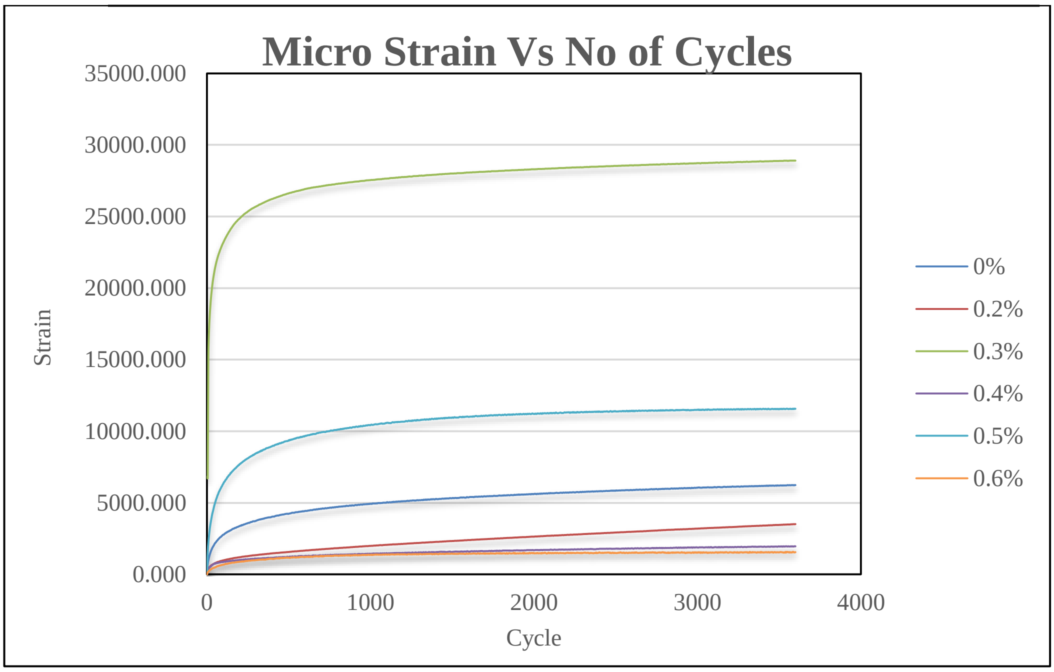


Figure 4.10 Graph of Dynamic Creep (Micro strain vs No of cycles) in 25°C

Based on figure below, it presents the effect of 3600 strain cycles of dynamic creep with loading strain by the cellulose fiber asphalt binder in 40°C. From the figure, the patterns shows all the unmodified and modified asphalt mixture was sharply increased. After the 100 loading of cycles, the pattern was steadily increase from 100 to 3600 cycles. The result of the 0.2% of fiber content give the slightly lowest value compared to unmodified asphalt mixture. The lower the strain per cycle, the higher the resistance from the creeps in SMA mixtures. The lowest strain per cycle is contributed by the cellulose fiber addition as the asphalt binder modifier for enhancing the performance of SMA. Adding fiber make a good combination of fiber and asphalt cement and the result is better compressive and tensile strength of asphalt mixtures. Thus, the 0.2% of modified asphalt mixture indicates the optimum fibre in dynamic creep. The performance of cellulose fiber in dynamic creep in 40°C is more effective and give the best performance to be selected than in 40°C based on the previous research.

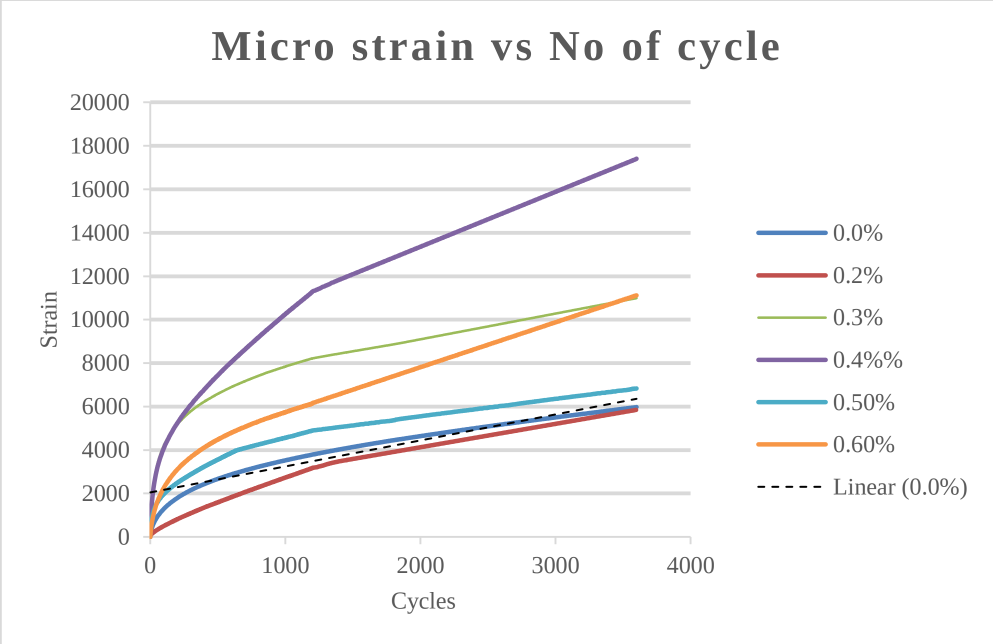


Figure 4.11 Graph of Dynamic Creep (Micro strain vs No of cycles) in 40ºC

4.4 Optimum Fiber Content

From all the performance tests, the optimum fibre content is 0.2% because the lowest value obtained in table 4.8, for the total ranking for all tests is 0.2% in addition of cellulose fiber in asphalt mixture.

Table 4.9 Ranking Result for Optimum Fiber Content

Fibre Content (%)	**0.0**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**
Resilient Modulus (25ºC)	2	3	1	5	4	6
Resilient Modulus (40ºC)	6	3	2	4	5	1
Dynamic Creep (25ºC)	4	3	6	2	5	1
Dynamic Creep (40ºC)	2	1	5	6	3	4
Stability	6	1	4	3	2	5
Stiffness	6	4	2	1	5	3
Density	4	3	3	1	2	2
Cantabro loss	4	1	6	3	2	5
Total marks	34	19	29	25	28	27

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Based on the range of cellulose fiber contents used, and the results of the test conducted, the following conclusion are drawn:

1) In terms of resilient modulus, the result indicated that the higher pulse repetitive period loads, the lower the resilient modulus value & better performance in modified fiber content in 25^{o}C. The addition 0.3% of cellulose fiber is consider as the optimum amount for enhancing the SMA performance and gain 53.73% of fiber improvements.

2) In terms of Marshall Stability, the result show that the stability value increases with the increase in fiber content, attains maximum and finally reduces with further increase in fiber content. The stability value is low at lower fiber content. The fiber content of 0.2% was the highest stability value and gain improvement for about 37.65%.

3) In terms of dynamic creep, the result indicated that the lower the strain per cycle, the higher the resistance from the creeps in SMA mixture. The addition 0.2% of cellulose fiber is consider as the optimum amount for enhancing SMA performance and SMA perform the dynamic creep better in 40^{o}C than 25^{o}C and gain improvement of 2.13%.

4) In terms of Cantabro Loss, give the result that the 0.2% of fiber content was the lowest value of abrasion loss, this indicated with the low the abrasion, the higher the strength of the SMA. The cellulose fiber helps to promote the hardness of modified asphalt binder that can improve the performance of the SMA and gain improvement of 10.06%.

5) From the ranking result of performance tests of this research, it was determined that 0.2% is the optimum fiber content cellulose fiber modified asphalt binder (SMA20).

5.2 Recommendation

From the result, based on the discussion and observation during this study, there are several recommendations that can be given to improve further study in this topic:

1) The used of cellulose fiber gives a positive feedback on the enhancement of SMA mixture as it can increased the resistance to rutting and cracking. However, it is recommended to evaluate the life cycle cost analysis to support the use of fibers in a cost effective and financially sustainable addition to asphalt pavement mixture.
2) Based on the result of performance test, it is recommended to conduct the resilient modulus test in 25^{o}C and dynamic creep in 40^{o}C for the mechanical performance of cellulose fiber- stone mastic asphalt (SMA), as it gives the effective result on the performance of SMA mixture. Therefore, the fixed temperature for the following test need to be conducted in order to improve the future study.
3) This study was limited to one binder type and one asphalt mixture. Further investigation is recommended to include different aggregate gradations, fiber addictive types, and asphalt binder types.
4) The additional laboratory performance tests such as the moisture susceptibility are also recommended to be incorporated with the SMA into future research studies.
5) The cellulose fiber is recommended to use as the addictive of asphalt binder as it can improve the viscosity of the unmodified asphalt and at the same time improved the tensile strength and the stiffness of SMA.

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APPENDIX A
RESULT FOR AGGREGATE IMPACT VALUE

Table 5.1 Result for Aggregate Impact Value

Sample	Aggregate size (mm)	Weight of Aggregate (g)			% Loss	Ave % Loss
		Before test (M_1)	Retain at 2.36 mm sieve (M_2)	Passing at 2.36 mm (Loss) (M_3)		
A	10	293.66	251.6	43	15	15

$$Aggregate\ Impact\ Value\ (\%) = \frac{Weight\ Loss\ (M_3)}{Initial\ Weight\ (M_1)} \times 100\ \%$$

APPENDIX B
RESULT FOR AGGREGATE CRUSHING VALUE

Table 5.2 Result for Aggregate Crushing Value

Sample	Aggregate Size (mm)	Weight of aggregate before (g)	Weight pass sieve 2.36mm (g)	Aggregate Crushing Value (%)
		M1	M2	
A	20-14	3001.6	453.02	15.09
B	14-10	3000.24	358.83	11.96

ACV (%) = (M2 / M1) x 100%

APPENDIX C
RESULT OF LOS ANGELES ABRASION TEST

Table 5.3 Result of Los Angeles Abrasion Test

Sample	Aggregate size (mm)	Weight of Crushed Aggregate(g)			% Loss
		Before (m1)	After (m2)	Loss (m3)	
A	20	5001.4	3941.77	1059.63	21.18
B	14				

$$LA\ Abrasion\ Value\ (\%) = \frac{weight\ loss\ (m3)}{Initial\ weight\ (m1)}\ x\ 100$$

APPENDIX D
RESULT OF FLAKINESS TEST

Table 5.4 Result of Flakiness Test

Sieve Size (mm)	Total Wt. of Aggregates Retained (g)	Wt. Retained on Flakiness Gauge (g)	Wt Passing on Flakiness Gauge (g)
50	0	0	0
37.5	0	0	0
20	224.45	174.21	50.24
14	582.45	491.91	90.54
12.5	98.52	84.17	14.35
10	158.6	113.44	45.16
6.3	248.76	119.17	129.59
Total	1312.78	982.9	329.88

Flakiness = 1500.11 g

Percentage of Flakiness Index:

(Total Wt. of Agg. Ret.on Flakiness Ga./Wt. Passing on Flakiness Ga.)(100)

= 329.88/1312.78 x 100% = 25%

APPENDIX E
RESULT OF ELONGATION TEST

Table 5.5 Result of Elongation Test

Sieve Size (mm)	Total Wt. of Aggregates Retained (g)	Weight Retained on Elongation Gauge (g)	Wt Passing on Elongation Gauge (g)
50	0	0	0
37.5	58.36	58.36	0
20	134.17	119.48	14.69
14	422.26	337.31	84.95
12.5	160.61	101.58	59.03
10	160.6	126.29	34.31
6.3	272.44	164.96	107.48
Total	1208.44	907.98	300.46

Elongation = 1500.74 g

Percentage of Elongation Index:

(Total Wt. of Agg. Ret.on Elongation Ga./Wt. Passing on Elongation Ga.)(100)

= 300.48/1208.44 x 100% = 24.9%

APPENDIX F
RESULT FOR CANTABRO LOSS VS FIBER CONTENT

Table 5 6 Result for Cantabro Loss vs Fiber Content

% Fibre	**Cantabro Loss %**		
	100 rep	**200 rep**	**300 rep**
0.00	0.67	1.23	1.86
0.20	0.57	1.02	1.69
0.30	0.68	1.63	2.24
0.40	0.37	1.33	1.81
0.50	0.46	1.10	1.80
0.60	0.65	1.54	2.01